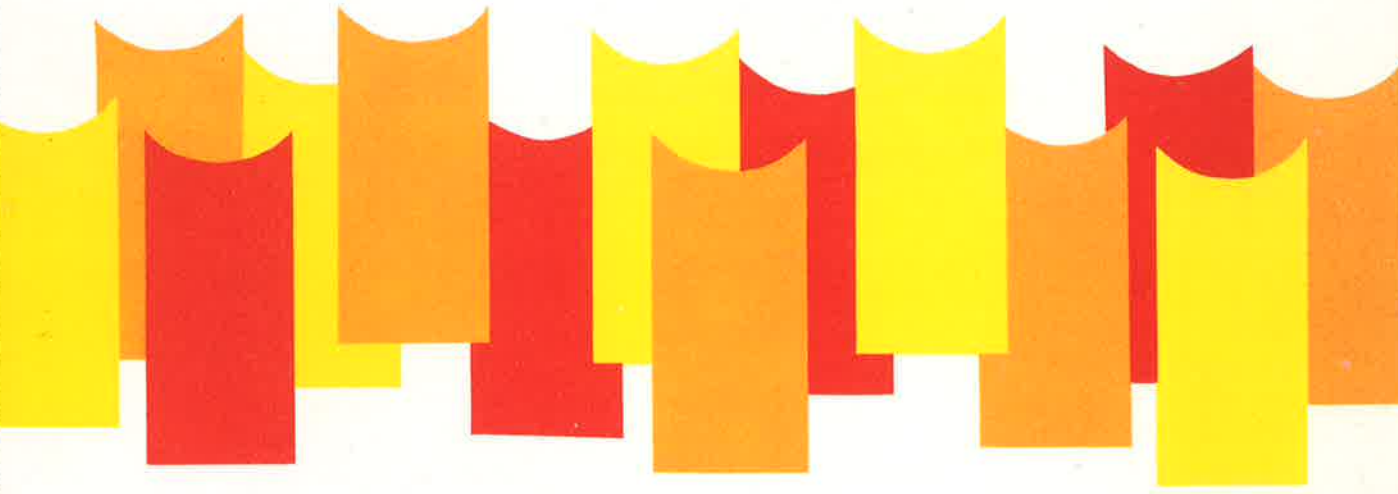
# WATER & SOIL

MISCELLANEOUS PUBLICATION

No. 49

# PROCEEDINGS OF THE RIVER AND ESTUARY MIXING WORKSHOP, HAMILTON, 17-18 NOVEMBER 1981



NATIONAL WATER AND SOIL CONSERVATION ORGANISATION

ISSN 0110-4705

## WATER AND SOIL MISCELLANEOUS PUBLICATIONS

1. Rainfalls and floods of Cyclone Alison, March 1975, on the north-eastern Ruahine Range. P. J. Grant, N. V. Hawkins, W. Christie. ($1) 1978
2. Water quality research in New Zealand 1977. Sally F. Davis ($2.50) 1978
3. Liquid and waterborne wastes research in New Zealand 1977. S. F. Davis ($2) 1978
4. Synthetic detergents working party report. ($1) 1978
5. Water quality control committee report. ($1) 1978
6. Suggestions for developing flow recommendations for in-stream uses of New Zealand streams. J. C. Fraser. ($1) 1978
7. Index to hydrological recording stations in New Zealand 1978. ($2) 1978
8. Water rights for the Clyde Dam, Clutha hydro power development. ($1.50) 1979
9. Index to hydrological recording stations in New Zealand 1979. ($2) 1979
10. Water quality research in New Zealand 1978. Denise F. Church. ($3) 1980
11. Liquid and waterborne wastes research in New Zealand 1978. D. F. Church. ($2) 1980
12. Catchment register for New Zealand, Volume 1. ($8) 1981
13. New Zealand recreational river survey. Pt 1: Introduction. G. D. and J. H. Egarr. ($5) 1981
14. New Zealand recreational river survey. Pt 2: North Island rivers. G. D. and J. H. Egarr. ($5) 1981
15. New Zealand recreational river survey. Pt 3: South Island rivers. G. D. and J. H. Egarr. ($12) 1981
16. Waimea East irrigation scheme information booklet. (Out of stock) 1980
17. Hawke's Bay area planning study: Urban capability assessment. ($4) 1980
18. Index to hydrological recording stations in New Zealand 1980. ($2) 1980
19. Rakaia water use and irrigation development. D. R. Maidment, W. J. Lewthwaite, S. G. Hamblett. ($3) 1980
20. Water quality research in New Zealand 1979. B. J. Biggs. ($4) 1980
21. Liquid and waterborne wastes research in New Zealand 1979. B. J. Biggs. ($2) 1980
22. Baseline water quality of the Manawatu water region 1977–78. K. J. Currie, B. W. Gilliland. ($3) 1980
23. Effects of land use on water quality—a review. R. H. S. McColl and Helen R. Hughes. ($5) 1981
24. Summaries of water quality and mass transport for Lake Taupo catchment, New Zealand. C. J. Schouten, W. Terzaghi, Y. Gordon. ($5) 1981
25. The report of the water quality criteria working party. ($3) 1981
26. Handbook on mixing in rivers. J. C. Rutherford ($8) 1981
27. Index to hydrological recording stations in New Zealand 1981. ($2) 1981
28. Bibliography of oceanography and sedimentology for the Northland-Auckland coast. T. F. W. Harris and T. Hume. ($3) 1981
29. Aquatic oxygen seminar proceedings. Hamilton, November 1980. ($10) 1982
30. Future groundwater research and survey in New Zealand. ($3) 1982
31. Land and water resource surveys of New Zealand: map coverage and reference lists. C. L. Clark. ($10) 1982
32. A procedure for characterising river channels. M. P. Mosley. ($8) 1982
33. The United States Environmental Protection Agency's 1980 ambient water quality criteria: a compilation for use in New Zealand. D. G. Smith. ($5) 1982
34. Water quality research in New Zealand, 1981. J. S. Gifford. ($5) 1982
35. Liquid and waterborne wastes research in New Zealand, 1981. J. S. Gifford. ($3) 1982
36. New Zealand river temperature regimes. M. P. Mosley. ($8) 1982
37. Landslip and flooding hazards in Eastbourne Borough—a guide for planning. ($8) 1982
38. Physical and chemical methods for water quality analysis. ($5) 1982
39. A guide to the common freshwater algae in New Zealand. ($5) 1982
40. Peatlands policy study; reports and recommendations. ($5) 1982
41. Index to hydrological recording stations in New Zealand 1982. ($5) 1982
42. A draft for a national inventory of wild and scenic rivers: Part 1, nationally important rivers. ($2) 1982
43. A review of land potential in the Bay of Plenty—Volcanic Plateau region. ($10) 1982
44. An approach to stormwater management planning. ($5) 1982
45. Catchment management for optimum use of land and water resources: Documents from an ESCAP seminar. Part 1—Introductory and country statements. ($10) 1982
46. Catchment management for optimum use of land and water resources: Documents from an ESCAP seminar. Part 2—New Zealand contributions. ($10) 1982
47. River Low Flows: Conflicts of water use. ($3) 1982
48. Catchment control in New Zealand. A. L. Poole. ($15) 1983

WATER & SOIL MISCELLANEOUS PUBLICATION No. 49

# PROCEEDINGS OF THE RIVER AND ESTUARY MIXING WORKSHOP, HAMILTON, 17-18 NOVEMBER 1981

Edited by

**J. C. Rutherford**

Water and Soil Science Centre
Ministry of Works and Development
Hamilton

WELLINGTON 1982

**Proceedings of the River and Estuary Mixing Workshop, Hamilton, 17-18 November 1981**

Water & Soil Miscellaneous Publication No. 49, 1982, 157p. ISSN 0110-4705.

These are the proceedings of the 1981 workshop of an annual series held on a topic which Hamilton Water and Soil Science Centre staff have been studying and which is of importance to water resource managers.

Included here are the short papers speakers presented and transcripts of the discussions which followed.

The papers focussed on mixing in rivers and tidal estuaries and emphasised practical procedures, difficulties, and achievements.

The majority described case studies from Taranaki, the Waitemata Harbour, Huntly, and other parts of New Zealand.

National Library of New Zealand
Cataloguing-in-Publication data

RIVER AND ESTUARY MIXING WORKSHOP (1981 : Hamilton, N.Z.)
Proceedings of the River and Estuary Mixing Workshop, Hamilton, 17-18 November, 1981 / edited by J.C. Rutherford. - Wellington, N.Z. : Water and Soil Division, Ministry of Works and Development for the National Water and Soil Conservation Organisation, 1983. - 1 v. -
(Water & soil miscellaneous publication, ISSN 0110-4705 ; no. 49)
628.16109931)
1. Rivers--New Zealand--Congresses. 2. Water--Analysis--Congresses. I. Rutherford, J. C. (James Christopher), 1949- . II. Title. III. Series.



**Published for the National Water and Soil Conservation Organisation by the Water and Soil Division, Ministry of Works and Development, P.O. Box 12041, Wellington, New Zealand.**

## TABLE OF CONTENTS


| | | | Page |
|---|---|---|---|
| 1. | Foreword | | 4 |
| 2. | Description of Mixing in Rivers | J.C. Rutherford | 5 |
| 3. | Design & Construction of the Outfall for the Hamilton City Water Pollution Control Plant | D.R. Wilkie | 13 |
| 4. | Huntly Power Station Outfall Design | E.M. Valentine | 25 |
| 5. | Taranaki Ring Plain Dye Studies : Preliminary Results | J. Kitto<br>J.C. Rutherford | 32 |
| 6. | Influence of Longitudinal Dispersion on Water Quality in Three N.Z. Rivers | J.C. Rutherford<br>B.W. Gilliland<br>G.B. McBride | 44 |
| 7. | Longitudinal Dispersion and the Effects of Dead Zones | E.M. Valentine | 64 |
| 8. | Dispersion of a Continuous Pollutant Source in Non-Uniform Flow | A.J. McNulty | 71 |
| 9. | Modelling Advection and Longitudinal Dispersion | G.B. McBride | 79 |
| 10. | Classification of New Zealand Inlets | R.A. Heath<br>K.R. Grange | 91 |
| 11. | Use of Fluorescent Tracers to Examine Mixing in Lucas Creek, Upper Waitemata Harbour | K.E. Parnell | 93 |
| 12. | Upper Waitemata Modelling Studies | M.J. O'Sullivan<br>R.K. Penny | 121 |
| 13. | Aspects of Dye Circulation in Waimea Inlet : Dye Tracer Studies | K.G. Westcott | 132 |
| 14. | Use of Radioactive Isotopes as Tracers in Rivers | B.J. Barry<br>W.J. McCabe | 146 |
| 15. | Transcript of Discussion Session | | 150 |
| 16. | Appendix A : Background Notes for Discussion Session | | 154 |
| 17. | Appendix B : List of Participants | | 156 |

## FOREWORD

Each year the Water & Soil Science Centre, Hamilton, holds a workshop on a topic which its staff have been actively studying and which is seen to be important to water quality managers. Recent workshops have considered coastal and estuarine research needs and aquatic oxygen problems (see McBride 1982).

The *River and Estuary Mixing Workshop* was designed to be of interest to water resource managers, scientists and engineers. Speakers were invited to present short papers discussing mixing in rivers and tidal estuaries, with particular emphasis on practical procedures, difficulties and achievements. The majority of the presentations described case studies from various parts of New Zealand. The opportunity was provided, both immediately after papers and in a special discussion session, for questions and discussions of the accuracy and usefulness of results obtained, currently available methods, areas of uncertainty, and topics which merit further research work.

The workshop coincided with publication of the "Handbook on Mixing in Rivers" (Rutherford 1981): the first in a series which the Ministry of Works and Development will publish for the National Water and Soil Conservation Organisation. This handbook is aimed at providing practising water resource managers in New Zealand with simple tools and the necessary supporting data to make initial "desk study" analysis of mixing problems in rivers. The opportunity was taken for a workshop session to demonstrate use of the handbook, and explain its aim, strong points and weaknesses.

I wish to acknowledge the major contributions made to organising and running the workshop by Mr Peter Ryan and Dr Robert Bell, Water and Soil Science Centre, Hamilton. Ms Sandie Glazer, Ms Carol Savage and Ms Sue Shaw typed and collated the papers.

## REFERENCES

McBride, G.B. (Ed) 1982 : Aquatic Oxygen Seminar Proceedings, Hamilton, November 1980. Water & Soil Misc.Pub. No. 29, Ministry of Works and Development, Wellington.

Rutherford, J.C. 1981 : Handbook on Mixing in Rivers. Water & Soil Misc. Pub. No. 26, Ministry of Works and Development, Wellington.

J.C. Rutherford
Seminar Organiser

*River and Estuary Mixing Workshop*
*Hamilton Science Centre November 17-18, 1981*

# DESCRIPTION OF MIXING IN RIVERS

*J C Rutherford, Water & Soil Science Centre, MWD, Hamilton*

## ABSTRACT

Mixing is a complex process which is usually best tackled using a combination of experimental and modelling studies. Although the general mixing problem is three-dimensional, in some practical problems vertical, transverse and/or longitudinal dispersion can be neglected. Simple analytical solutions can be obtained by assuming velocity and dispersion coefficients are steady and uniform. These solutions are at best approximate but provide a potentially valuable tool for making a preliminary analysis of a problem and may help decide what additional work is required.

## INTRODUCTION

One of the first steps when assessing the impact of an effluent on water quality is to estimate the concentrations of potentially troublesome constituents. This requires knowledge of the velocities and rates of mixing in the receiving waterway. The mechanics of mixing are complex and by no means completely understood but mixing problems can be tackled by experimental studies, mathematical modelling or a combination of both.

Experimental studies conducted *in situ* using natural or injected tracers can furnish the required data directly. Such data are likely to be accurate in the test reach for the prevailing conditions (e.g. flow, wind etc). One is usually restricted by logistics problems in the choice of test reach and/or in the range of conditions under which one can conduct studies.

Modelling studies use differential equations derived from mass balance principles which incorporate semi-empirical "laws" to describe some aspects of mixing. Such equations need to be "calibrated" against field or laboratory data to ensure appropriate values of the model coefficients are used. Modelling studies offer the considerable advantage of being able to make predictions easily for a range of conditions over a wide area. One potential source of error arises from the need to use model coefficients outside the range of conditions under which they are measured.

Most problems benefit from a preliminary analysis using simple modelling techniques. These may help to decide whether or not the problem merits closer attention, to highlight the data which are required to solve the management problem and to plan the appropriate experimental and/or modelling study.

Examples will be presented at this workshop of both modelling and experimental studies. It should be remembered that both have their place in solving management problems.

## DESCRIPTION OF MIXING

When material (hereafter referred to as tracer for convenience) is discharged into a waterway two things happen to it. Firstly, it is carried away from the outfall by the current, a process which is termed *advection;* and secondly, it spreads out, a process which is termed *dispersion.*

In stagnant water and uniform flow, spreading is attributable solely to molecular motion and is called *molecular diffusion.* Tracer moves from a region of high concentration to a region of lower concentration at a rate proportional to the concentration gradient between the two regions. This is Fick's Law which

in one dimension can be written mathematically:

$$R_x = -D\frac{dC}{dx} \qquad (1)$$

where $R_x$ = transfer rate per unit area in the x direction, C = concentration, dC/dx = gradient in the x direction, and D = molecular diffusion coefficient, a constant.

In turbulent and non-uniform flow, spreading proceeds at a much higher rate than in laminar flow. The reason for this is that velocity gradients are present which increase concentration gradients and hence allow molecular diffusion to occur more rapidly. This is illustrated in Fig. 1. Such spreading is termed *dispersion* to distinguish it from *molecular diffusion*.

It has been found that in many situations the rate of dispersion can be approximated by Fick's Law. However, the value of D may be several orders of magnitude larger than for molecular diffusion and is highly variable. The variability arises partly because the size and intensity of turbulent eddies may vary considerably with position in the river channel, with changes in flow or location, and from one channel to another. For example, the rate of dispersion can be expected to be smaller very close to the river bed, where velocity and intensity of turbulence are low, than at mid depth. Also as the size of the tracer patch being investigated increases, the velocity gradients may change and larger eddies may become involved in mixing. Thus very close to an outfall the rate of dispersion can be expected to be smaller than it is further downstream.

In the most general problem, advection and dispersion will occur in each of the three coordinate directions, and the governing equations will be comparatively complex. In many practical problems, however, the analysis can be simplified by neglecting terms which are small. This can be done if any of the velocities is small or if any of the concentration gradients (and hence dispersion rates) is small.

Concentration gradients are likely to be small in the vertical and transverse directions when tracer has impinged on the bed and banks and has become well mixed. Longitudinal gradients are likely to be small below steady discharges. This means that vertical, transverse and longitudinal mixing can sometimes be considered as separate one-dimensional problems.

## MIXING PROBLEMS IN RIVERS

In rivers various simplifications to the general three-dimensional mixing problem can be made which give rise to a number of separate sub-problems.

Vertical and lateral velocities in rivers are small and can often be neglected although their effects may sometimes need to be accounted for in those dispersion terms which are retained. For example, longitudinal dispersion arises principally because of lateral velocity gradients (which cause tracer to travel downstream faster in the centre of the channel than at the edges). These velocity gradients do not appear explicitly in the one-dimensional longitudinal dispersion model but determine the value of the dispersion coefficient.

If the injection rate does not vary with time (termed "steady") then longitudinal concentration gradients are usually negligible and hence longitudinal dispersion can be ignored. If the injection is made uniformly across the channel width (e.g., from a diffuser pipe) then transverse concentration gradients are small and transverse dispersion can be ignored. Similarly, if the injection is made uniformly over the depth then there is no need to consider vertical dispersion.

Near a point source both vertical and transverse dispersion will occur. Some distance downstream, however, the tracer will become vertically well mixed and thereafter only transverse dispersion need be considered.

Fig. 2 illustrates the types of mixing problem encountered below steady tracer injections. For non-steady injections, longitudinal dispersion must be considered as well.

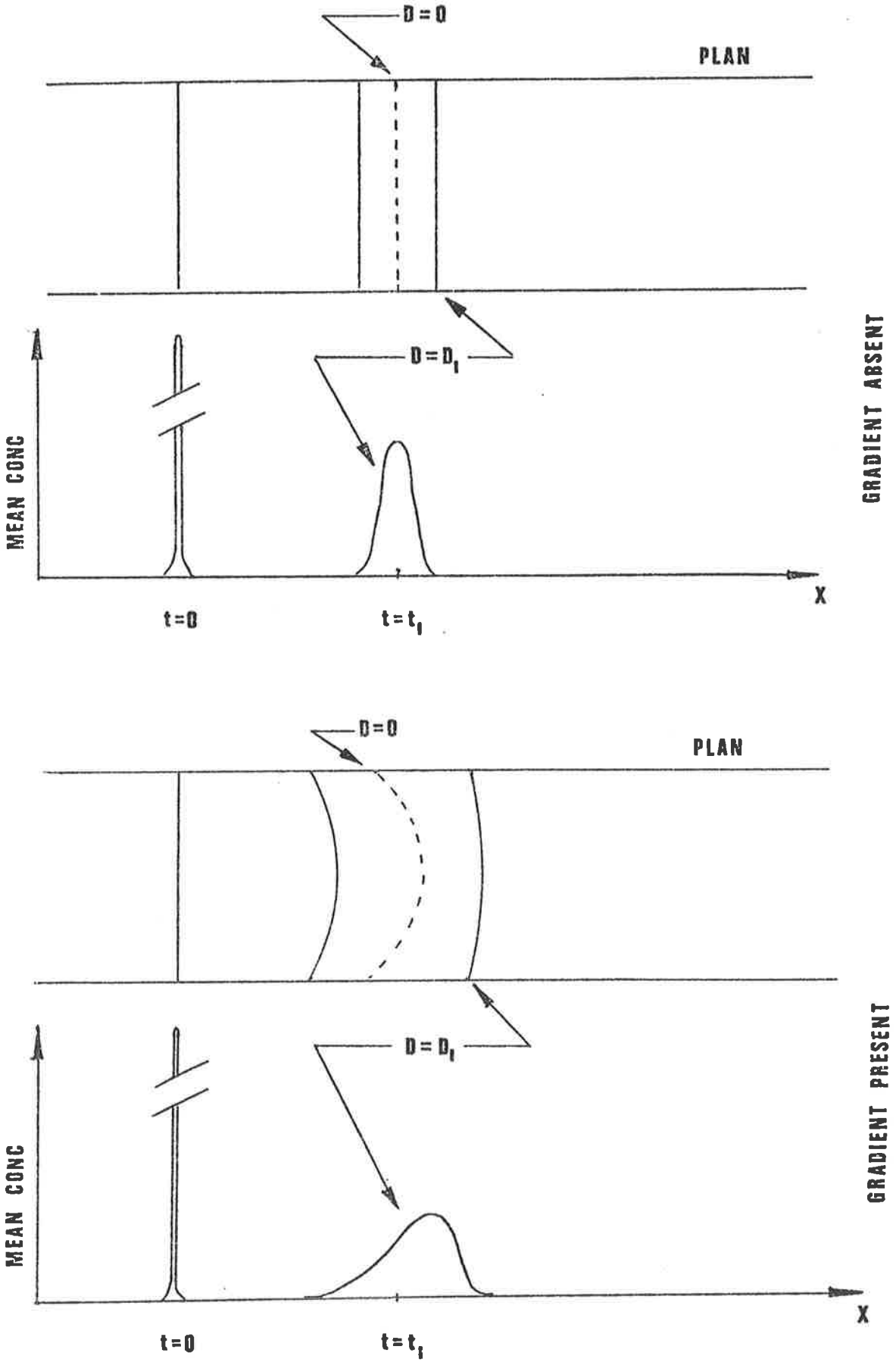



**Figure 1** Sketch showing how a velocity gradient increases the dispersion rate.

## MIXING MODELS FOR RIVERS

Simple models for river dispersion have been developed which make the following assumptions:

(a) longitudinal velocity does not vary vertically, transversely, longitudinally or with time (termed "steady" and "uniform");
(b) dispersion occurs in one, two or three dimensional directions;
(c) the rate of dispersion obeys Fick's Law, see equation (1);
(d) the dispersion coefficients do not vary vertically, transversely, longitudinally or with time (termed "steady" and "uniform");
(e) boundaries act as perfect reflectors.

These assumptions lead to rather simple equations for which analytical solutions may be obtained to determine concentration as a function of time and/or location for several problems of practical importance. Table 1 summarises the important river mixing problems and the terms needed to solve them. The analytical solutions can be evaluated readily using a programmable calculator or mini-computer and in many cases are amenable to presentation in the form of nomographs. These methods form the basis of the "Handbook of Mixing in Rivers" (Rutherford 1981).

Problems for which solutions may be presented in the form of nomographs are:

1 vertical mixing below a steady transverse line source;
2 vertical and transverse mixing below a steady point source;
3 transverse mixing below a steady vertical line source (analagous to problem 1);
4 longitudinal dispersion below an instantaneous point source.

It is also possible to superpose solutions to these simple problems and to obtain solutions for more complex problems such as steady sources of finite width, and unsteady point sources.

Several caveats need to be placed on the use of these simple models.

1 In most natural channels velocity, vertical and transverse dispersion coefficients vary vertically and transversely. Thus assumptions (a) and (d) are violated which suggests that the simple methods for studying vertical and transverse mixing are at best only approximate. McNulty (*ibid*) will present results of a theoretical study which quantifies this.

2 Near the bank and bed of rivers tracer may become trapped in "dead-zones" and assumption (e) violated. Valentine (*ibid*) will describe how this affects longitudinal dispersion.

3 When considering longitudinal dispersion, velocity gradients determine the value of the dispersion coefficient. Their effect can be approximated by a Fickian model only some way downstream from the point of injection.

Clearly, therefore, the simple dispersion models are best only approximate. They have considerable value, nevertheless, for they provide a simple tool which can easily be used to make a preliminary analysis of a mixing problem. This analysis may help to decide whether or not the problem merits closer attention, to highlight data requirements, and to plan the appropriate experimental work.

## DATA REQUIREMENTS

For any preliminary analysis of mixing which uses the simple models described above estimates are required of channel parameters (e.g., mean depth, width etc.), velocity and dispersion coefficients.

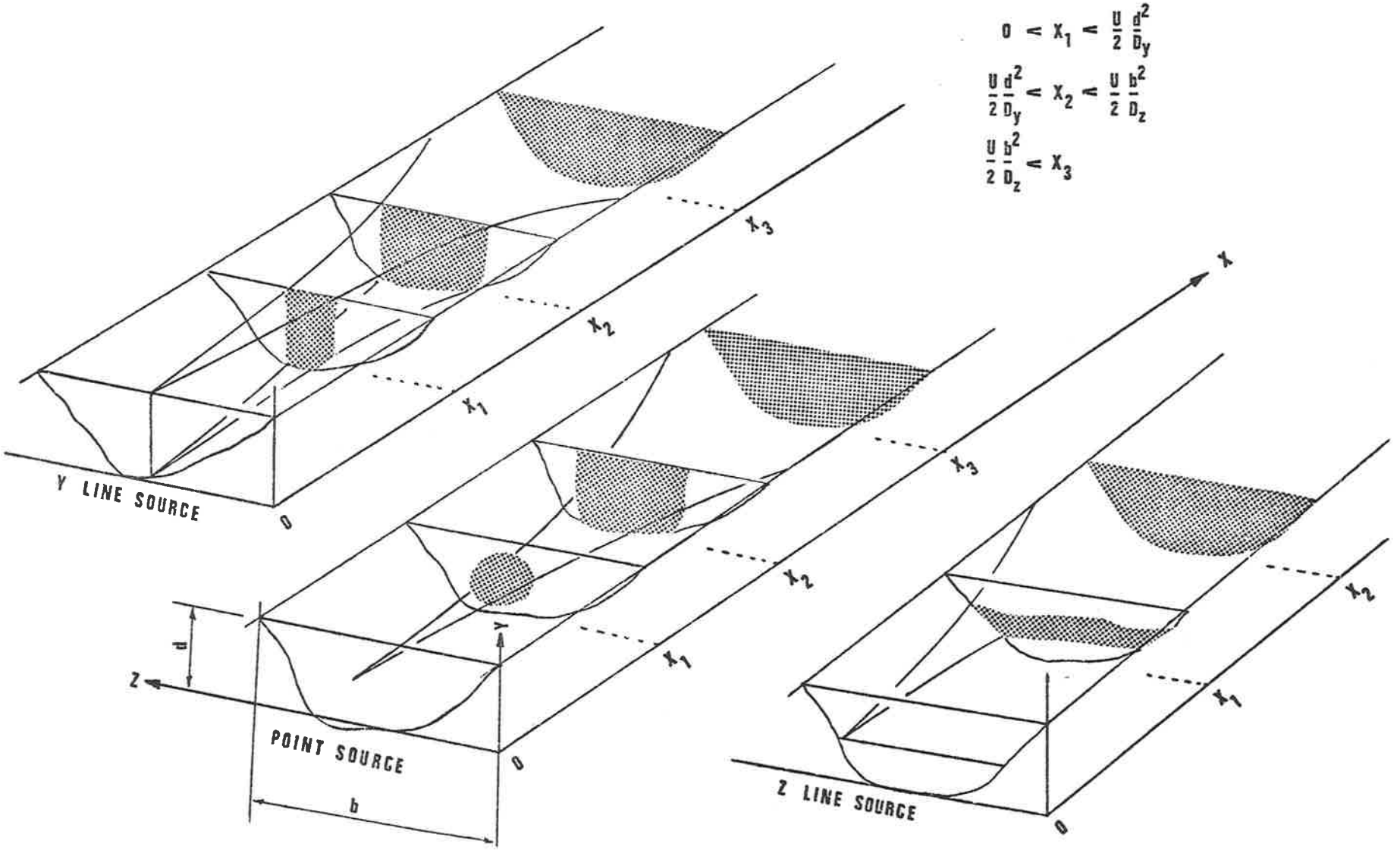



Figure 2 : Sketch of three types of river dispersion problem.

**Table 1** Summary of important dispersion problems in rivers and terms required to study them

| Type of Source | | | Type of Solution | Terms required (1) | | Number of dimensions | |
|---|---|---|---|---|---|---|---|
| time | space (1) | (2) | (3) | advection | dispersion | | |
| instantaneous | point source | near field | $0 < x < \frac{U}{2} d^2/D_y$ | $x$ (4) | $x$, $y$, $z$ | 3 | |
| | | mid field | $\frac{U}{2} d^2/D_y < x < \frac{U}{2} b^2/D_z$ | $x$ | $x$, $z$ | 2 | |
| | | far field | $\frac{U}{2} b^2/D_z < x < \infty$ | $x$ | $x$ | 1 | |
| steady | point source | near field | $0 < x < \frac{U}{2} d^2/D_y$ | $x$ (4) | $y$, $z$ | 3 | (6) |
| | | mid field | $\frac{U}{2} d^2/D_y < x < \frac{U}{2} b^2/D_z$ | $x$ | $z$ | 2 | (6) |
| | | far field | $\frac{U}{2} b^2/D_z < x < \infty$ | – (5) | – (5) | 0 | (5) |
| instantaneous | $z$-line source | near field | $0 < x < \frac{U}{2} d^2/D_y$ | $x$ | $x$, $y$ | 2 | |
| | | far field | $\frac{U}{2} d^2/D_y < x < \infty$ | $x$ | $x$ | 1 | |
| steady | $z$-line source | near field | $0 < x < \frac{U}{2} d^2/D_y$ | $x$ | $y$ | 2 | (6) |
| | | far field | $\frac{U}{2} d^2/D_y < x < \infty$ | – (5) | – (5) | 0 | (5) |
| instantaneous | $y$-line source | near field | $0 < x < \frac{U}{2} b^2/D_z$ | $x$ | $x$, $z$ | 2 | |
| | | far field | $\frac{U}{2} b^2/D_z < x < \infty$ | $x$ | $x$ | 1 | |
| steady | $y$-line source | near field | $0 < x < \frac{U}{2} b^2/D_z$ | $x$ | $z$ | 2 | (6) |
| | | far field | $\frac{U}{2} b^2/D_z < x < \infty$ | – (5) | – (5) | 0 | (5) |

NOTES:

(1) co-ordinate directions are shown in Fig. 1.3

(2) near field = very close to the outfall, mid field = moderately and far field = some considerable distance away

(3) $D_x$, $D_y$, $D_z$ are dispersion coefficients and $U$ = mean velocity (see equations 2.12 and 3.9 in text)

(4) on a very small scale, $y$, $z$ advection may be present in the prototype

(5) concentration is constant (fully mixed)

(6) the dimensionality can be reduced by one if the co-ordinate system used travels downstream at mean velocity

Channel parameters will usually be known from surveys and velocities can be estimated either from gaugings, or from channel parameters using semi-empirical flow formulae (Henderson 1966). Dispersion coefficients are highly variable but a first estimate can be made from published data (for example Rutherford 1981). In most cases these estimates will have a large inherent uncertainty which may outweigh the shortcomings of the simple dispersion models.

## FIELD STUDIES

Preliminary analysis may indicate that further analysis is justified. Often this will take the form of experimental work, possibly followed by modelling work to extrapolate from experimental results to derive design data. Various experimental methods can be adopted but it is beyond the scope of this paper to discuss them.

Once reliable field data have been obtained, the simple models described above can sometimes be used to extrapolate experimental data, but in other cases more sophisticated modelling techniques may need to be applied. This will be a matter of judgment and is best left to be considered for each problem separately.

It is at this second phase of analysing a mixing problem that consultation with a specialist in experimental or modelling techniques may be valuable. Such specialists may be found in DSIR, MWD, universities, engineering consultants and in some regional water boards. One of the aims of this workshop is to make participants aware of where various expertise resides.

## CONCLUSIONS

1 Mixing arises from both advection and dispersion.
2 Advection is the result of averaged velocities.
3 Diffusion is the process whereby tracer moves from a region of high concentration to a region of low concentration as a result of molecular movements.
4 Dispersion results from the combination of molecular diffusion and the effects of velocity gradients and turbulence. Rates of dispersion are usually much higher than rates of diffusion.
5 In many circumstances dispersion can be modelled by Fick's Law although values of dispersion coefficients may vary with time and location in a particular waterway, and from one waterway to another.
6 Although the general dispersion problem is three-dimensional simplifications can sometimes be made to reduce the complexity of the problem.
7 Simple methods, which assume that velocity and dispersion coefficients are uniform and constant, yield analytical solutions to a range of problems of practical interest.
8 Such solutions are at best approximate but are potentially useful for making preliminary analysis of a problem which may help to decide what further action is required.

## REFERENCES

Henderson, F.M. 1966 : Open Channel Flow. McMillan, New York.
Rutherford, J.C. 1981 : Handbook on Mixing in Rivers. Water & Soil Misc. Pub. No. 26, MWD, Wellington.

## ACKNOWLEDGEMENTS

Dr A G Barnett for discussions during the preparation of this paper.

DISCUSSION

M E U Taylor Is it possible to estimate the uncertainty of these predictions?
Rutherford It is difficult to undertake a formal analysis of uncertainty, but one should always undertake a sensitivity analysis in which one makes predictions with a range of model coefficients (e.g., dispersion coefficients) and thereby gets a feel for the possible range of tracer concentrations.

P N McFarlane Can you comment on what flow conditions are limiting and hence for what flow conditions this analysis should be carried out?
Rutherford Normally one conducts tracer tests at low flow to reduce the quantities of tracer needed. The relationships between dispersion coefficients and flow conditions are not well understood. In many cases it is simply a matter of trying various combinations of coefficients to determine what are critical conditions.

J D Boyle Is the dispersion coefficient material dependent?
Rutherford It is generally assumed that tracers give a good estimate of the dispersion coefficients for pollutants. Fairly large uncertainties are often introduced when results from field tests are used to make predictions and these probably swamp other sources of uncertainty.
A G Barnett In rivers one is usually concerned with low concentrations and turbulent mixing dominates mixing, so dye probably gives a good estimate of pollutant dispersion.
J D Boyle and A J Sutherland This may not be true for particulates or pollutants with densities significantly different from water.

P Tortell What effect does a temperature difference between tracer and receiving water have?
Rutherford Stratification reduces the rates of vertical mixing and influences the rates of transverse mixing. Modelling buoyant effluents is difficult but in doing desk studies one can sometimes get an idea of likely concentrations by making (often crude) assumptions about the likely trajectory and mixing zone of the effluent.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre November 17-18, 1981*

# DESIGN AND CONSTRUCTION OF THE OUTFALL FOR THE HAMILTON CITY WATER POLLUTION CONTROL PLANT

*D R Wilkie, Steven, Fitzmaurice and Partners, Christchurch*

## ABSTRACT

Primary sewage effluent from the Hamilton City Water Pollution Control Plant is discharged into the Waikato River through a sub-fluvial outfall which extends full-width across the river.

In this paper, various aspects of the outfall are outlined and discussed including: design considerations; the diffuser model study; the novel contractural arrangement agreed upon for construction of the outfall; and the actual construction procedure. It concludes that the effluent discharged from the diffusers is quickly and effectively entrained into the full body of the river flow.

## INTRODUCTION

In 1970 the Hamilton City Council undertook a major construction programme to provide the City with a modern sewerage system and sewage treatment plant. The sewage treatment plant is a primary plant which discharges treated effluent into the Waikato River by means of a sub-fluvial outfall that extends across the full width of the river. The need for this form of outfall arose from an undertaking given to the Hamilton City Council that effluent from the sewage treatment plant would be mixed into the full body of the flow of the Waikato River. This undertaking was one of those that is relatively easy to say or write, but quite difficult to put into practice. As a result, design of this sub-fluvial outfall was delayed by procrastination by the writer until the treatment plant was well underway; eventually it was apparent that since the problem would not go away on its own, the matter had to be tackled and the appropriate factors and related problems solved.

## DESIGN CONSIDERATIONS

Before any significant design work could be carried out it was necessary to establish factors such as: the characteristics of river flow; the anticipated variations in level that occurred due to flooding; the characteristic of bed movement within the river; any other idiosyncrasies or peculiarities that might have a bearing on the outfall design.

In addition, it was necessary to review the sewage treatment plant flow characteristics, both for the initial design and ultimate design conditions. A summary of these conditions is set out in Table 1.

Lastly, the general feasibility of constructing a pipeline across the river was also seen as a major factor.

Initial inquiries of the Waikato Valley Authority, the Ministry of Works and other parties who had been building structures such as bridges across the river revealed that, while the river level did not fluctuate to a large extent, there was a complicating factor involving the bed movement characteristics. Evidently it was well established that waves of bed material tended to move down the river and these waves were known to have a trough to crest height of approximately 2.4 m. Nobody was able to give very much in the way of assistance as to depths of scour or depths to which bed movement occurred, so, it was decided to adopt the concept of a pipeline buried to a depth of 30 m below the riverbed at the deepest point with pedestals of varying heights rising vertically from this pipeline across the full width of the river to ensure an even distribution of effluent into the river flow. Before any details of such a scheme were resolved, it became apparent that our expertise in nozzle design and the identification of nozzle shape, and even direction in which the nozzles should face, was lacking and that the questions posed would best be answered by a model study.

TABLE 1 Characteristics of the sewage treatment plant outfall

| | STAGE 1 | ULTIMATE DEVELOPMENT |
|---|---|---|
| Minimum Flow (l/s) | 210 | 380 |
| Average Flow (l/s) | 527 | 950 |
| Peak Flow (l/s) | 2635 | 4790 |
| Number of Diffusers | 10 | 20 |
| Minimum Flow : Discharge/Diffuser (l/s) | 21 | 19 |
| : Velocity in 229 m Ø throat (m/s) | 0.5 | 0.45 |
| Average Flow : Discharge/Diffuser (l/s) | 52.7 | 47.50 |
| : Velocity in 229 m Ø throat (m/s) | 1.26 | 1.13 |
| Peak Flow : Discharge/Diffuser (l/s) | 263.50 | 238.00 |
| : Velocity in 229 m Ø throat (m/s) | 6.27 | 5.67 |

MODEL STUDY WORK

As a result, after discussions with Dr Alex Sutherland of Canterbury University, a model study was commissioned that was required to answer the following questions.

1 What was the best shape of the diffuser to ensure that:
   a it remained unobstructed at all times; and
   b achieved the maximum amount of dispersion of the effluent into the body of the river?

2 What direction should the diffuser nozzles face to achieve optimum dispersion?

3 What is the behaviour of such a diffuser if it should be submerged by a sand dune passing down the river?

4 What was the scour pattern and the consequences of this pattern if the bed of the river scoured down to the top or below the diffuser pedestal?

In order to answer these questions we first obtained a physical profile across the river at the outfall site. This gave the bed profile at the time the soundings were taken, and indicated velocity contours within the body of the river flow. This profile is shown on Fig. 1.

A model was constructed (scale 1:12) and a series of flow tests carried out in the Fluids Laboratory at the School of Engineering, Canterbury University. A number of diffuser profiles and directions of flow were tried. The general arrangement of the model diffuser nozzle is shown on Fig. 2 and the flow patterns established around the diffuser under varying bed conditions are shown in Figs. 3 (a), (b), (c), (d), and (e). The nett result of this model study was the following list of findings and recommendations.

1 A dune-covered bed can be expected at all flow conditions with a maximum dune height of up to 2.4 m.

2 The best dispersion pattern would be achieved by facing the diffuser nozzle upstream, but the advantage of discharging upstream was reduced at low effluent flows and by the presence of bed forms. Indeed, it was the bed form and scour hole geometry that largely controlled the dispersion pattern when the adjacent bed level was comparable with the nozzle level.

3 The passage of sand dunes over the diffuser resulted in some obstruction of the effluent flow. This obstruction was less if the nozzle discharged downstream and was cleared rapidly for the recommended diffuser shape which is shown in Fig. 4.

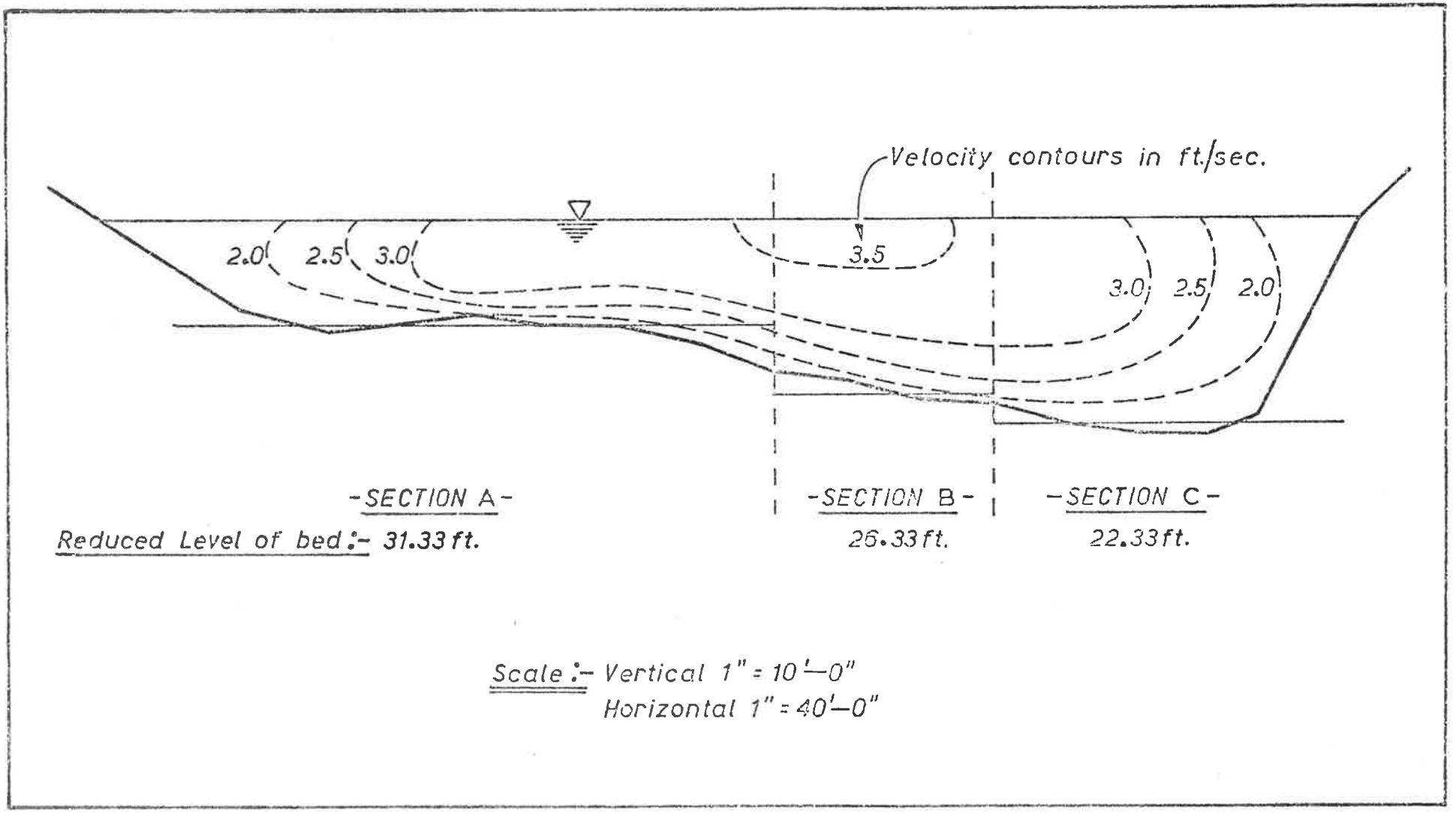



Figure 1 : RIVER CROSS-SECTION AT OUTFALL SITE

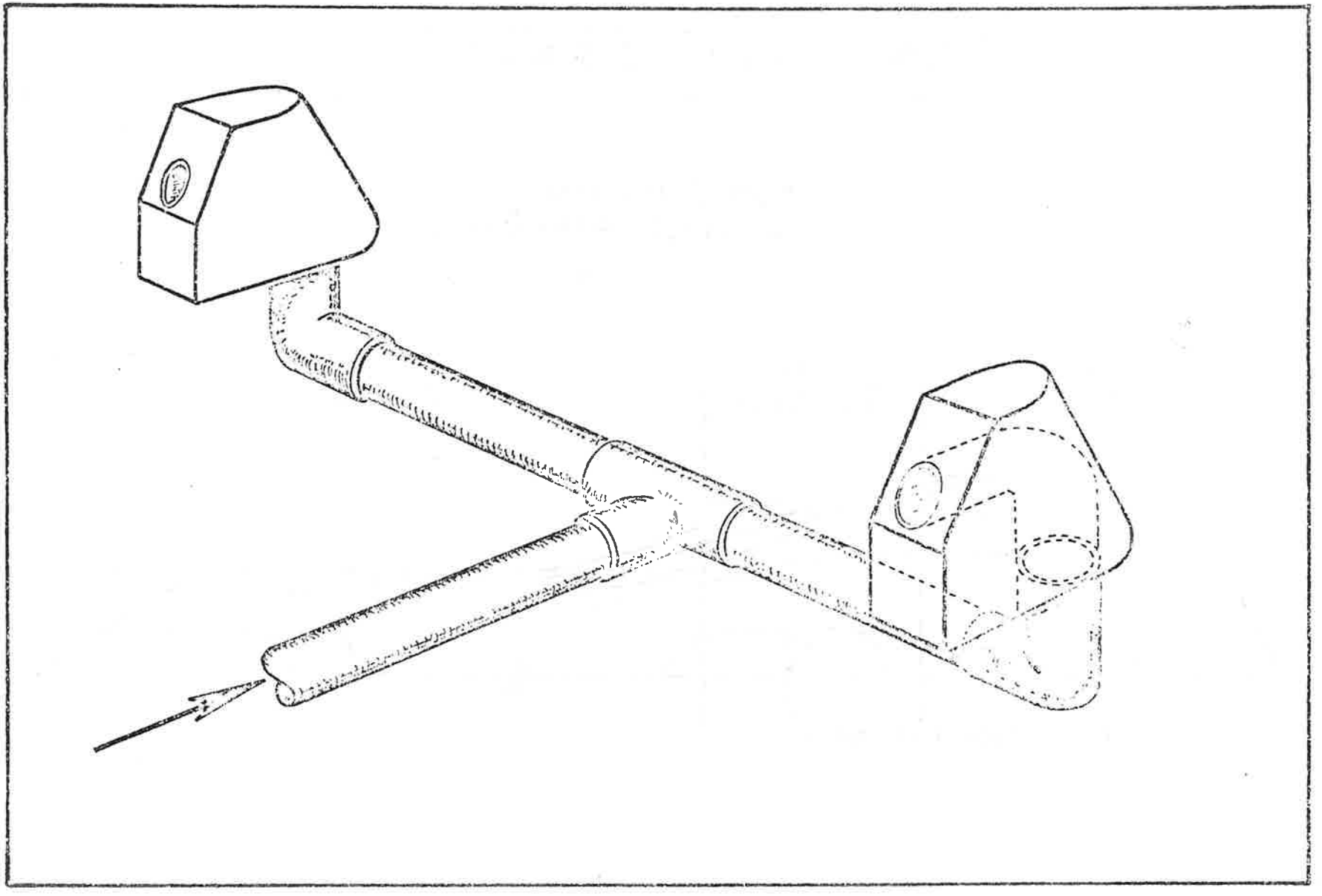
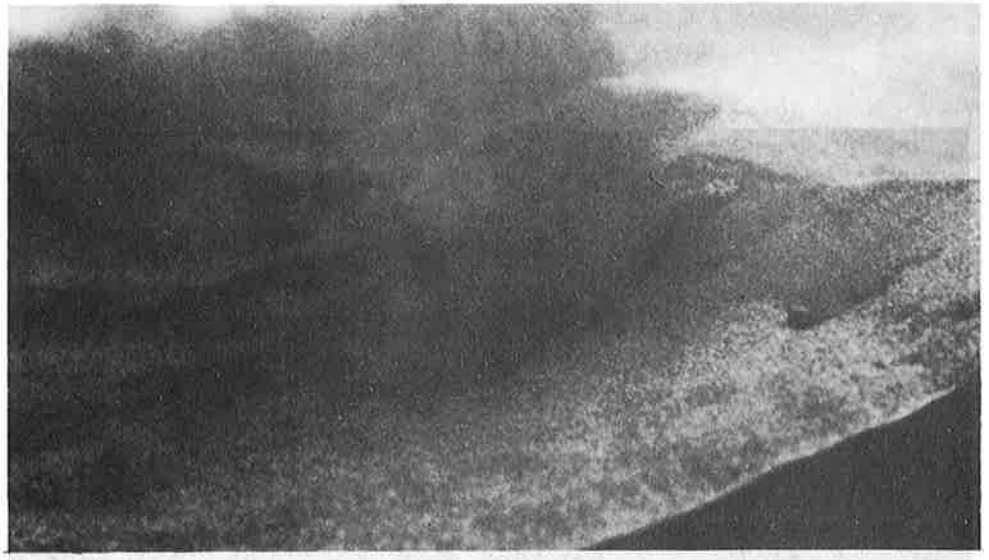


Figure 2 : NOZZLE ARRANGEMENT

FIGURE 3 : DUNE PASSAGE OVER RECOMMENDED DIFFUSER

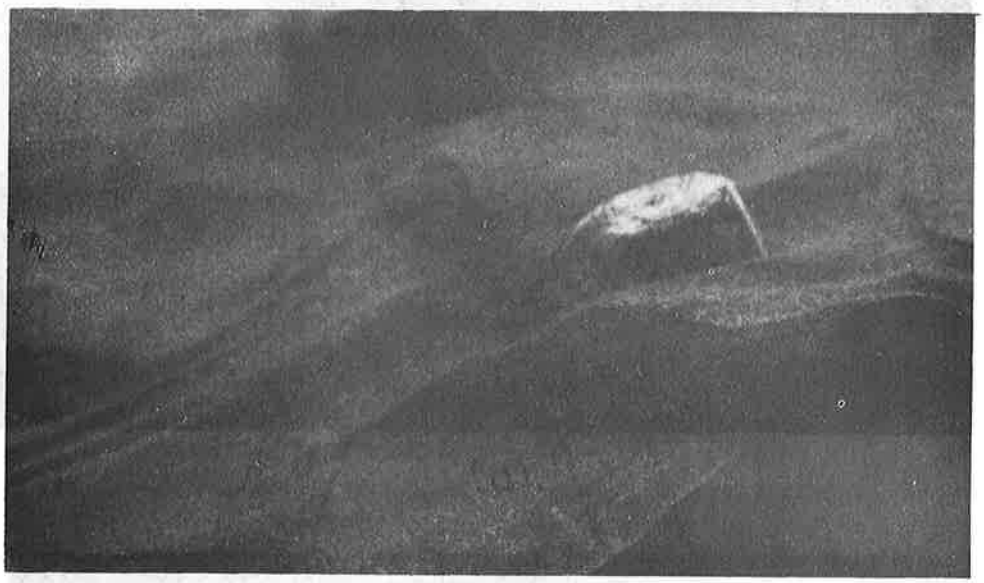
(a)



(b)

(c)

(d)



(e)

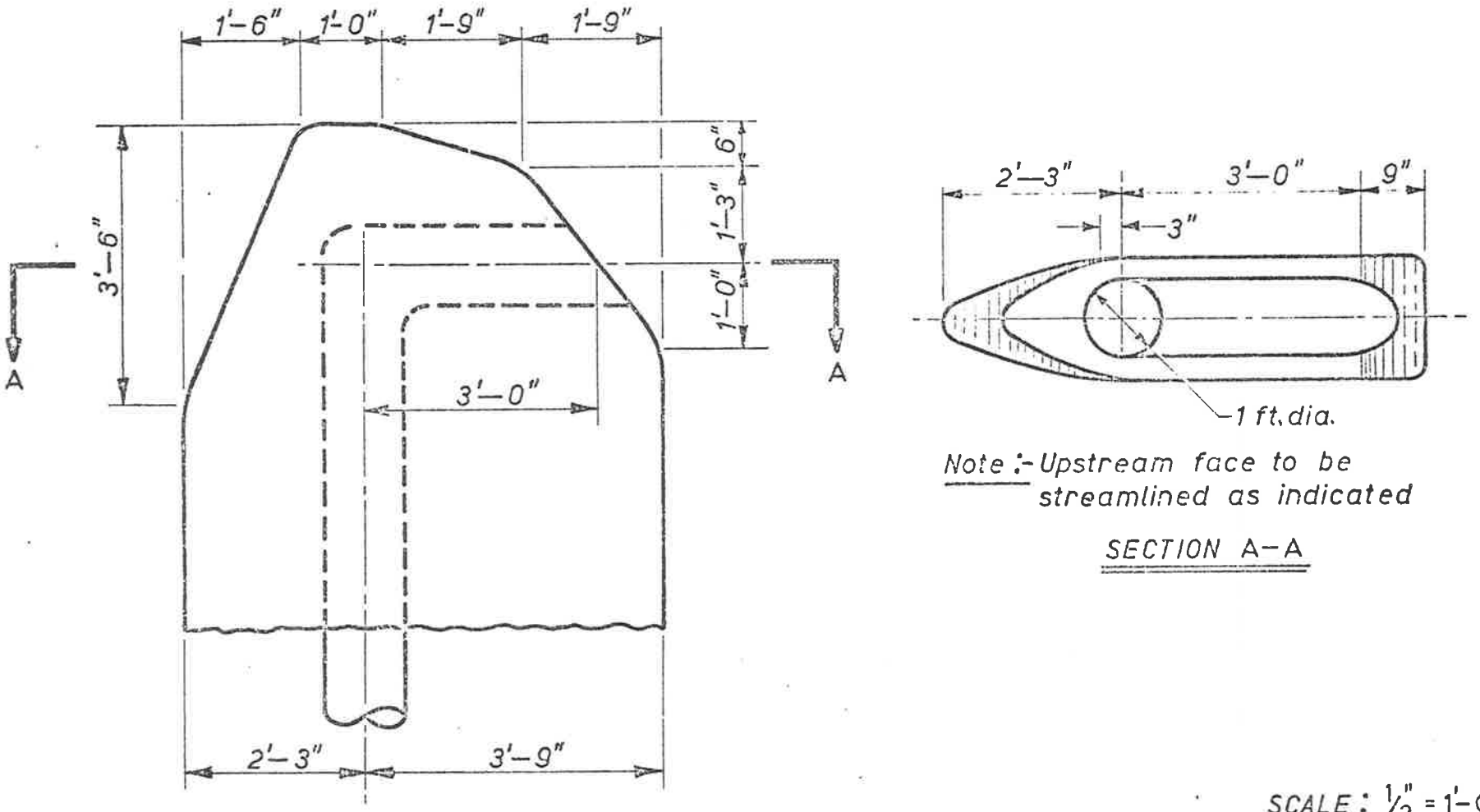



Figure 4 : RECOMMENDED DIFFUSER SHAPE

4 If the main pipe was placed at a reasonable depth below the riverbed it would be necessary to protect the pipe with a riprap blanket.

Following the completion of this study the main design of the outfall was undertaken and the general form of the project is shown on Figs. 5, 6 and 7. It was almost entirely based upon the model study recommendations. The only recommendation not acted upon was that of the riprap protection for the main pipe. Instead it was decided to place and drive a concrete pile on the downstream side of each diffuser (not upstream as shown on the drawings) in the hope that this would give to the outfall a measure of support and resistance against bed movement should it occur.

From the flow characteristics of the diffusers - as set out in Table 1 - it can be seen that the outflow has been designed to cater for an initial flow of some 210 l/s and an ultimate flow of 4760 l/s. It was our intention to maintain a minimum velocity of 0.3 m/s through the diffuser nozzle to prevent the entry of river bed material and it was recognised at the outset that approximately half of the diffusers would have to be plugged initially in order to achieve this. Having put down our thoughts as to how the pipe should be assembled, and the form it should take, we then gave some consideration to the manner in which this pipeline might be constructed.

## CONTRACTURAL ARRANGEMENTS

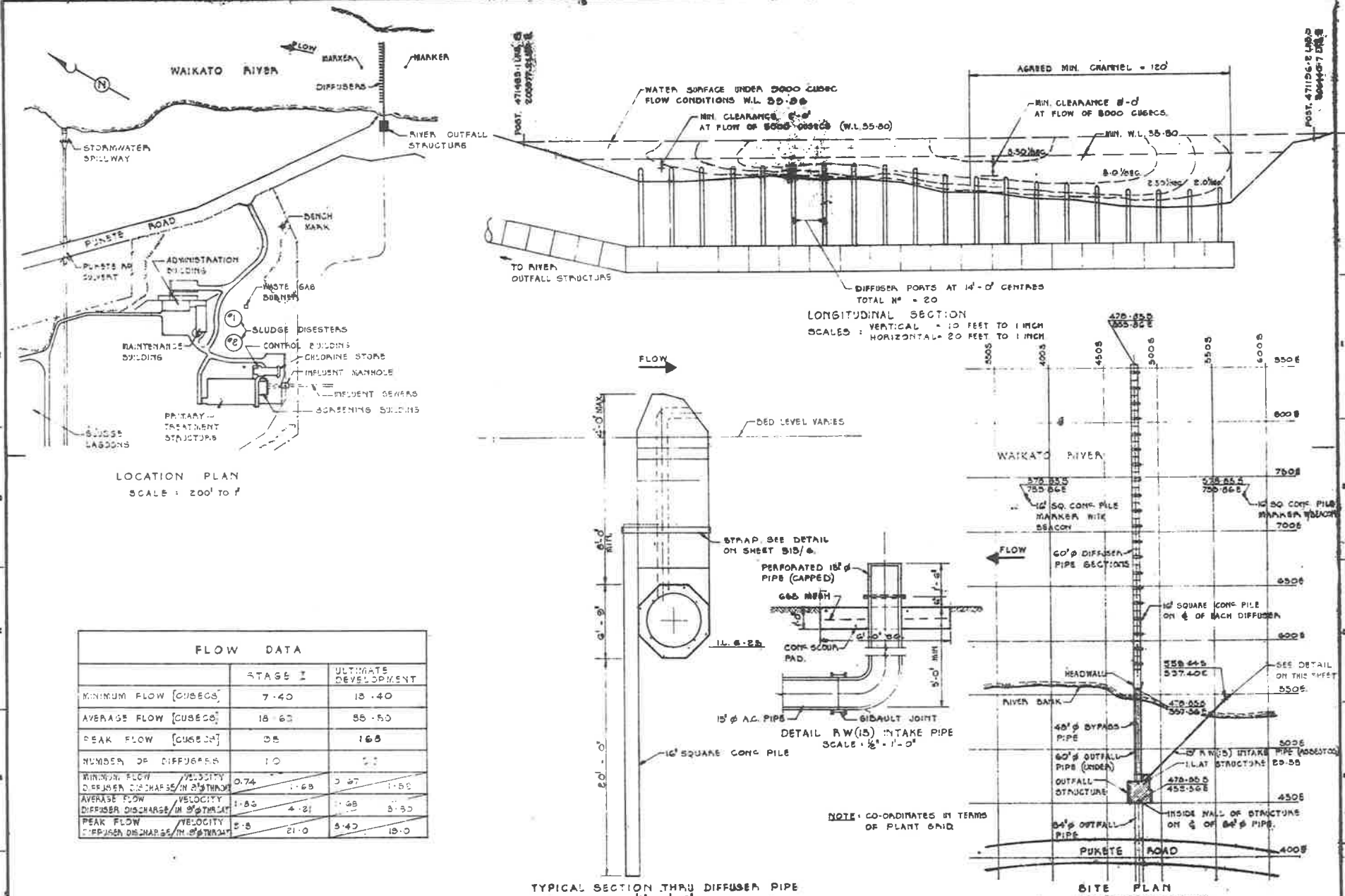
Because there was already a reasonably large contract running on the construction of the treatment plant, it was decided, after consultation with the City Council, that it would be appropriate to extend this contract on the basis of a Target Estimate contract to cover the construction of the outfall. By way of explanation for those who are not aware of what is meant by this term, a Target Estimate contract is a process of establishing with the contractor a genuine pre-estimate of the actual cost of the works which does not include profit, overheads, nor ancillary items. The profit and overhead is then added in, again as an estimate, and this final figure is agreed to by all parties; the principal, the consulting engineers and the contractor. This is then put away in a drawer and work is commenced and recorded on a pure cost basis. If, at the end of the job, the work on a pure cost basis comes out at less than the target estimate, then the savings are shared on an equal or prearranged basis with the contractor and the principal, i.e., if the works cost $100,000 less than the target estimate then the saving made is shared $50,000 to the contractor and $50,000 to the principal, or on other such prearranged terms. Similarly, if the costs go over the target estimate, then the additional costs are shared on an equal or a prearranged basis as with the savings.

## CONSTRUCTION

Work commenced on the outfall in mid-1974 and had a contract period of some 10 months. Briefly, the construction consisted of driving a sheet pile coffer-dam or cell out across the river, and carrying this cell across the river on a staging that was extended as each pipe was laid. As each stage proceeded a cell was driven, excavated of metal, a pipe placed in it adjoining the last pipe, the diffuser sections added on top of the pipe, the pipe then backfilled, the sheetpiling withdrawn and redriven to form another cell and so on across the river. The whole of the work was done "in the wet" by divers operating airlifts and other such necessary equipment.

The project ultimately finished up with a staging across the full width of the Waikato River. This of course was the most dangerous period in the contract as a flood could cause a significant amount of damage to this staging. As was to be expected, the river varied in depth, being shallow on the western side and deep on the eastern side so that the difficulty of carrying out the work increased as the work proceeded across the river. Upon placing the last pipe, the cell sheet piling was withdrawn and the staging demolished step-by-step working back across the river.

As each diffuser was completed the port was temporarily plugged, to prevent the ingress of sand while the diffuser was not in operation. Upon commissioning of the treatment plant in July 1975 a number of ports



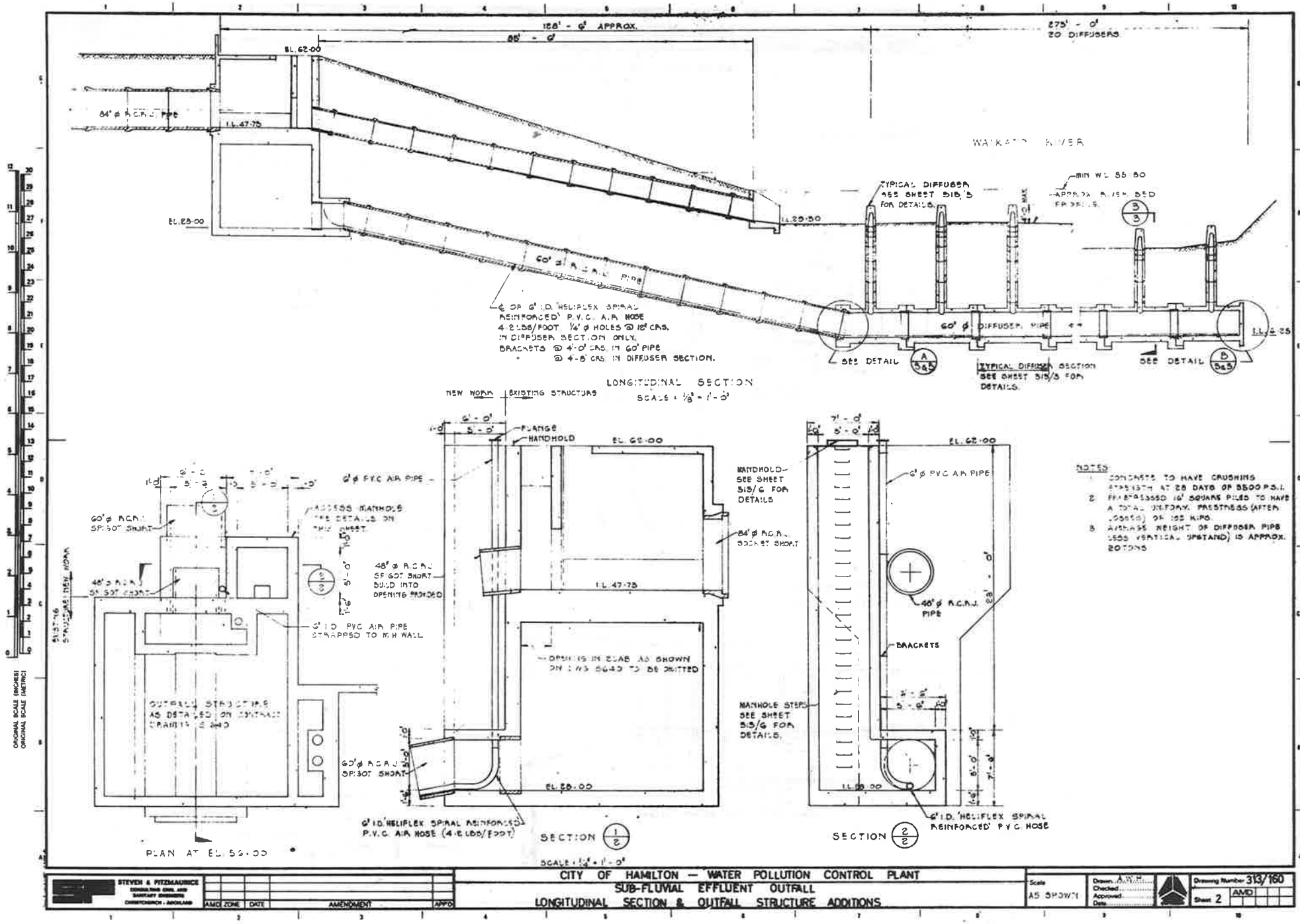

| FLOW DATA | | |
|---|---|---|
| | STAGE I | ULTIMATE DEVELOPMENT |
| MINIMUM FLOW [CUSECS] | 7·40 | 18·40 |
| AVERAGE FLOW [CUSECS] | 18·60 | 55·50 |
| PEAK FLOW [CUSECS] | 35 | 168 |
| NUMBER OF DIFFUSERS | 10 | 20 |
| MINIMUM FLOW — VELOCITY DIFFUSER DISCHARGE/IN 84'ø THROAT | 0.74 / 1·68 | 0·87 / 1·50 |
| AVERAGE FLOW — VELOCITY DIFFUSER DISCHARGE/IN 84'ø THROAT | 1·86 / 4·21 | 1·68 / 3·80 |
| PEAK FLOW — VELOCITY DIFFUSER DISCHARGE/IN 84'ø THROAT | 3·5 / 21·0 | 5·40 / 19·0 |

ORIGINAL SCALE (INCHES)
ORIGINAL SCALE (METRIC)

STEVEN & FITZGERALD CONSULTING CIVIL AND SANITARY ENGINEERS WELLINGTON - AUCKLAND

CITY OF HAMILTON — WATER POLLUTION CONTROL PLANT
SUB-FLUVIAL EFFLUENT OUTFALL
SITE PLAN LOCALITY PLAN & SECTIONS

Scale AS SHOWN
Drawn A.W.H.
Checked
Approved
Date
Drawing Number 383/150
Sheet 1 AMD




STEVEN & FITZMAURICE CONSULTING CIVIL AND SANITARY ENGINEERS CHRISTCHURCH - AUCKLAND

| A.M.D. ZONE | DATE | AMENDMENT | APPD |
|---|---|---|---|

CITY OF HAMILTON — WATER POLLUTION CONTROL PLANT

SUB-FLUVIAL EFFLUENT OUTFALL

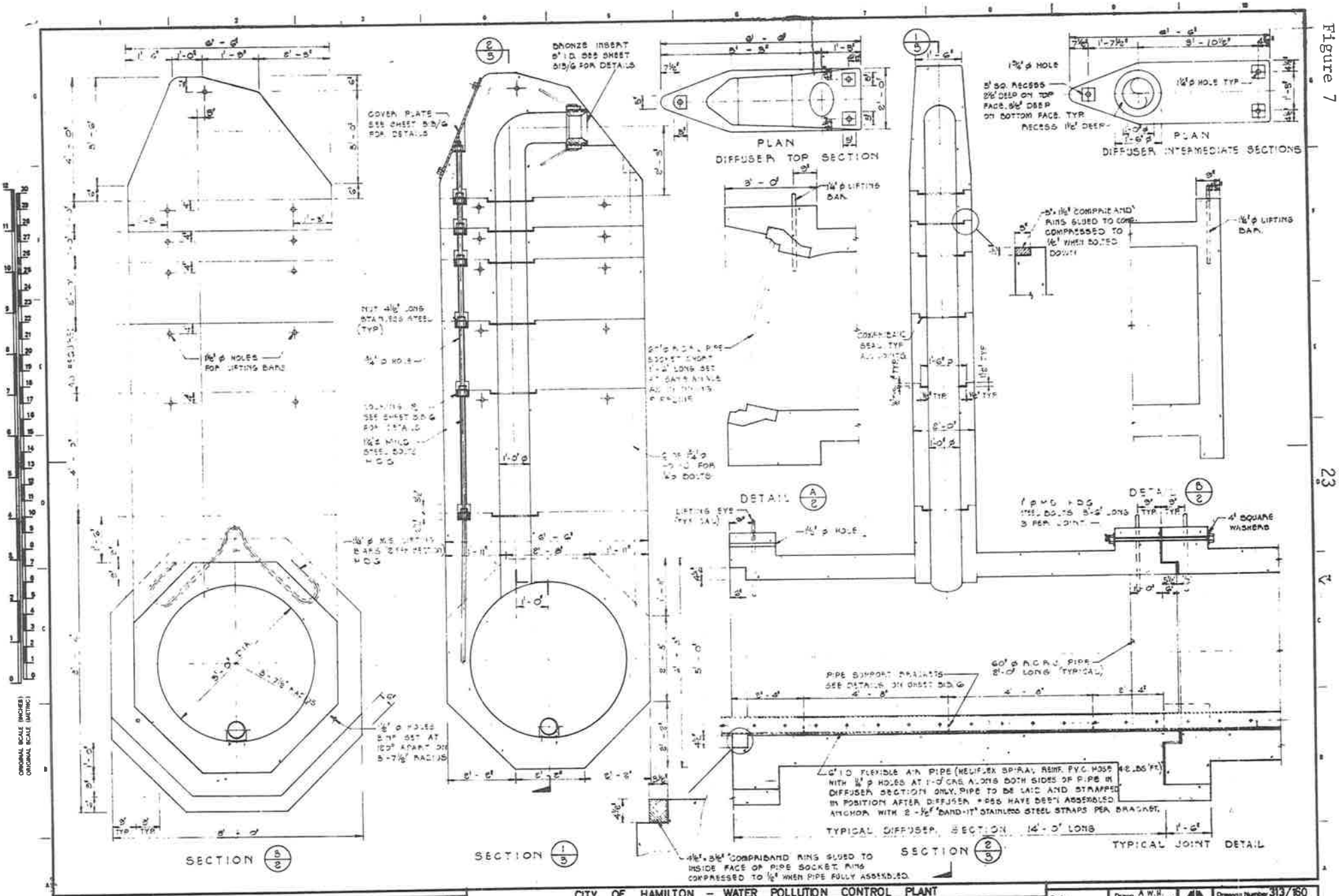
LONGITUDINAL SECTION & OUTFALL STRUCTURE ADDITIONS

Scale: AS SHOWN

Drawn: A.W.M.
Checked:
Approved:
Date:

Drawing Number 313/160 AMD
Sheet 2

Figure 6




STEVEN & FITZMAURICE CONSULTING CIVIL AND SANITARY ENGINEERS CHRISTCHURCH - AUCKLAND

| AMD | ZONE | DATE | AMENDMENT | APP'D |
|---|---|---|---|---|

CITY OF HAMILTON – WATER POLLUTION CONTROL PLANT
SUB-FLUVIAL EFFLUENT OUTFALL
DIFFUSER PIPE DETAILS

Scale 3/4" = 1'-0"
Drawn A.W.H.
Checked
Approved
Date

Drawing Number 313/160
Sheet 3 AMD

Figure 7

23

were opened and because of the low flows it was necessary to place restrictors in the diffuser ports in order to keep the exit velocity sufficiently high to ensure the nonentry of sand.

The diffuser has now been in operation for 6 years and regular checks have been made as to its condition and the conditions prevailing around each diffuser. It has been established that the diffusers generally remain clear and unclogged, as anticipated by the model study. There have been instances of course when passing sand dunes, or sand waves, have completely submerged the diffusers, but generally their performance has been extremely good, as is borne out by regular divers' reports.

In addition, I believe that the City Council has carried out a number of checks on the effectiveness of the diffuser and, without entering into details of these checks, I think the best way to describe its effectiveness is to quote the now City Engineer, Mr M G G Gurr, when he described the following situation:

"We were out in a boat tethered by a length of rope to the marker pile and therefore positioned about the centre of the river some 10's of metres downstream of the outfall. Some fluorescein dye was poured into the outfall structure perhaps a little over-enthusiastically as a full five gallon bucket was poured into the outfall manhole. After what seemed ages, but was probably around a minute or so, the whole river from bank to bank changed colour in the wink of an eye. It was a highly graphic demonstration but perhaps one which should not be repeated, certainly not without a prior water right."

## CONCLUSION

It would be fair to state that experience has shown the outfall achieves good mixing into the full body of the Waikato River, in line with the initial undertaking given by ourselves to the Hamilton City Council.

## DISCUSSION

Wilkie In many rivers it would not be possible to build a diffuser pipeline of this type across the river. At Westport a new bridge was being built across the Buller River and a dropper pipe was incorporated into each pier at a total cost of about $40,000 compared with the Waikato River pipeline at a cost of about $650,000. In the Waimakiriri River pipes will also be installed on an existing bridge.

T R Healy In the model study what scaling down was used and was the sediment size scaled down?
A J Sutherland The linear model scale was 1:12. The sediment size was not scaled down because we were not interested in quantifying accurately the rates of motion, depth of scour etc. but just wanted to see if the ports would clear.

T M Hume When and how do movements in sand waves manifest themselves and are they still present?
Wilkie Divers report that sand waves are still present and at times cover ports (especially those presently still plugged) while at other times obscuring ports (even those which discharge effluent).

T R Healy Has the cross-section shown changed since installation of the pipeline?
Wilkie Divers have noticed that several of the 12 m piles supporting the pipeline now have a 5-8 mm lean downstream. They also report that parts of the cross-section have aggraded while others have degraded.

J C Rutherford Could the same degree of mixing have been achieved with a different (and cheaper) outfall structure?
Wilkie If repeating this job we would re-examine mixing carefully. A point source was examined but mathematical modelling suggested that the streaming effect was likely to cause unacceptable quality to downstream abstractors near Horotiu. In hindsight we might have been able to use the existing bridge just upstream for the diffuser rather than installing a pipeline, but operating costs would have been higher.
M G G Gurr The Waikato Valley Authority have been monitoring the river for coliforms and their results show very rapid mixing across the river which confirms the Hamilton City Council dye experiment mentioned in the paper.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre* *November 17-18, 1981*

# HUNTLY POWER STATION OUTFALL DESIGN

*E M Valentine, Central Laboratories, MWD, Lower Hutt*

## ABSTRACT

This paper is a summary of the results of the hydraulic model studies carried out at Central Laboratories in order to examine and optimise the design of the Huntly Power Station cooling water outfall.

## INTRODUCTION

The discharge of once-through cooling water from power stations is a common method of achieving the cooling necessary in the generation of electrical power by thermal means.

The Huntly Power Station's cooling water will be drawn from the Waikato River. Each of the power turbine units will take a flow of about 9.5 $m^3/s$. When the water passes through the condensers its temperature will rise by approximately $8^oC$ when the station is operating on full load.

At full load the station will require 37.9 $m^3/s$ from the river, which represents nearly one quarter of the river flow at lowest permissible stage. As thermal discharges are of the order of 10 times greater in quantity than typical wastewater discharges, the need for effective outfall design is apparent.

The initial basis for the cooling water outfall designs tested were restrictions imposed by the Water Right, that the maximum temperature rise of the river above ambient should be $3^oC$ downstream of the outfall, and that the cooling water should be dispersed uniformly over at least 40%, but not more than 50% of the river width at the point of discharge.

There are two basic problems which make the design of a cooling water outlet structure located on a river very important.

1 The returning cooling water must dissipate its excess heat as efficiently as possible. The increase in temperature of surface waters resulting from the discharge may be detrimental if the water is to be used again downstream for further cooling processes, or if it is required as a potable water supply. Secondary effects, induced by increased temperature, involving ecological changes in the biology and chemistry may also be of significance.

2 The returning cooling water should not cause major damage to the bed of the river.

The appropriate structure may well represent a compromise solution as the most effective method of thermal dispersion is often an effective scouring mechanism.

## MIXING MECHANISM

Whatever the design of the outfall, mixing of the cooling water with the receiving water takes place through two major processes.

1 Jet mixing. This type of mixing occurs when the jet velocity is different to the velocity of the receiving water. It represents a most efficient method of heat dispersion.

2 Dispersion by convective shear flow. Additional minor mixing is caused by buoyancy effects.

At the point of discharge the momentum of the discharge is greater than the receiving fluid and jet-like mixing of the heated and ambient water occurs. Ambient water is entrained into the discharge jet and the average temperature and velocity of the heated flow are reduced.

Further from the discharge point the jet velocity is reduced to the magnitude of the ambient current. In this region entrainment does not occur and the temperature distribution is determined by natural turbulent diffusion and convection. Ultimately, all of the rejected heat is passed to the atmosphere through the water surface, a process driven by the elevation of the water surface temperature.

The density difference between the heated discharge and the ambient water may inhibit mixing in the initial jet region and may promote spreading of the discharge as a heated layer.

The temperature distribution induced by a heated discharge is thus determined near the outlet by jet-like diffusion and in an outer region by natural processes. Surface heat loss and density effects occur throughout the flow making the process very complex.

Analytical predictions of temperatures in the vicinity of heated discharges are possible under the assumption that the flow is either a pure jet, or a plume.

However, in cases where the geometry is complex or where the dominant physical process cannot be isolated, it is customary to construct a physical model of the discharge and ambient waterway.

## MODEL SCALING

For accurate similarity the dispersion process must be modelled in the following way.

The Reynolds number must be high enough to allow the viscous effects to be neglected, making the jet shape independent of scale. In addition to this the turbulent dispersion process requires full similarity and a Chezy resistance coefficient equal to the prototype. This condition is satisfied by making the slope in the model the same as in prototype and providing artificial roughness. Similarity of the minor heat dispersion from buoyancy effects is achieved by making the density difference at the cooling water outlet, the same in both model and prototype. These requirements resulted in a natural scale of 1/60 (Keller and Rogers 1974; Keller 1975; Bewick and Keller 1977).

A second model with a mobile bed was used to study the scouring problem. This model (Keller and Rogers 1974; Keller 1975; Bewick and Keller 1977) was built to a distorted scale of 1/25 (horizontal) and 1/35 (vertical). The distortion was necessary to comply with mobile bed scaling rules and so that a model of sufficient depth could be built in the available space.

Later, a larger distorted scale (horizontal 1/41.84, vertical 1/15) model was built to examine the scour hole of the final design in greater detail (Bewick and Keller 1977).

## PRELIMINARY OUTFALL DESIGN

Four different discharge structures were tested and may be classified broadly into two groups according to the cooling water pump heads required. Structures 1, 2 and 3, described below, all discharge at an elevation of approximately 107.0 m. They require lower pump heads than structure 4, which discharges at an elevation of approximately 116 m (see Fig. 1).

Structure 1 Four pipe cluster. This structure was designed to provide a concentrated cooling water discharge which would promote rapid jet mixing, and hence heat dispersion adjacent to the outlet. The jet-mixed flow then drifts downstream with further heat dissipation taking place through natural turbulence. The jets were designed to spread outwards in a fan shape at angles to the main stream as shown in Fig. 1a. The prototype velocity of the cooling water jet was 2.4 m/s.

Structure 2 Multi-port Structure. This structure was developed in an attempt to lessen the scouring action of the four pipe cluster while retaining the extensive heat dissipation through jet mixing. A 20-port structure was built with the port sizes such that the prototype jet velocity of 2.4 m/s was maintained. The ports were grouped into four sets of five outlets each. The groups of outlets were established at different angles to the river flow as shown in Fig. 1b.

Structure 3 Three Point Discharge. This layout was designed to conform literally with Clause 7 of the Water Right which specified that the cooling water should be dispersed uniformly over at least 40%, but not more than 50% of the river width at the point of discharge. The layout consisted of two piers mounted in the river with a third, bank mounted structure. Each port discharged equal amounts of the cooling water (Fig. 1c).

Structure 4 Side Weir on Bridge. This structure, shown in Fig. 1d, was also designed to conform specifically with Clause 7 of the Water Right. The cooling water was piped into an aqueduct structure spanning about 40% of the river width. The water was evenly distributed over a side weir mounted on the downstream side of the aqueduct. This aqueduct must be mounted above the flood level of the Waikato River, therefore the pumping head required is higher than for the three previous designs.

## RESULTS OF PRELIMINARY TEST

(a) Structure 4, the side weir, was ineffective as a heat disperser and would have required a massive foundation to be protected against scour. Hot spots and streams occurred over extensive areas of the river flow.

(b) The four pipe cluster, structure 1, was effective as a heat disperser, but exerted strong scouring action on the river bed. The resulting scour hole could have detrimentally effected the dispersion characteristics.

(c) Structure 3, the three point discharge configuration, was found to be unsatisfactory for both heat dispersion and scouring action.

(d) Structure 2, the multi-port diffuser, was effective as a heat disperser and exerted only a mild scouring action on the bed. Further testing was, therefore, directed to the optimisation of this alternative.

## THE MULTI-PORT DIFFUSER

Two sets of tests were performed to optimise the dispersion characteristics of this outlet design (Fig. 2).

The first set were carried out to examine different port angle configurations to eliminate a warm stream which clung to the western bank and had a temperature at the limit of the model which was above the allowable $3^{o}C$ over ambient. A second set of tests were carried out to assess the effects of the Waikato Valley Authority river training scheme, which calls for a change in bankline (shown in Fig. 3).

Further large scale mobile bed model tests were also carried out.

## CONCLUSIONS

These conclusions are a summary of the results from the final tests leading to the proposed multi-port diffuser design.

(a) It was found that the temperature characteristics of the thermal plume were negligibly influenced by the port orientations but that this had a large influence on the scouring characteristics. It was concluded

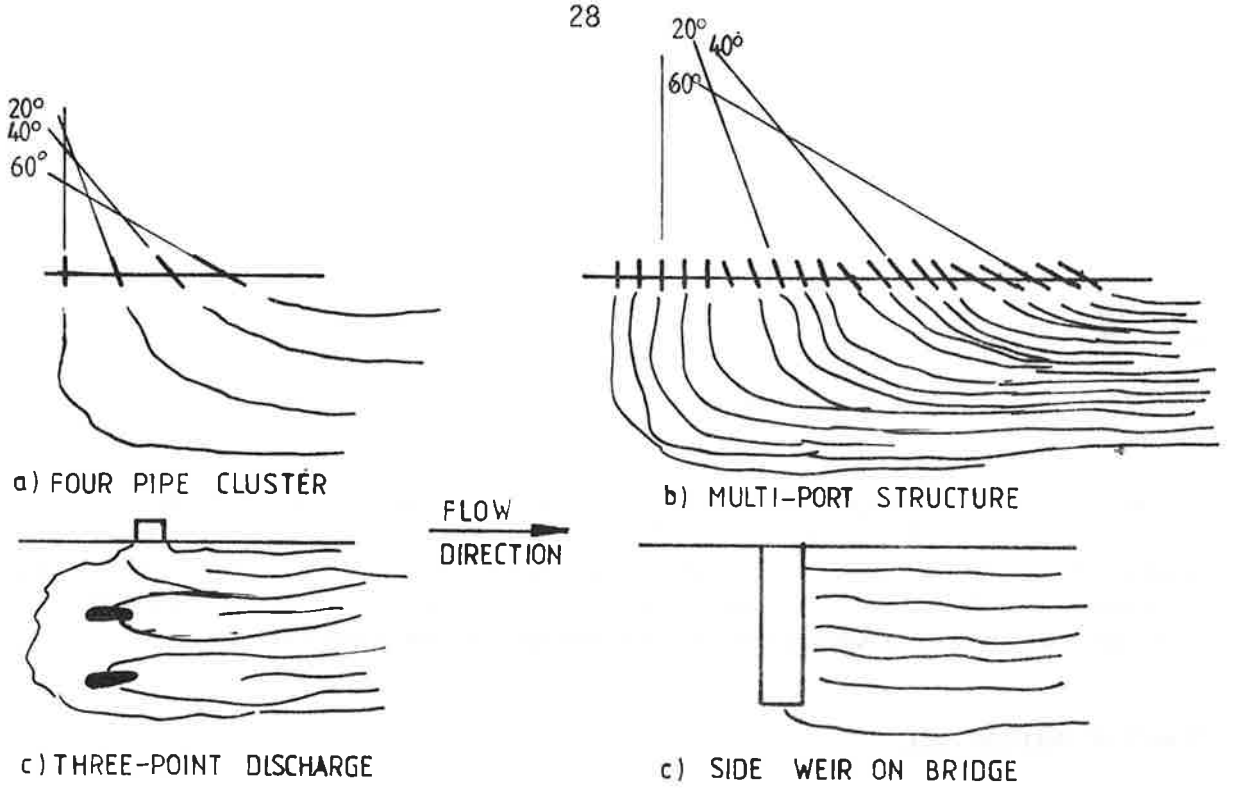



FIG.1 Schematics of Cooling Water Outlet Structures

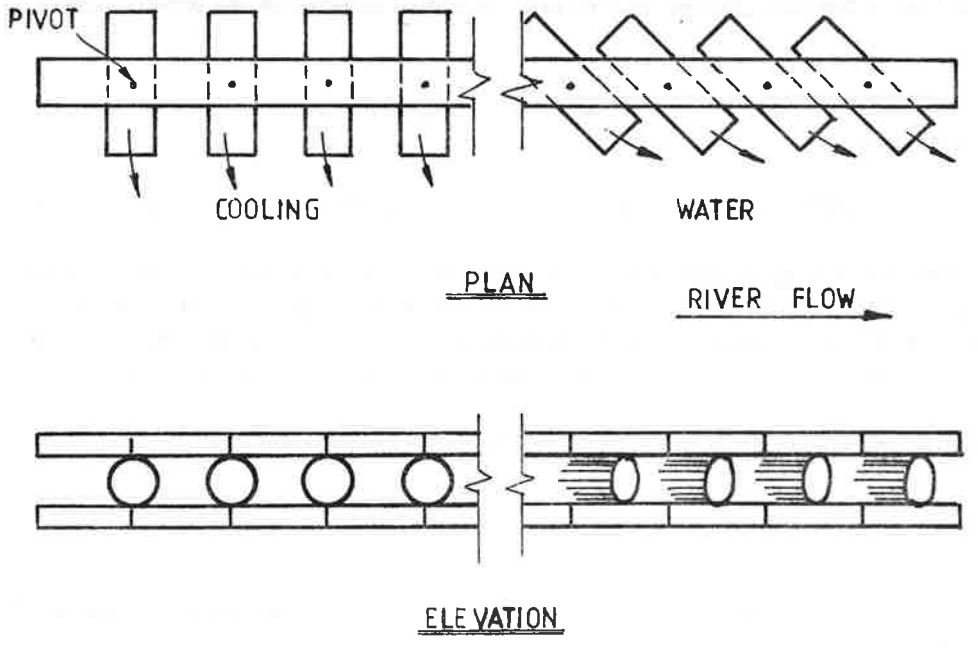



FIG. 2 Schematic of Multi-Port Variable Angle Outlet Structure

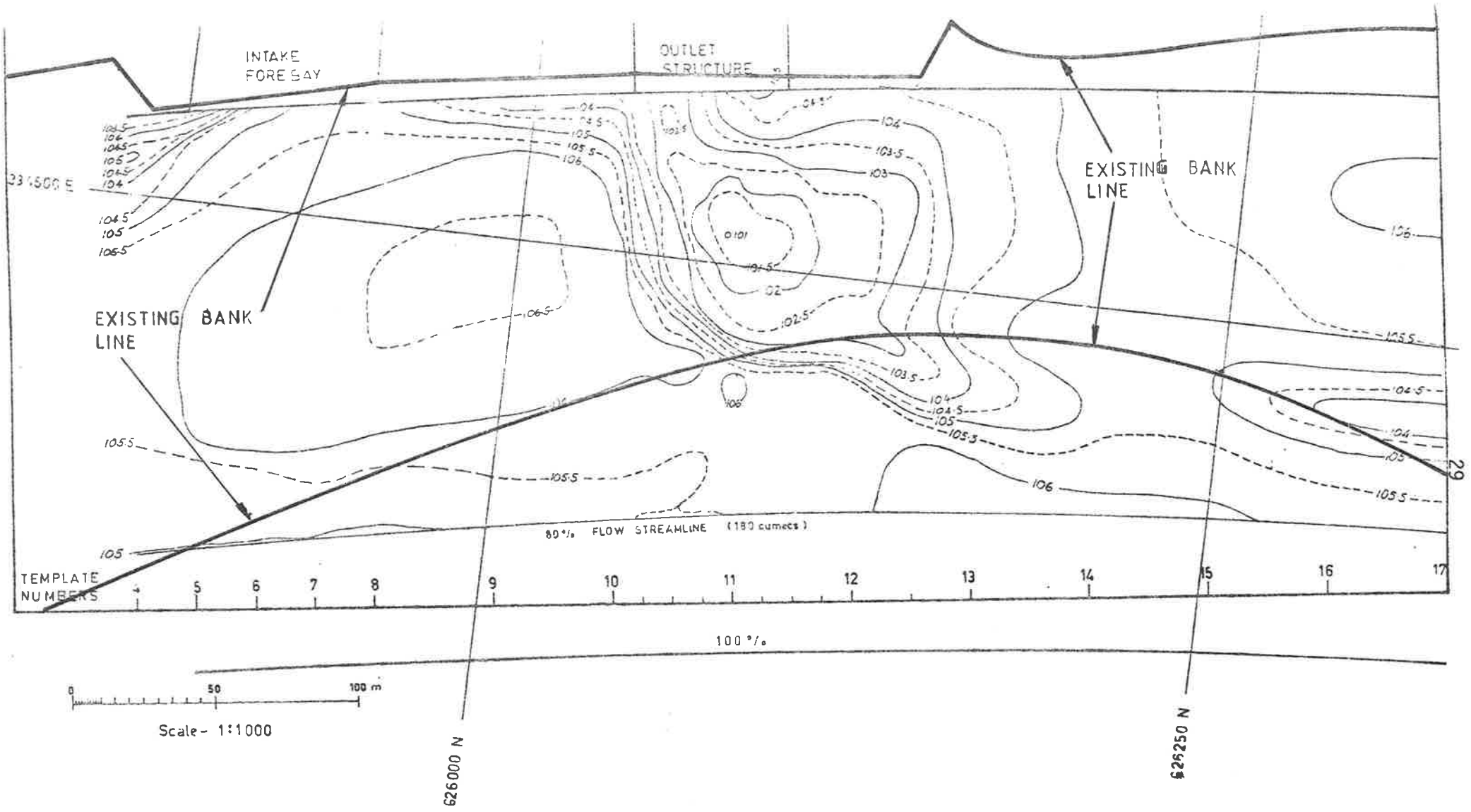



FIG. 3 BED AND SCOUR HOLE CONTOURS OF W.V.A. BANKLINE STUDY WITH COOLING WATER TOTAL RIVER FLOW 180 cumecs, WATER LEVEL 107.0 m R.L.

from this that the best port orientations were those which discharged the cooling water as uniformly as possible over as wide an area as possible.

(b) This configuration was found to be $90^{o}$ - $70^{o}$ - $50^{o}$ - $30^{o}$. (These are the respective angles of each set of four outlets to the channel bank line.) The temperature characteristics of the thermal plume were shown to be most critical at the lowest river discharges. However, even for this flow, the maximum temperature in the river dropped to less than $3^{o}C$ above ambient within 100 m of the downstream edge of the outfall. As the river discharge increased so the temperature in the river reduced, as did the width of the thermal plume.

(c) The presence of a scour hole adjacent to the outlet structure shown in Fig. 3 enhanced thermal dispersion by drawing an increased quantity of the river flow into the thermal plume.

(d) The WVA training scheme (see Fig. 3) did not allow satisfactory performance of the multi-port structure with the lowest flows. However, the thermal mixing of the structure was improved greatly by halving the area of the outlet ports (from 1 m diameter to 0.71 m diameter). The additional head to drive this will be available with the proposed seal pit arrangement.

(e) Scour associated with the thermal plume was more severe from the WVA scheme than for the existing channel but was not excessive.

ACKNOWLEDGEMENTS

The work described here was carried out at Ministry of Works and Development's Central Laboratories by Mr N C Rogers, Dr R J Keller and Mr D J Bewick.

REFERENCES

Keller, R.J.; Rogers, N.C. 1974: Huntly Power Station Cooling Water System: Preliminary Studies of Outlet Structure Geometry. MWD Central Laboratories Report No. 501.

Keller, R.J. 1975: Huntly Power Station: Hydraulic Studies of Multi-Port Outlet Structure. MWD Central Laboratories Report No. 3-75/2 Vol. 1 and Vol. 2.

Bewick, D.J.; Keller, R.J. 1977: Huntly Power Station: Large Scale Mobile Bed Hydraulic Model Study. MWD Central Laboratories Report No. 3-77/3.

DISCUSSION

M J O'Sullivan In the case of the "side weir" discharge did significant thermal stratification occur?
Valentine I believe that some thermal stratification must have occurred in the model of this alternative because this was shown to be an inefficient method of thermal dispersion. This must have been largely due to the lack of the true 'jet mixing' phase.

J C Rutherford Based on the previous paper, David Wilkie's solution to this problem might have been 10 point sources on a diffuser pipe stretching halfway across the channel. Does this strike you as an efficient structure?
Valentine David Wilkie's solution would appear to be an efficient diffusion method. However, the scouring mechanism may have been more of a problem at Huntly. It must be recalled that the quantities of thermal discharge can represent up to 25% of the river flow, which are very much greater than sewage effluent discharges. Construction of such an alternative may have increased costs in a similar fashion to the 3-point discharge and the overflow weir.

A J Sutherland What were the prototype scour hole depths and how did they compare with expected natural fluctuations in bed level due to passage of sand waves? Would these sand waves interfere significantly with the action of the diffuser?
Valentine Maximum prototype scour depths predicted by the model were:
(a) for existing bankline approximately 1.2 m, and
(b) for WVA bankline approximately 3 m.
These depths would be greater than the height of expected sand waves in the natural riverbed but would be very unlikely to interfere with the diffuser in the case of the multi-port bank mounted structure.

P Tortell Since, in dispersing heat one could use the atmosphere as well as the water medium, has any thought been given to using a curtain of bubbles to remove some of the excess heat before the thermal effluent reaches the receiving water?
Valentine This method was not considered for this power station as far as I am aware but supplementary cooling (e.g., cooling towers, canals or ponds) are being considered for the Second Waikato Thermal Power Station.

D A Carter Would you comment on the ecological constraints looked into as part of the study and temperature dispersion in the river because the 750 m boundary zone appears to be somewhat excessive. Was this a condition on the Water Right?
Valentine Ecological constraints were considered by NWASCO in drawing up water right conditions. This study was aimed at designing a discharge structure which would comply with those conditions. The 750 m boundary was not mentioned in the Water Right but was adopted by common consent.

A G Barnett Since the ultimate heat sink is the atmosphere what consideration was given to possible differences between the model and prototype with respect to heat losses to the atmosphere?
Valentine In the model studies water temperatures, atmospheric temperature and humidity were kept the same as in the prototype. However, any differences in scaling for the atmospheric heat loss would probably only become important in the far field region beyond the scope of the model and compliance with the Water Right. The model would probably tend towards a lower heat loss to the atmosphere.

G J MacDonald Did the model study investigate changes in the vertical orientation of the diffuser ports as well as looking at the effects of pivoting the ports in the horizontal plane?
Valentine I am not aware that vertical angles for the diffuser ports were considered. It would probably have induced more severe scour on the river bed if downward angles were employed to encourage vertical mixing, and jet dilution might have been decreased.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre* *November 17-18, 1981*

# TARANAKI RING PLAIN DYE STUDIES
# PRELIMINARY RESULTS

*J Kitto, Taranaki Catchment Commission, Stratford*
*J C Rutherford, Water & Soil Science Centre, MWD, Hamilton*

## ABSTRACT

Dye experiments have been conducted in five Taranaki Ring Plain rivers to measure time of travel and longitudinal dispersion. Mass deficits were encountered, attributable to underestimation of river flow and long "tails" in dye profiles. The effects of "dead zones" may need to be quantified in order to simulate observed profiles accurately.

## INTRODUCTION

The work described in this paper is part of the Taranaki Ring Plain Water Resources Survey. This study is not concerned with addressing the impact of any particular water user, but is being conducted to develop techniques and assemble a data bank which will enable water managers to make a rapid and accurate assessment of the potential impact of specific projects as and when they arise. One aspect of the study is to measure the time of travel and the rate of longitudinal dispersion of a conservative tracer in several streams. The intention is to extrapolate from these data to other waterways as required. The streams mentioned here were chosen as being representative of the types found in the region. It is envisaged that the time of travel and dispersion will be used to assess the impact of industrial accidents, intermittent discharges and abstractions, while the time of travel data alone will be used to assess the impact of continuous discharges.

## METHOD

The dye used was Rhodamine WT, an anionic xanthene derivative which has low adsorption properties and photochemical decay rate, is stable at natural stream pH and temperature ranges, and is detectable at low concentrations. Rhodamine WT was manufactured as a 20% solution by du Pont. Dye was poured into each river from a bridge to make a line-source across the whole width. Dye was measured continuously in the field using a Turner Filter Fluorometer Model III. The fluorometer was fitted with a far-UV lamp, 546/590 filters, and a continuous flow door. Readings were recorded on a Toa Polyrecorder. Streamwater was pumped through ½ inch plastic tubing by a ½ hp Davies centrifugal pump situated upstream of the fluorometer. A fine mesh plastic filter just before the fluorometer caught large particles in the water and dissipated air bubbles in the line. A petrol generator powered the fluorometer and pump.

Samples for calibration (15-20 for each experiment) were taken at regular pre-determined increments of concentration using a valve situated between the pump and fluorometer and were stored in polyethylene containers in darkness at ambient air temperature. Water temperature readings were taken at the valve at half-hourly intervals. Calibration curves were determined in the laboratory using a sample-holder door prior to the experiments. Immediately after each experiment the samples taken for calibration were read on the sample door and the results used to calibrate the flow door readings. Standard temperature correction factors (Wilson 1968) were applied to both sample and flow door readings. The accuracy was approximately $\pm$ 15% at high concentrations, increasing to $\pm$ 50% at the detection limit (0.02 $mg/m^3$).

Streams were wade-gauged using a Gurley current meter at the injection and sampling sites and in some instances at intermediate sites. The Stony, Punehu and Kapuni were gauged at sites located at the upstream end of pools where the bed was rocky and the flow probably accurately measured to $\pm$ 10%. The Waiongana was

gauged at a weir and the Manganui, whose bed comprised fine gravels, at a natural rock sill, giving an accuracy of about ± 5%.

Mass determinations

The integral under the concentration versus time profile at each sampling site was evaluated using a common numerical technique (the trapezoidal rule), and the total mass passing each site was estimated by multiplying this integral by the gauged flow. The accuracy of mass determinations was taken as the sum of the accuracy of the concentration measurement (± 15%) and the flow measurement (± 10% or ± 5%).

Velocity and dispersion coefficient

The one-dimensional Fickian model for longitudinal dispersion in a uniform channel was fitted to experimental data. This well-known model is

$$\frac{\partial C}{\partial t} + U\frac{\partial C}{\partial x} = D\frac{\partial^2 C}{\partial x^2} \qquad (1)$$

where t = time, x = distance downstream, C(x, t) = concentration, U = mean velocity and D = dispersion coefficient. If concentration is known as a function of time at site $x_1$, then the concentration at the downstream site $x_2$, as a function of time may be obtained from the "frozen cloud" solution to equation (1).

$$C(x_2, t) = \int_{-\infty}^{\infty} \frac{C(x_1, \tau)}{\sqrt{4\pi D\,(t_2 - t_1)}} \exp\left(\frac{(x_2 - x_1 - U(t-\tau))^2}{4D(t_2 - t_1)}\right) U d\tau \qquad (2)$$

where $t_1$, $t_2$ = times at which the centroid of the dye profile passes sites $x_1$ and $x_2$ and $\tau$ = dummy time variable of integration.

The mini-computer program ROUTE (Rutherford 1981), which makes use of equation (2), was used to find values of U and D which gave the best fit between observed and predicted dye concentration profiles at the downstream site in each experiment. Upstream profiles were either the observed profiles at the intermediate site or a synthetic concentration "spike" one minute long at the input site containing the amount of mass discharged.

Two sets of D values were derived. The first set used concentration profiles exactly as they had been measured. The second set used profiles which had been scaled up so that the mass at each site equalled the mass discharged.

## RESULTS

Profile shape

Figure 2 shows dye concentration versus time profiles measured in each of the six dye experiments. A striking feature of these Taranaki data when compared with results of similar experiments from other NZ rivers (Day 1975; Rutherford *et al.* 1980; Rutherford *et al. ibid*) is the presence of long "tails". These are most noticeable in the Punehu, Manganui and Kapuni 2 profiles. Sampling did not continue over the complete profile at some sites, notably in the Stony. It is, however, difficult to make an objective comparison of these with other data using just dye concentration versus time profiles.

Figure 3 shows the data standardised using the transformation suggested by Day and Wood (1976). This facilitates making a comparison of shape between profiles. However, a quantitative comparison was not possible because the uncertainty of the transformed concentrations was not determined.

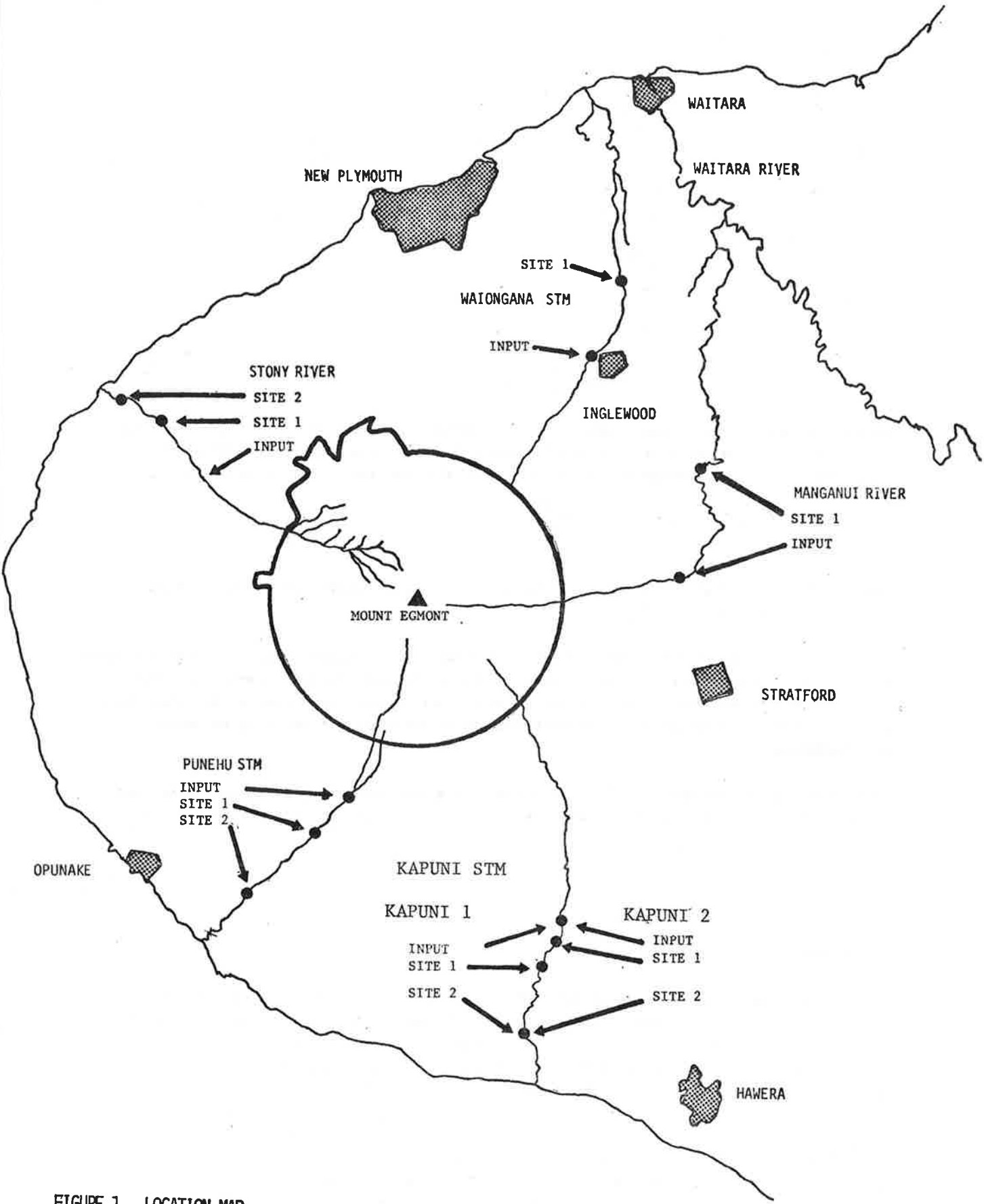



FIGURE 1 LOCATION MAP

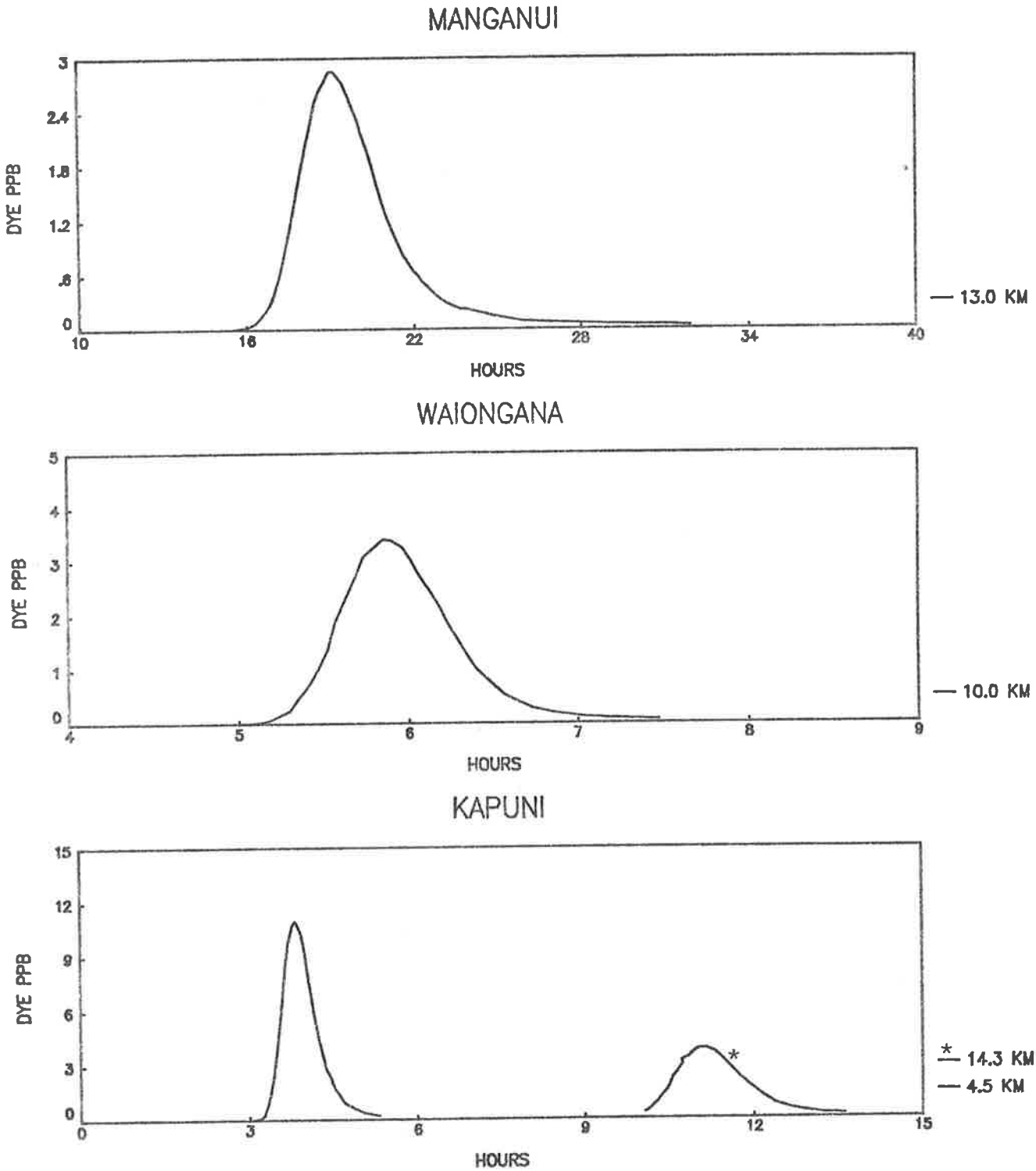



FIGURE 2 Observed dye profiles.

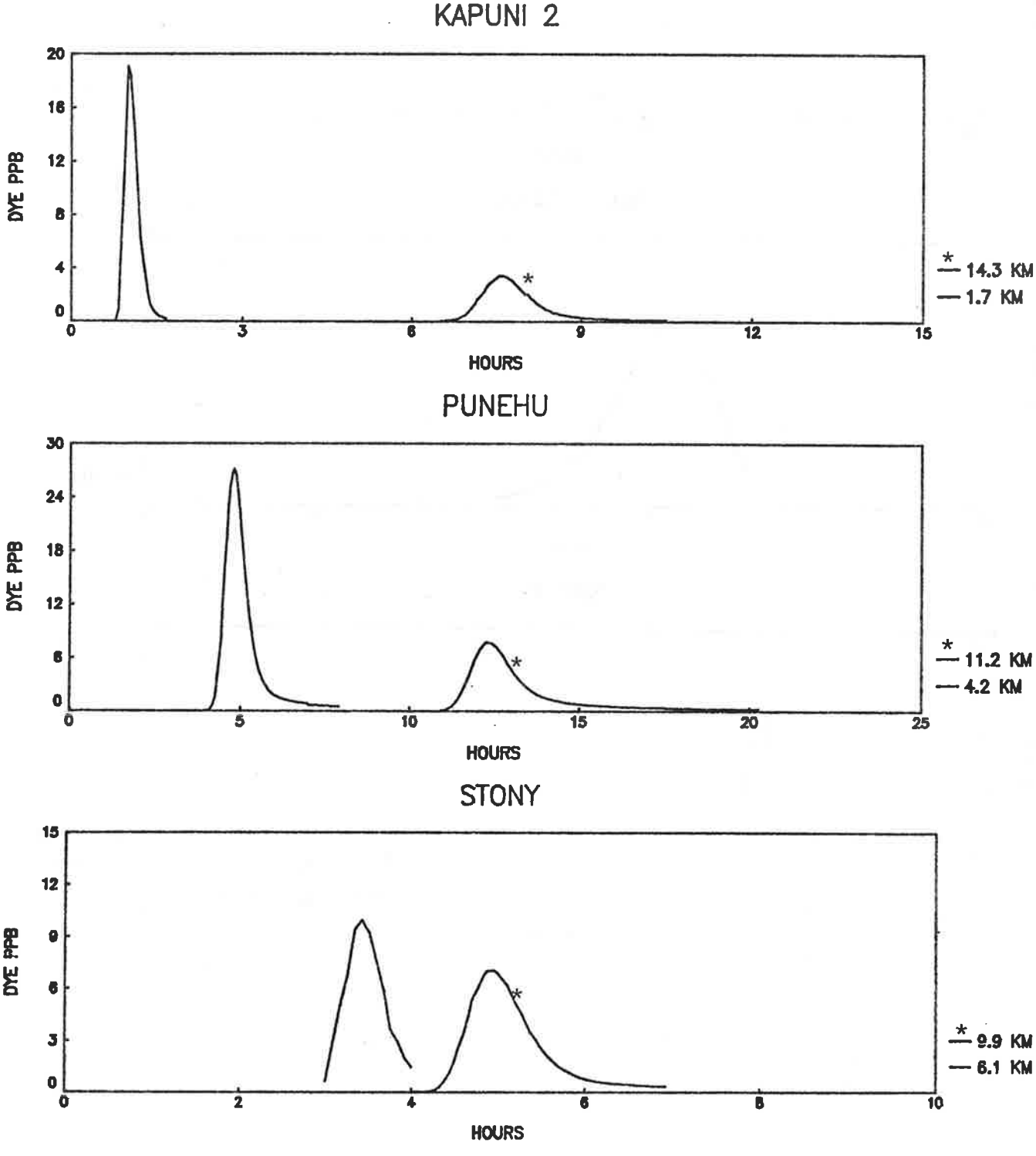



FIGURE 2 cont.

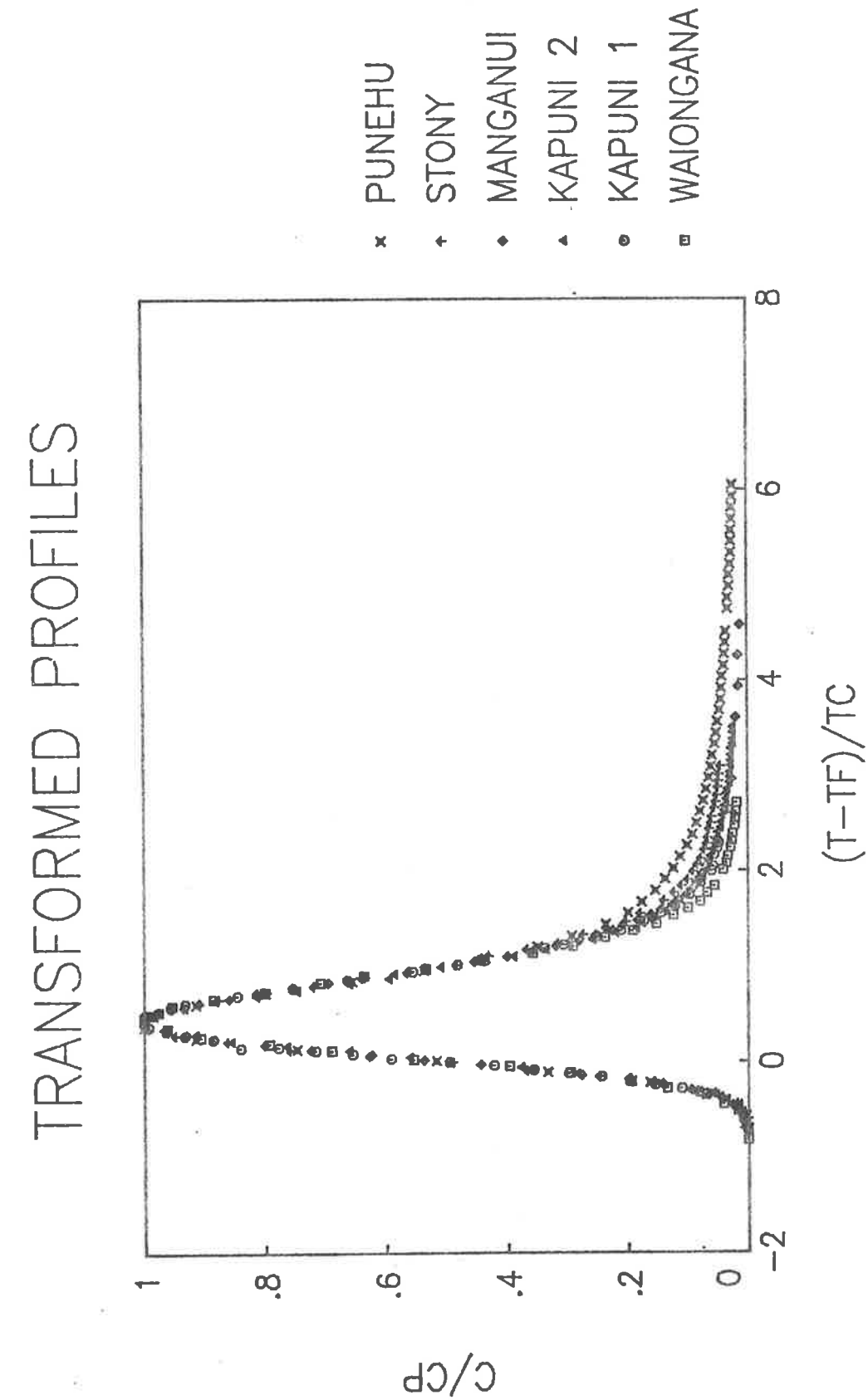



FIGURE 3 Transformed dye profiles

Mass balance

From Table 1 it can be seen that in each experiment less mass was observed to pass each sampling site than was discharged. To test the significance of the observed mass deficit, a null hypothesis was that the deficit was zero while the 90% confidence interval of the deficit was assumed to be either ± 25% (Punehu, Kapuni 2, Stony) or ± 20% (Waiongana, Manganui) of the injected mass.

Considering the Waiongana and Manganui separately, one would conclude that at the 90% confidence level the observed mass deficit was not significantly different from zero. Assuming a normal distribution, one might estimate that there was a 10% probability of the mass deficit in the Waiongana and Manganui being equal to or greater than the observed value. Taking these two experiments as independent, then the joint probability of the mass deficit in both rivers being as high as observed is 1%.

Considering each of the other rivers separately, one would conclude that the observed mass deficit was significant in the Punehu and Kapuni 1 at both sites and in the Kapuni 2 and Stony at the furthest downstream site only. The joint probability that all six experiments should produce the observed mass deficits where none existed would be very small indeed.

There are at least two possible factors which may have contributed to the observed mass deficits.

Firstly, the river flow may have been underestimated. This was most likely on the Punehu, Kapuni and Stony Rivers where the river bed was very rocky at each gauging site and appreciable flow might have occurred amongst the boulders where it could not be measured. By comparison the Manganui was gauged at a site where there are few rocks, and the Waiongana at a weir; in these rivers the mass deficit was small.

Secondly, low dye concentrations may have persisted in the "tail" of the dye profiles for a considerable time after sampling stopped. Making the assumption that each flow gauging was correct then, for all the dye discharged to have passed the bottom sampling points in the Stony, Punehu and Kapuni Rivers, concentrations of about 0.1 $mg/m^3$ would need to have persisted for 24-36 hours.

It is possible that a combination of these two factors could have produced the observed mass deficits. Comparison of the mass passing the intermediate and downstream sites in the Kapuni 1, Kapuni 2, and Punehu provides some indication of the relative importance of these two factors. Were the flow to be underestimated and the "tail effect" to be negligible then one would anticipate a mass deficit between the input and the intermediate site but not between the intermediate and downstream site. Conversely, if flow were to be estimated with the same accuracy at all sites, a significant discrepancy between the intermediate and downstream site would indicate that the "tail effect" was important. From Table 1 it would appear that the predominant factors are: flow underestimation for Kapuni 1, "tail effect" for Kapuni 2 and both for Punehu. Flow effect was thought to be small in the Manganui and Waiongana and the deficits observed may be attributed to the "tail effect".

Overall the "tail effect" would appear to cause a mass deficit of about 20% in the Waiongana and Manganui while the flow underestimation and "tail effect" appears to cause a deficit of 25-40% in the Kapuni, Punehu and Stony.

Velocity estimates

Table 2 summarises the values of velocity which gave the best fit between observed profiles and predictions made using the Fickian dispersion model described above. Identical velocity values were derived using both the "unmodified" observed concentration profiles and the profiles scaled up to achieve mass conservation.

TABLE 1 Summary of dye test results

| River | Input | | | Site 1 | | | Site 2 | |
|---|---|---|---|---|---|---|---|---|
| | flow $m^3/s$ | mass g | distance km | flow $m^3/s$ | mass g | distance km | flow $m^3/s$ | mass g |
| Punehu | 0.378 | 40 | 4.2 | - | 30.4 | 11.2 | 0.445 | 23.4 |
| Manganui | 1.65 | 120 | 13.0 | 2.94 | 104 | - | - | - |
| Waiongana | 1.29 | 40 | 10.0 | 3.6 | 34.3 | - | - | - |
| Stony | 3.11 | 100 | 6.1 | 3.27 | 64.9* | 9.9 | 3.41 | 77.7 |
| Kapuni 1 | - | 35.5 | 4.5 | 0.997 | 27.8 | 14.3 | 1.16 | 24.2 |
| Kapuni 2 | 1.65 | 40 | 1.8 | 1.75 | 33.5 | 14.3 | 1.73 | 22.6 |

Note * sampling stopped early, see Figure 2.

Dispersion coefficient estimates

Figure 4 illustrates the situation encountered when attempting to reproduce the observed peak concentrations using the "frozen cloud" solution to the one-dimensional Fickian longitudinal dispersion model.

If the known mass input was routed so that the peak concentrations matched those of the unmodified observed profiles, then large values of D were needed and the predicted concentrations were consistently higher than the observed concentrations in the rising and falling limb (as a simple consequence of the differences of mass). These D values are a likely upper bound on the true value in each channel.

If the observed profiles were scaled up so that their mass equalled that discharged, then smaller D values were needed. The observed concentrations still tended to rise later (but then more rapidly) and to fall more slowly than the predicted concentrations. This is partly attributable to the shortcomings of the "frozen cloud" approximation, but also reflects the inability of the Fickian model to describe accurately the processes operating. These latter D values give a likely lower bound on the true value in each channel.

The two sets of D values are summarised in Table 2. These values can be used to make estimations of peak concentrations of conservative tracers. The two D values will give upper and lower bounds on the peak concentration. They are unlikely to give an accurate estimate of the concentrations in the rising and falling limbs of the profiles.

Dead zones

The long "tails" exhibited by these Taranaki data and the poor match obtained between observations and predictions made using the Fickian dispersion model suggest that "dead zones" may be important : notably in Punehu, Kapuni and Stony Rivers.

"Dead zones" are regions of stagnant water associated with, for example, backwaters, boundary layers around obstacles, and holes in the river bank or bed which temporarily trap dye and subsequently release it slowly. They have been shown to afford an explanation for long "tails" in observed dye profiles. Valentine (*ibid*) will address the effects of "dead zones" in detail.

In this preliminary analysis no attempt was made to use the recently developed theory and methodology relating to "dead zones" to analyse the Taranaki data, although this is a possibility for the future.

TABLE 2 : Summary of channel characteristics and derived velocities and dispersion coefficients

| River | Channel Characteristics | | | Input - Site 2 | | | | Site 1 - Site 2 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Raw | | Scaled | | | Raw | | Scaled | |
| | Width | Depth | Shear Velocity | Velocity | | | | | Velocity | | | | |
| | b | d | u* | U | D | D/du* | D | D/du* | U | D | D/du* | D | D/du* |
| | m | m | $m.s^{-1}$ | $m.s^{-1}$ | $m^2.s^{-1}$ | - | $m^2.s^{-1}$ | - | $m.s^{-1}$ | $m^2.s^{-1}$ | - | $m^2.s^{-1}$ | - |
| Punehu | 5 | 0.28 | 0.21 | 0.26 | 16 | 272 | 7 | 119 | 0.27 | 13 | 221 | 7.3 | 124 |
| Manganui | 20 | 0.40 | 0.18 | 0.19 | 8.5 | 118 | 6.5 | 90 | | | | | |
| Waiongana | 13 | 0.60 | 0.24 | 0.48 | 9.0 | 62 | 6.8 | 47 | | | | | |
| Stony | 10 | 0.63 | 0.30 | 0.55 | 23.5 | 124 | 13.5 | 71 | | | | | |
| Kapuni 1 | 9 | 0.30 | 0.15 | 0.36 | 16.5 | 367 | 7.7 | 171 | 0.38 | 20 | 445 | 9 | 200 |
| Kapuni 2 | 10 | 0.35 | 0.17 | 0.53 | 36.5 | 613 | 11.8 | 193 | 0.53 | 26 | 437 | 13 | 213 |

Notes : 1 Raw = dye profiles used as measured.

2 Scaled = dye profiles scaled up so that mass equals mass injected.

3 Channel characteristics are derived from gauging cards and aerial photographs.

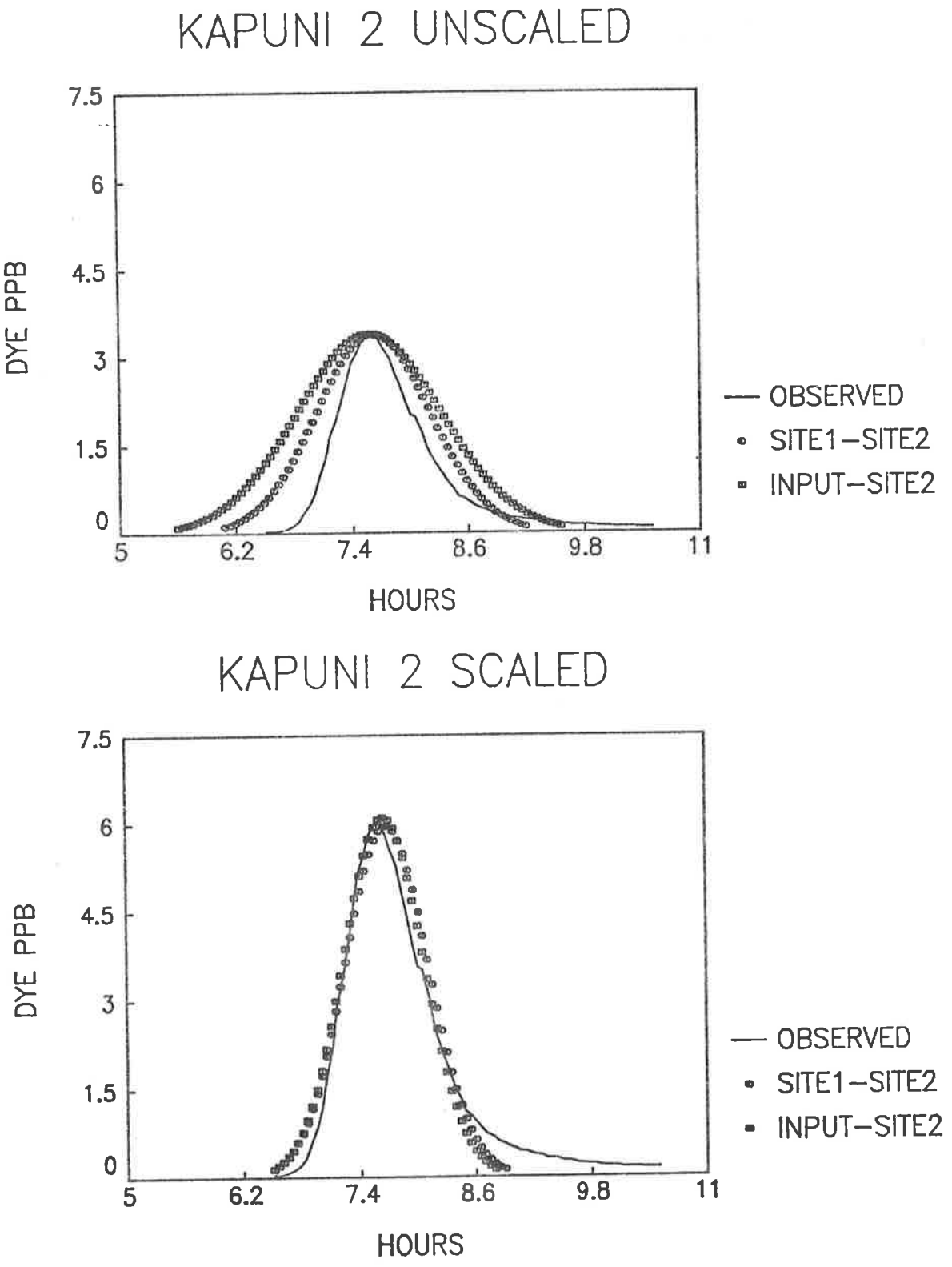



FIGURE 4 Observed and predicted profiles, Kapuni 2.

## CONCLUSIONS

Dye tests conducted on five Taranaki Rivers have yielded estimates of velocity which should prove useful if and when pollution impact assessments are needed.

There were deficits between the dye mass discharged and the mass observed to pass each sampling site. In two of the experiments the uncertainty in the gauged flow was known to be low, and the small mass deficits were attributable to the presence of long "tails" on the profiles in which concentrations were very low (at or below the limit of detection) but which, because of their length (possibly 24-36 hours) contained appreciable mass. In the other four experiments it was likely that the gauged flows were underestimated because the channels were rocky and the mass deficit was the result of both the "tail effect" and the underestimation of flow.

Two sets of longitudinal dispersion coefficients were derived which give upper and lower bounds to the likely true values.

## SUGGESTIONS FOR FURTHER WORK

Consideration could be given to applying "dead zone" models to these channels. The accuracy of flow gauging techniques in rocky channels may benefit from some investigation, possibly using continuous dilution gauging procedures.

These refinements are considered necessary prerequisites to any attempt to relate dispersion coefficients to hydraulic and flow parameters : required if dispersion rates are to be extrapolated to other channels.

## REFERENCES

Day, T.J. 1975: Longitudinal dispersion in natural channels, Water Resources Research, Vol. 11, No. 6, 909-918.

Day, T.J.; Wood, I.R. 1976: Similarity of mean motion of fluid particle dispersing in a natural channel, Water Resources Research, Vol. 12, No. 4, 655-666.

Rutherford, J.C.; Taylor, M.E.U.; Davies, J.D. 1980: Waikato River pollutant flushing rates, ASCE Vol. 106, No. EE6, 1131-1150.

Rutherford, J.C. 1981: Handbook on Mixing in Rivers. Water & Soil Misc. Pub. No. 26, MWD, Wellington.

Wilson, J.F. 1968: "Fluorometric procedures for dye tracing". Techniques of Water-Resources Investigations of the US Geological Survey Book 3 : Applications of Hydraulics Chapter A12. US Dept. of the Interior.

## DISCUSSION

G B McBride Did you sample at just one point across the river?
Kitto We sampled at one point in the channel near the centre of the main flow.

R W Fullerton We have found that Rhodamine B adsorbed onto the plastic tubing in our sampling apparatus. Have you experienced such difficulties and could the long tails be simply an artifact of adsorption?
Kitto We checked this by taking river samples to supplement the continuous traces and although these indicate there could be some slight adsorption, nevertheless, we are certain that the long tails are real. We have not used Rhodamine B which has the reputation for adsorbing readily but are investigating the adsorption of Rhodamine WT on glassware etc. and its breakdown in sunlight.

E M Valentine Could you comment further on how the quantity of tracer recovered at the weir compared with the other sites?
Kitto For the Waiongana 40 g were discharged and 34.3 g were recovered at the weir.

D A Carter Was any sampling done immediately below the injection site?
Rutherford During the Kapuni 2 survey 40 g were injected, 33.5 g were recovered at site 1 and 22.6 g at site 2. Thus 6.5 g were lost in the first reach and a further 10.9 g lost in the second reach.

M E U Taylor Were there any temperature differences in the stream which might account for this loss?
Kitto Temperature correction factors were applied to all samples.

K E Parnell In converting from volume to mass what specific gravity did you use?
Kitto The manufacturers quote a specific gravity of 1.20 whereas in fact it is bout 1.09. Since we used dye to make up standards for calibration, the initial concentration of the stock solution is not critically important.

M E U Taylor You have to dilute stock solution by many thousands to get standards. Is this a possible source of error? Normally you need to use at least 100 ml at each dilution step.
Kitto We took 10 g into 2 litre (stock solution) nominally 20% and then 10 ml into 1000 ml for successive dilutions.
Rutherford Errors in making up standards could explain the difference between mass injected and mass observed at any site but some other mechanism must be responsible for the apparent loss of dye between sampling sites.

A J Sutherland Have you compared your results with those of Dr T J Day who carried out several studies in Canterbury rivers?
Kitto Some of his transformed profiles looked quite similar to our results.
Rutherford From my reading of Dr T J Day's papers it appears that he scaled all of his profiles to achieve 100% mass conservation but I could find no comments on whether he found mass conservation or not.

E M Valentine How did the mean velocity from gaugings compare with the mean transport velocity?
Kitto Gaugings were made at only 2-3 sites often in pools, and it is difficult to compare mean water with mean transport velocity.

A G Barnett Some years ago during tests at Tekapo B Power Station where there was effectively a long pipe between input and sampling site we got very close to 100% dye recovery, but at Tekapo A Power Station where there was a surge chamber between input and sampling site we got very long tails and a much reduced mass recovery.

A G Barnett Can you say how the results can be useful in the case of a continuous discharge?
Kitto If continuous chemical or biological processes occur then time of travel is important for assessing the yield of such processes.

A J Sutherland You mentioned that these studies are being done in the expectation that results will be useful in assessing the impact of petrochemical and other development. Who funded the study?
Kitto It was partly funded by the Taranaki Catchment Commission and partly by the National Water and Soil Conservation Organisation.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre November 17-18, 1981*

# THE INFLUENCE OF LONGITUDINAL DISPERSION ON WATER QUALITY IN THREE NZ RIVERS

*J C Rutherford, Water & Soil Science Centre, MWD, Hamilton*
*B W Gilliland, Manawatu Catchment Board, Palmerston North*
*G B McBride, Water & Soil Science Centre, MWD, Hamilton*

## ABSTRACT

A one-dimensional model of mass transport has been used in studies of the Manawatu, Waikato and Waipa Rivers. Methods and results are reviewed briefly.

Longitudinal dispersion was found to have a negligible influence on DO in the Waikato River, where aquatic plant metabolism dominated. It was found to have only a minor effect on BOD concentration in the Manawatu River where inflow rate varied diurnally but where velocity and removal rate dominated.

In the Waipa River after the emergency dumping of milk, dispersion influenced BOD slightly and DO only very slightly. Predictions were very sensitive to variations of BOD decay and reaeration rate.

In the Waikato River, longitudinal dispersion was critically important in predicting concentrations of conservative tracer downstream from an instantaneous point discharge.

The Fickian model of longitudinal dispersion was found to be adequate for predicting tracer concentrations near the peak, but mismatches were obtained consistently for low concentrations in the leading and trailing edges of profiles.

## INTRODUCTION

Close to the point at which tracer or pollutant is injected into a river channel, concentration is influenced by advection and dispersion in each of the three coordinate directions. As it is carried downstream by the current, tracer impinges on the bed and eventually becomes vertically well-mixed. Further downstream tracer impinges on the banks and eventually becomes laterally well-mixed. This leaves downstream advection and longitudinal dispersion as the important transport processes. Following work by G I Taylor on longitudinal dispersion in pipeflow it was suggested that tracer transport in an open channel could be described by the so-called "Fickian" model (Fischer 1973) :

$$\frac{\partial}{\partial t}(AC) + \frac{\partial}{\partial x}(AUC) = \frac{\partial}{\partial x}\left(AD \frac{\partial C}{\partial x}\right) - kAC \qquad (1)$$

where t = time, x = distance downstream, C = cross-section average concentration, A = cross-section area, U = mean velocity, D = longitudinal dispersion coefficient, and k = a first order decay rate coefficient (zero for a conservative solute such as dye).

Strictly equation (1) is not valid close to the point of injection but only applies outside the "advective zone" which extends downstream for a distance estimated by Fischer (1973) to be

$$L = 0.45\ Ub^2/R\ u^* \qquad (2)$$

where b = channel width, R = hydraulic radius, and u* = shear velocity.

Within the advective zone concentration versus distance profiles measured after slug injections are invariably more highly skewed than solutions to equation (1) and values of D and U required to match observed and predicted concentrations vary with time. Below the advection zone in a uniform channel concentration versus distance profiles approach the Gaussian bell-shape while D and U can often be taken as constants.

For accurate simulation of pollutant transport within the advective zone a more complex model than equation (1) is required such as the "streamtube model" (Fischer 1966).

If concentration is known as a function of time at location $x_1$ and U, D and A are constant, then concentration may be predicted as a function of time at downstream location $x_2$ using the approximate "frozen cloud" solution to equation (1):

$$C(x_2,t) = \int_{-\infty}^{\infty} \frac{C(x_1,\tau)}{\sqrt{4\pi D\,(t_2-t_1)}} \exp\{-\frac{(x_2-x_1 - U(t-\tau))^2}{4D\,(t_2-t_1)} - k(t_2-t_1)\}U d\tau \qquad (3)$$

where $t_1$, $t_2$ = times of passage past sites $x_1$, $x_2$, and $\tau$ = dummy time variable of integration.

Equation (3) provides a powerful tool for estimating values of U and D from field data (most commonly when both $x_1$ and $x_2$ are below the advective zone) and can also be used to make predictions of concentration.

This paper summarises the findings of studies in the Manawatu, Waikato and Waipa Rivers where equations (1) and (3) have been used to model pollutant concentration.

## MANAWATU RIVER

### INTRODUCTION

The Manawatu River is approximately 226 km long and drains a predominantly agricultural catchment of some 6,000 $km^2$. At Palmerston North (79 km from the sea) the mean flow is 54.6 $m^3/s$ (range 9-2430 $m^3/s$ over the period 1971-1981), the river is steep (1 m/km), shallow, fairly swift, meanders extensively and has a shingle bed.

In the 5 km reach below Palmerston North the river receives effluent from three major discharges (see Table 1, and Figure 1). The combined effect of these discharges presently causes some degradation of water quality in the river, notably dissolved oxygen depletion and sewage fungus growth (Currie and Rutherford 1982). The quality of the river was seen as unsatisfactory and a Technical Committee established in December 1979 adopted as a target receiving water standard a maximum daily and night time $BOD_5$ concentration of 5 $g/m^3$ at a control site located downstream from all three discharges. The committee then had to apportion equitably amongst the three discharges the acceptable BOD massflow, taking into account the BOD removal known to occur in the river between outfalls.

TABLE 1 The main discharges to the Manawatu River near Palmerston North

| | Volume $m^3$/day | Raw BOD kg/day | Treated kg/day |
|---|---|---|---|
| Palmerston North Sewage | 16,000 | 6,700 | 4,500 |
| Manawatu Dairy Company | 1,610 | 12,600 | 12,600 |
| Borthwicks/CWS Freezing Works | 5,000 | 10,000 | 6,500 |
| Total (including other minor contributions) | 23,000 | 30,000 | 23,700 |

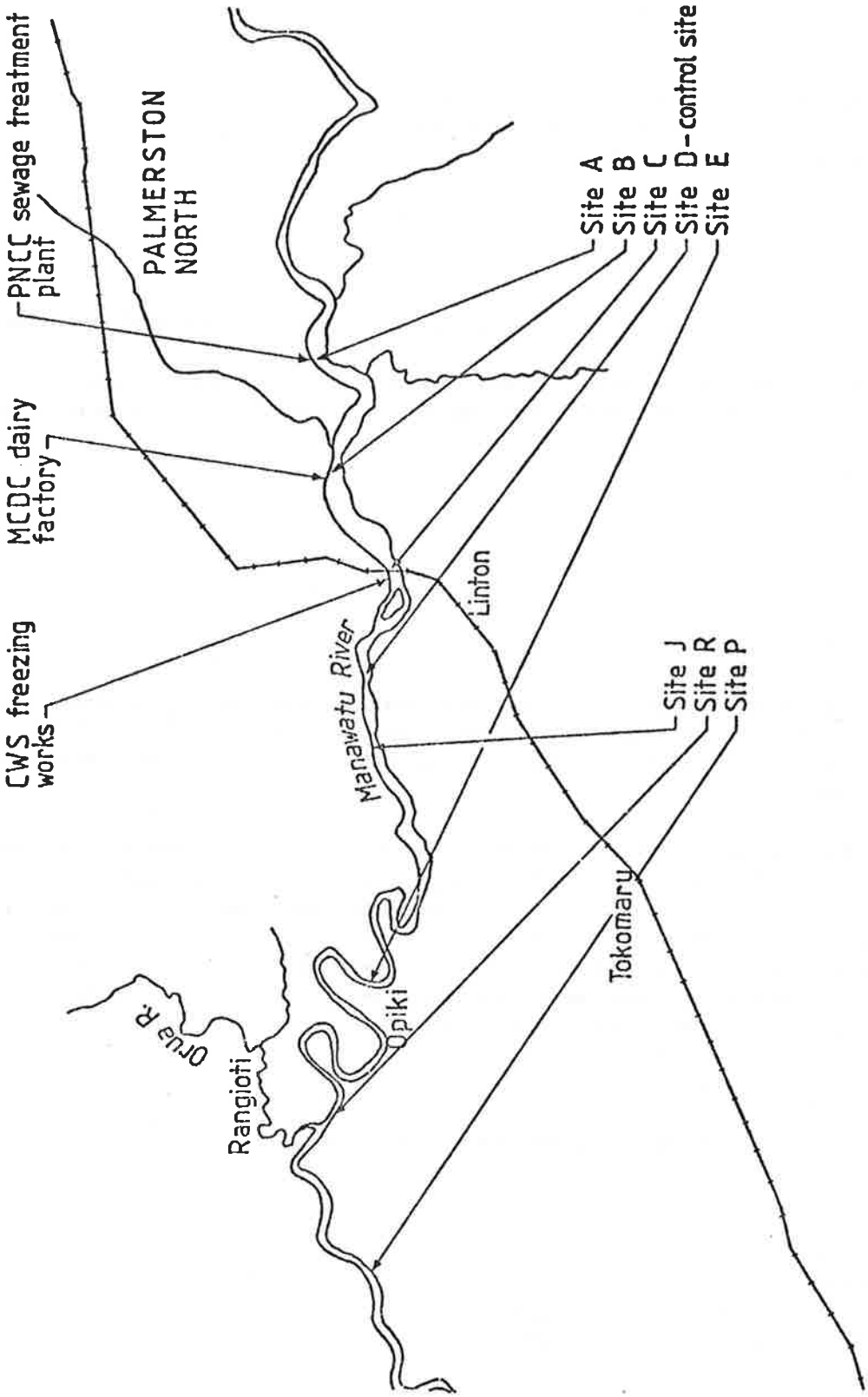



Figure 1 Manawatu Study : Location Map

In a preliminary study of the river Currie (1977) measured river BOD concentrations, BOD inflows and time of travel. BOD inflows were highly variable, resulting in variable river BOD concentrations, and it was impossible to obtain consistent estimates of BOD removal rates. Subsequent examination of these data suggested that better estimates of BOD removal rate might have been obtained if it had been possible to take longitudinal dispersion into account. No data were available to do this, however, and it was necessary to conduct another more detailed survey.

The outfalls are separated by distances comparable with the length of the advective zone (estimated from equation (2) as 5 km) and it was appreciated at the outset that equation (1) would provide at best an approximate description of pollutant dispersion between outfalls.

## METHODS

5.4 kg of Rhodamine WT dye was released into the sewage treatment plant effluent at 0700 on 22 March 1979 and was fully mixed in the river some 15 minutes later. Each of the three discharges was sampled regularly to determine BOD inflow rate. River flow was low and fairly steady at about 26.3 $m^3/s$. The river water was sampled at five sites.

Site A - Immediately upstream of the first outfall (PNCC sewage treatment plant).
Site B - Approximately 2.7 km downstream of the PNCC sewage outfall and immediately upstream of the Manawatu Cooperative Dairy Company outfall.
Site C - Approximately 2.3 km downstream of the Manawatu Cooperative Dairy Company outfall and immediately upstream of the Borthwicks/CWS Freezing Works outfall.
Site D - Approximately 1.4 km downstream of the Borthwicks/CWS outfall. This site corresponded to the "control site" as determined by the Lower Manawatu Technical Committee.
Site E - Approximately 9.3 km below site D.

At sites A and E samples were taken from as near to mid-river flow as possible. At sites, B, C and D all sampling was done from a dinghy attached to a No 8 wire tagline tensioned between anchors on each side of the river. Each sample collected for dye and BOD analysis was a composite of samples taken at the mid-points of 5 sections of equal discharge across the river cross-section. The sections of equal discharge were determined by gauging the river prior to sampling.

Samples were collected at either 15, 10 or 5 minute intervals following a pre-determined schedule based on estimates of when dye would be present at each site.

Samples were analysed for dye concentration using a Turner model III fluorometer on discrete sample mode, and for $BOD_5$ using standard incubation at $20^oC$.

Dye concentration versus time profiles at each river site were used to estimate velocity, U, and dispersion coefficient, D. Values were selected which gave the best fit between the observed profile at each site and the profile predicted from the profile observed at the site next upstream using equation (3) (with k set to zero for the dye).

BOD removal rate coefficients, k, were estimated initially for each river reach from

$$k = \frac{x_{j+1} - x_j}{U} \ln \left( \frac{B_j + M_j/Q}{B_{j+1}} \right) \qquad (4)$$

where $x_j$, $x_{j+1}$ = location of outfall and next downstream sampling site, $B_j$, $B_{j+1}$ = average river BOD just upstream of outfall and at downstream sampling site, $M_j$ = average mass flow rate at the outfall, and Q = river flow rate.

These values of U, D and k were then used in equation (3) to predict BOD versus time profiles at each sampling site. Values of D and k were varied to investigate the sensitivity of predicted BOD profiles.

## RESULTS AND CONCLUSIONS

Figure 2 shows BOD observed concentrations at each river sampling site and estimated* increments below each discharge. It is clear that a substantial increase in river BOD concentration occurs as a result of the discharges and that the inflow rates and river concentration are highly variable.

Table 2 summarises the velocity estimated from the time of travel of the dye peak. The estimates of velocity between sites A and B assuming the dye was discharged at 0715 was much lower (0.30 m/s) than between sites further downstream (0.50 m/s). This may in part be due to the fact dye was placed into the sewage flow and took an appreciable time to reach the river and mix transversely. By assuming a delay of 30 minutes this discrepancy is greatly reduced and so in the ensuing analysis a delay of 30 minutes was assumed to operate at each of the outfalls.

TABLE 2 Derived velocity, dispersion and BOD removal rates

| | Mean Velocity U m/s | Dispersion D $m^2/s$ | BOD Removal k day |
|---|---|---|---|
| A-B | 0.48+ | 50+ | 3.6 |
| B-C | 0.49 | 34 | 10.5 |
| C-D | 0.50 | 25 | 4.1 |
| D-E | 0.51 | 57.5 | - |

+ taking discharge time as 0745 i.e. 30 minute delay to allow for transverse mixing

Figure 3 compares observed and predicted dye profiles. A satisfactory match was obtained near the peak of each profile but there were consistent discrepancies in the rising and falling limbs of dye profiles. This mismatch has also been noted in the Waikato River (Rutherford *et al.* 1980) and the Hokio Stream (Manawatu Catchment Board, unpub.).

Figures 4 and 5 compare observed and predicted river BOD concentrations made using values of k and D selected from the range of anticipated values and the U values given in Table 2. The value of k greatly influences the BOD concentrations. The value of D has only a fairly small influence. This is surprising in view of the finding mentioned earlier that reliable k estimates could not be obtained when longitudinal dispersion was neglected. A possible explanation is that in the early study the BOD data were not sufficiently detailed and/or the time of travel data were not accurate.

Having derived values of U and k, a steady state version of equation (1) (in which longitudinal dispersion was neglected and daily average BOD inflow rates used) was employed to calculate the maximum BOD inflow rate from each outfall such that the BOD receiving water standard adopted for the control site would not be breached. A recommendation was then made to the Manawatu Regional Water Board that they adopt these values as maximum permissible effluent standards (Currie and Rutherford 1982).

* increment taken as observed BOD massflow divided by river flow.

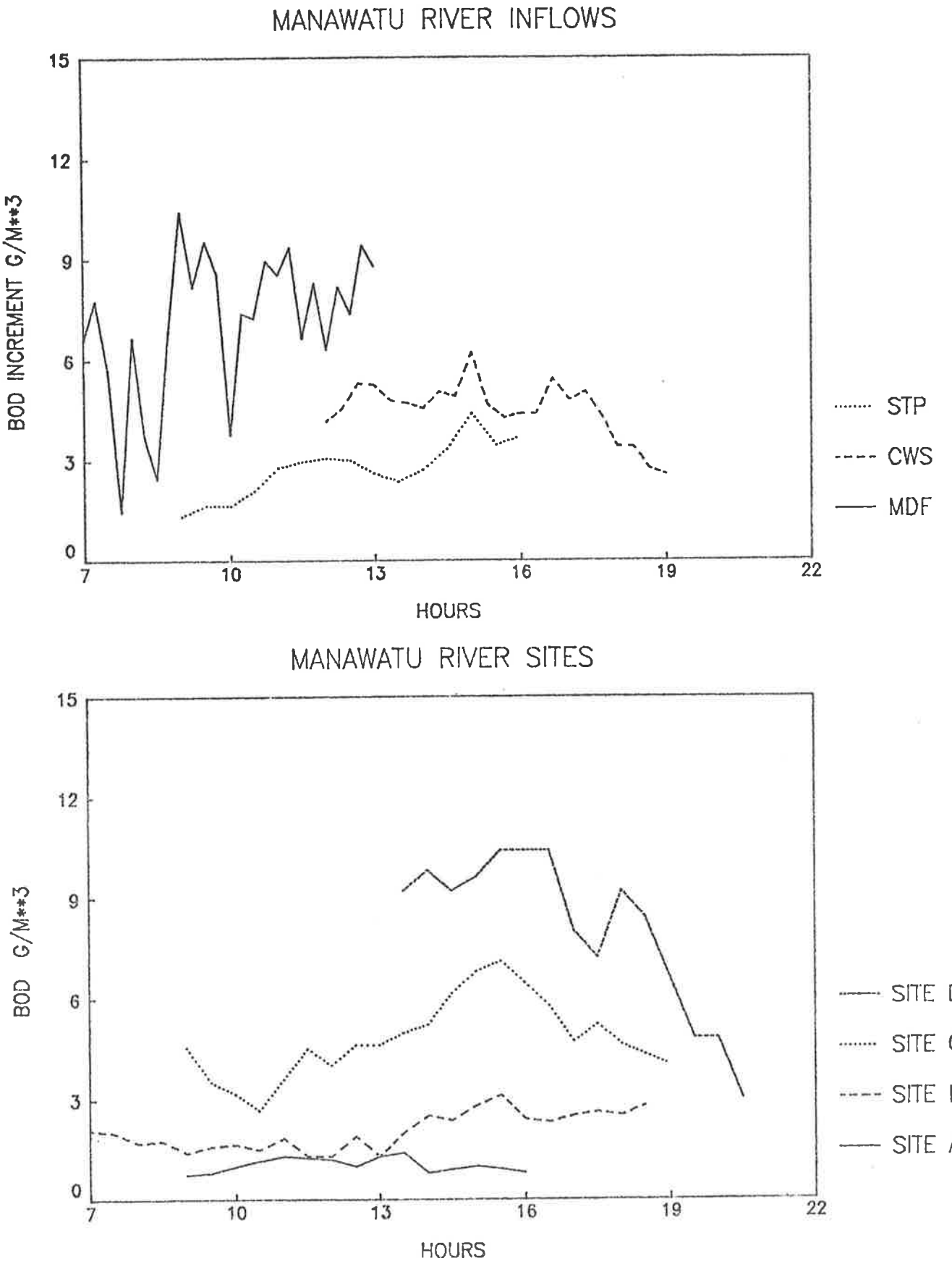



Figure 2 Manawatu Study : BOD inflows and river concentration

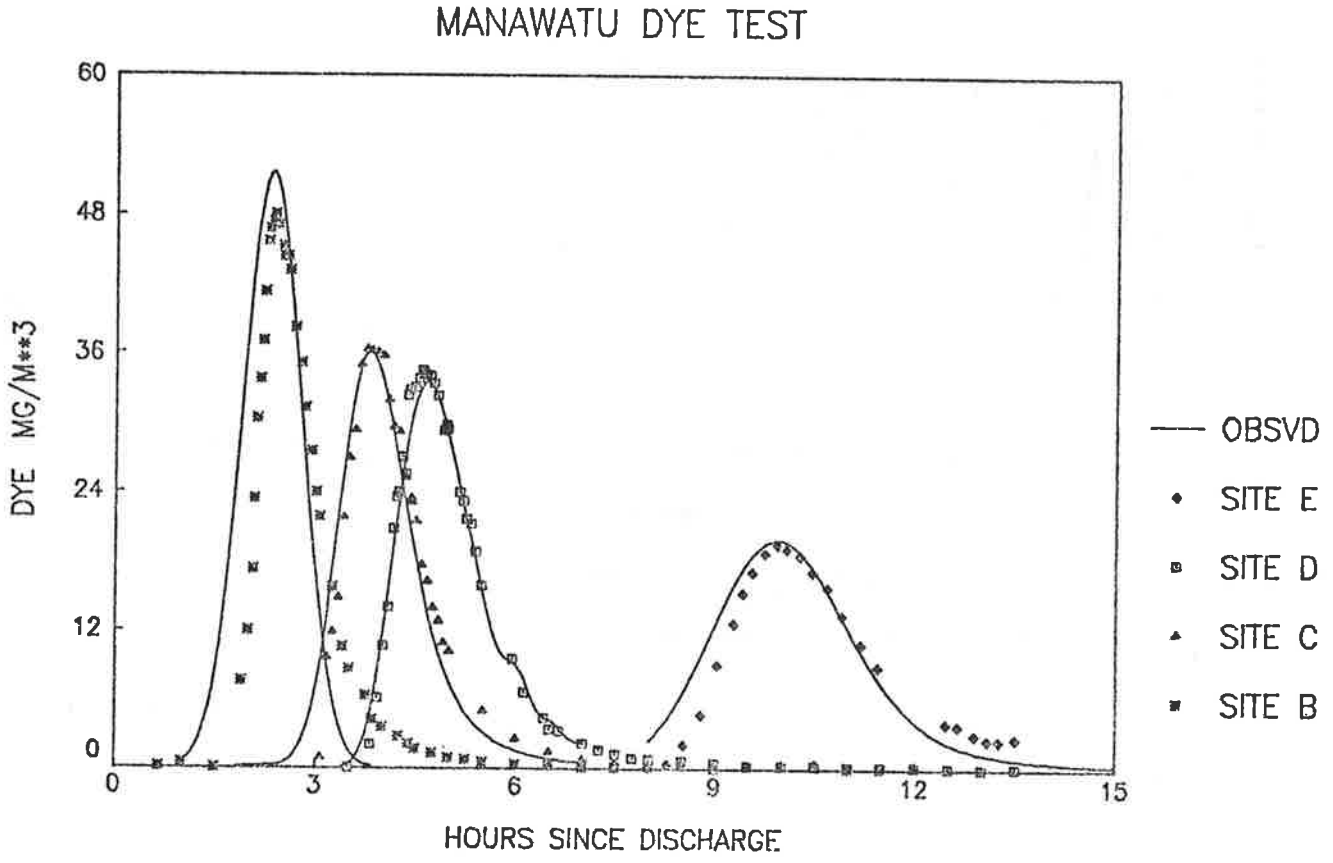
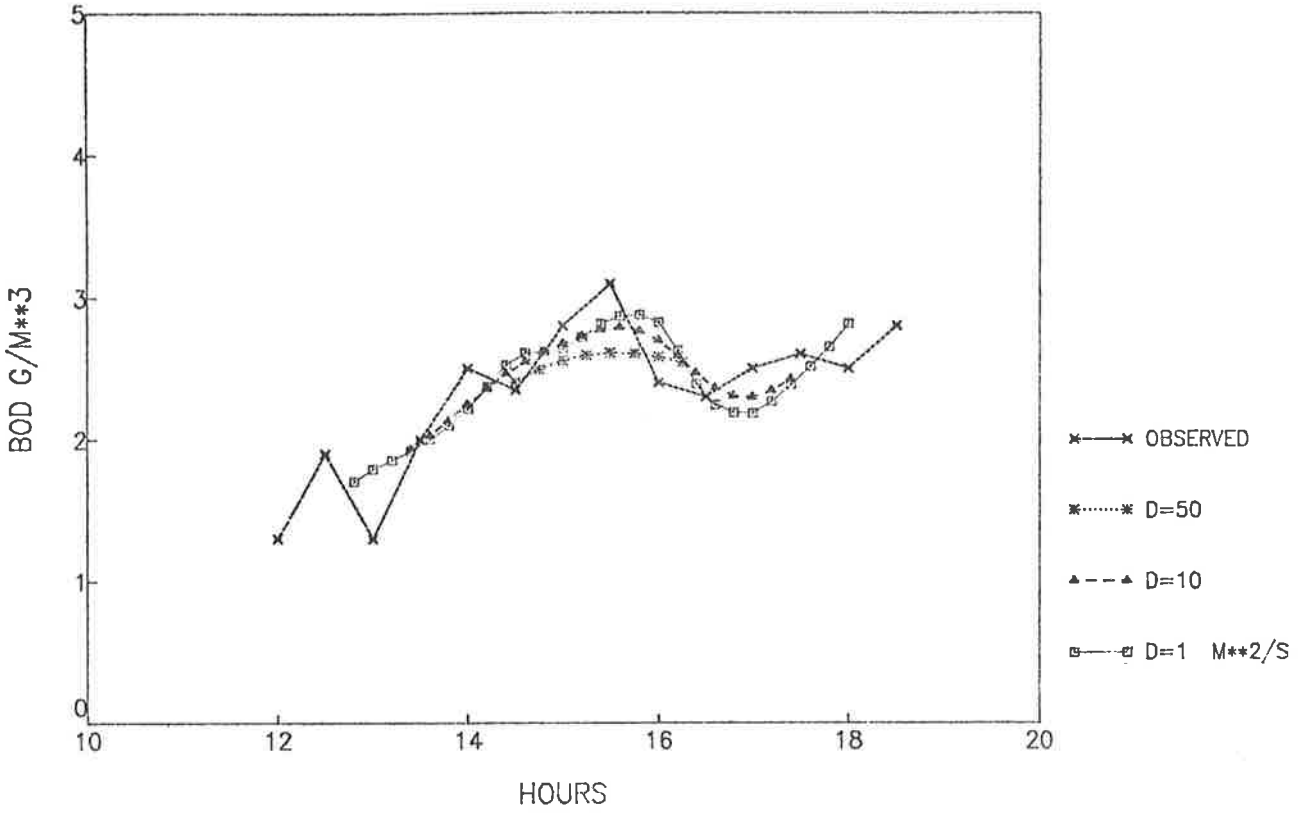



Figure 3 Manawatu Study : Observed and predicted due profiles



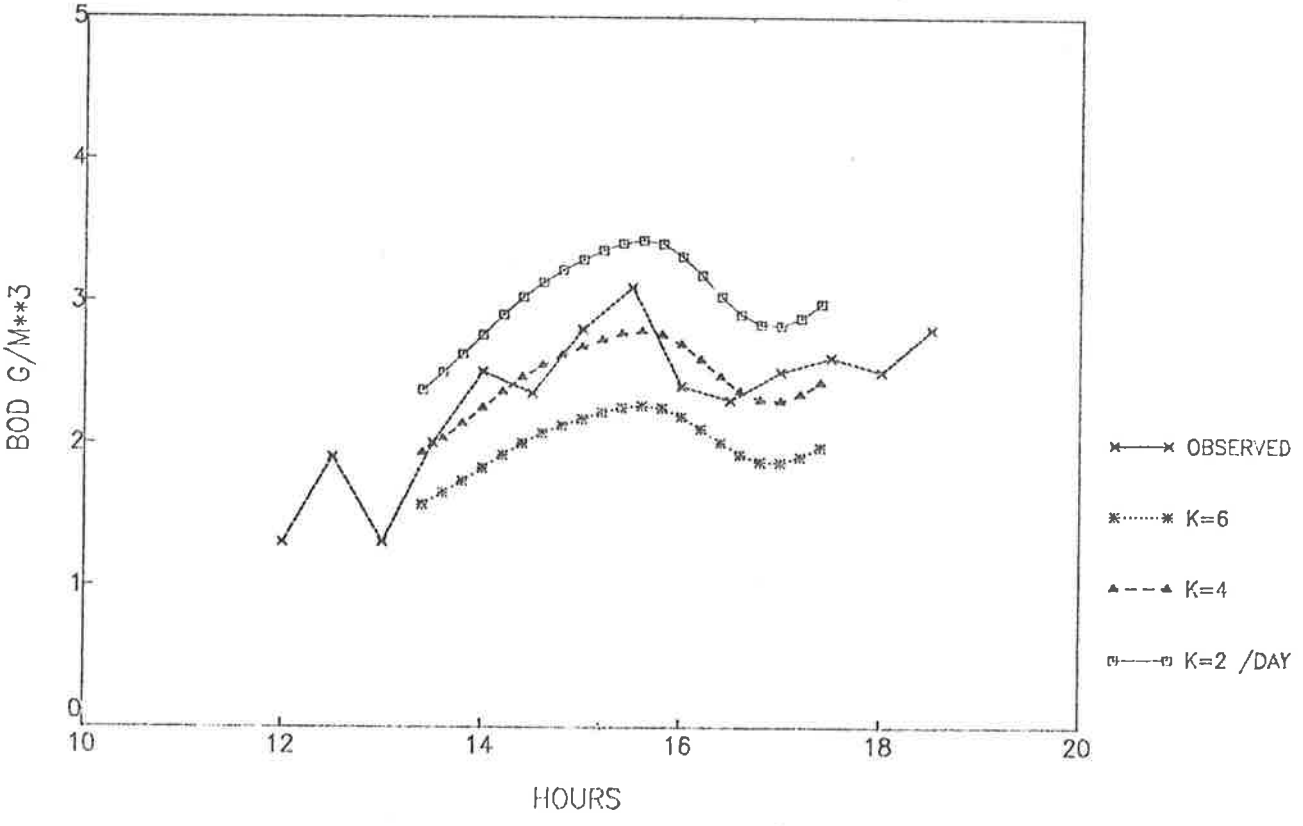
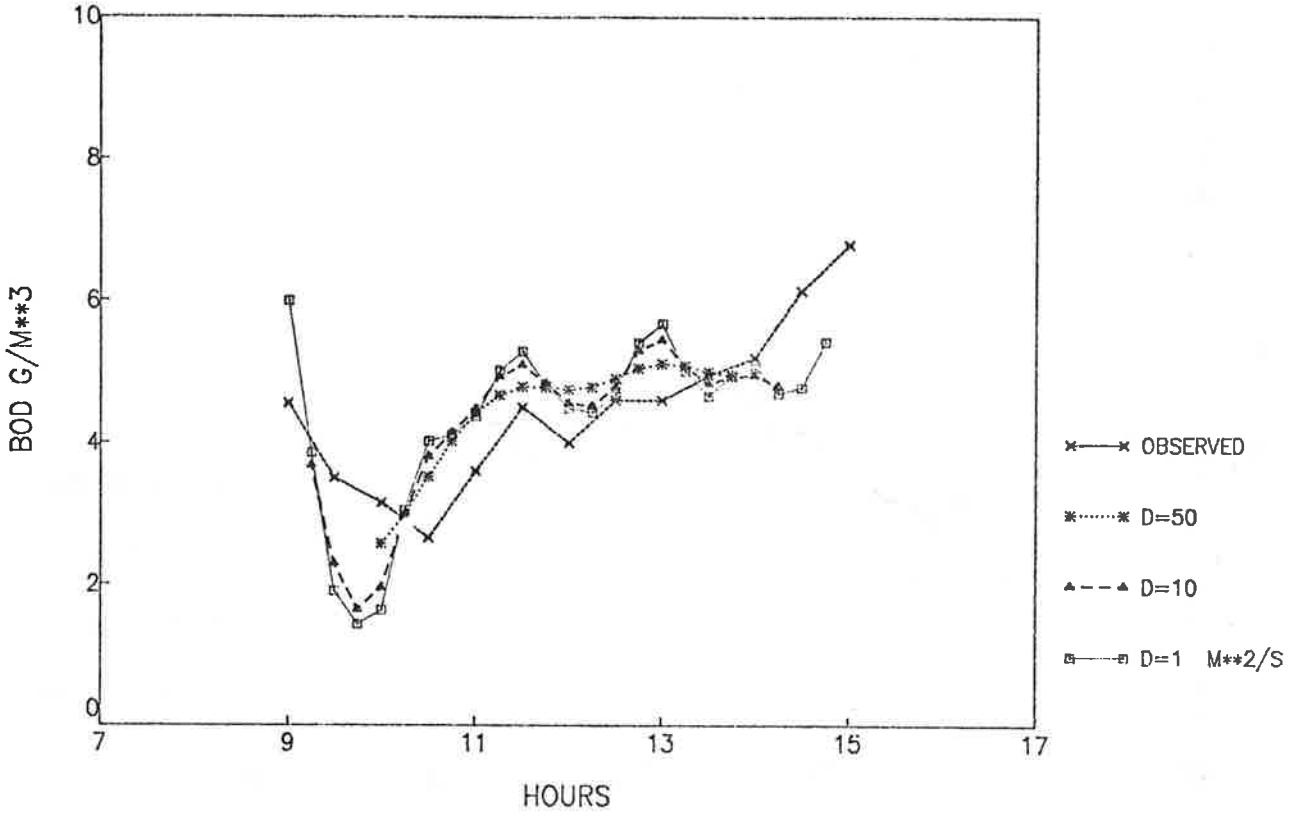




Figure 4 Manawatu Study : Observed and predicted BOD, Reach one STP-B



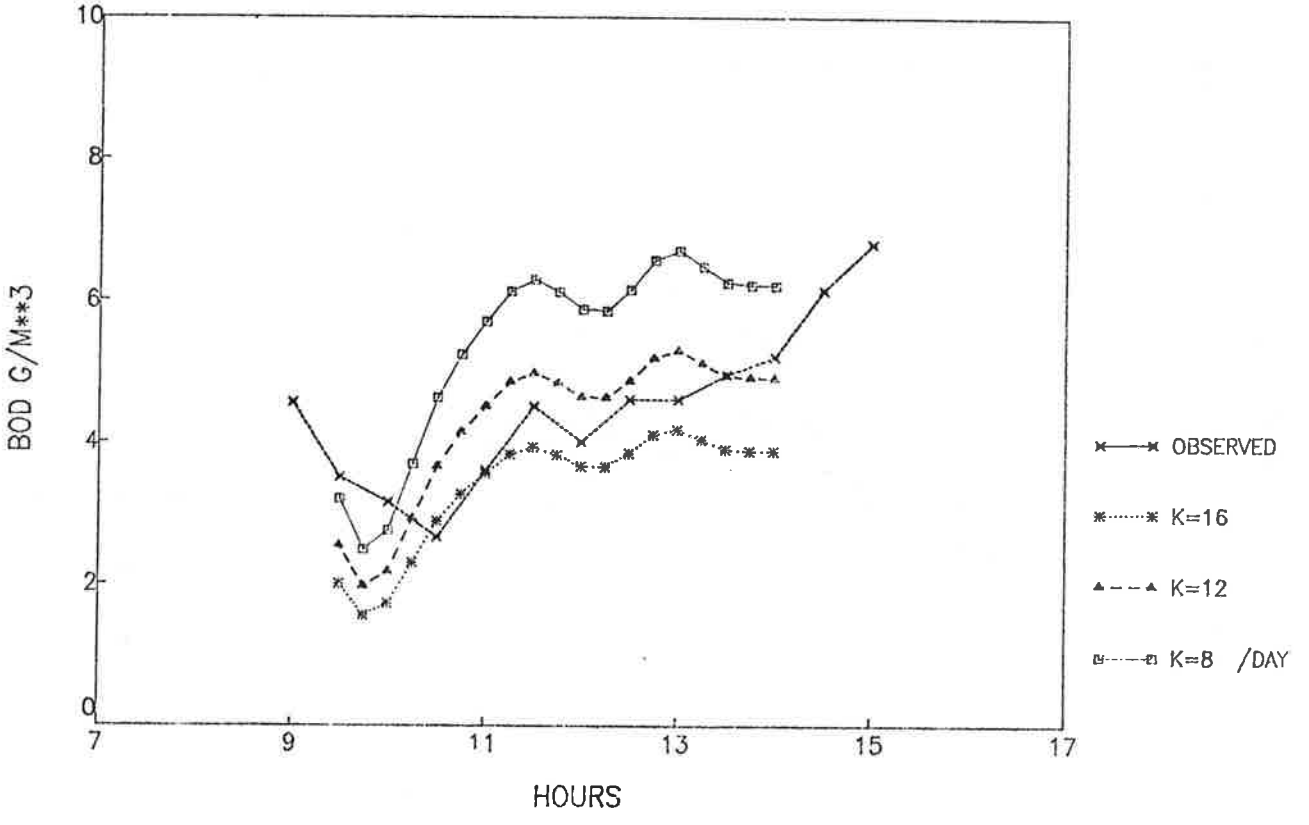




Figure 5 Manawatu Study : Observed and predicted BOD, Reach two MDF-C

## WAIKATO RIVER DISSOLVED OXYGEN

### REVIEW

Details of this study are given elsewhere (Rutherford 1977; Strachan 1979) and in this paper the main conclusions are simply listed insofar as they relate to the importance of longitudinal dispersion.

DO was found to be controlled to a very large extent by the metabolism of aquatic plants, notably phytoplankton. During summer low flows effluent discharged to the river resulted in BOD concentrations which seldom exceeded 3 $g/m^3$ and which combined to produce a DO sag averaging 1 - 2 $g/m^3$. Diurnal variations in effluent loading caused some local variation in river BOD near outfalls but had very little effect on the overall DO sag. Consequently longitudinal dispersion was of comparatively minor significance when modelling DO, and could be neglected.

## WAIKATO RIVER POLLUTANT FLUSHING

### INTRODUCTION

In the mid 1970's consultants to the Auckland Regional Authority were faced with designing off-river storage for the proposed drinking water abstraction works at Mercer. They wanted estimates of the time taken by a pollutant injected as a slug (e.g., following an accident) to travel from various points of injection to proposed drinking water abstraction sites and the length of time for which concentrations at the abstraction site would exceed a specified limit.

The one-dimensional Fickian model, equation (1), was considered adequate for making such estimates but reliable data on velocity and dispersion coefficient were needed.

### METHODS

At this time several members of the Waikato River Technical Committee expressed a desire to obtain data on time of travel and mixing rates and so a joint exercise was planned involving about fifty people from several agencies.

Prior to the test being conducted, estimates of the longitudinal dispersion coefficient were made from data published in the literature and from velocity measurements made in the test reach. The range of values obtained for D was 50 - 350 $m^2/s$.

Figure 7a-b show the simulated dye concentration profiles at each of six proposed sampling sites in the test reach for these two D values. It can be seen that the value of D greatly influences the peak concentration, the time the dye concentration first reaches a specified limit (say 5 $mg/m^3$), and the time for which dye concentration exceeds that limit.

36 kg of dye (Lissamine Red 4B) was released from the Fairfield Bridge, Hamilton, in March 1975 and tracked over a reach extending some 25 km downstream (see Fig. 6). Cross-section average dye concentration versus time profiles were obtained at six sites and were analysed, as described above for the Manawatu dye data, to derive estimates of velocity, U, and dispersion coefficient, D.

### RESULTS

Figure 7c shows the observed dye concentrations. Higher peak concentrations were observed than had been predicted : implying that the rate of longitudinal dispersion was lower than had been anticipated.

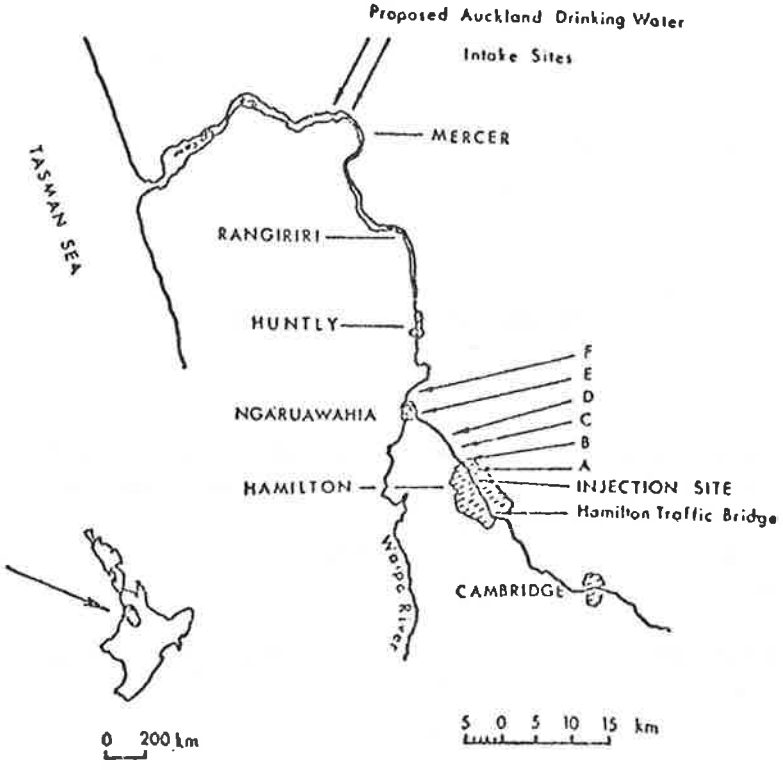



Figure 6 Waikato Dye Test : Location Map

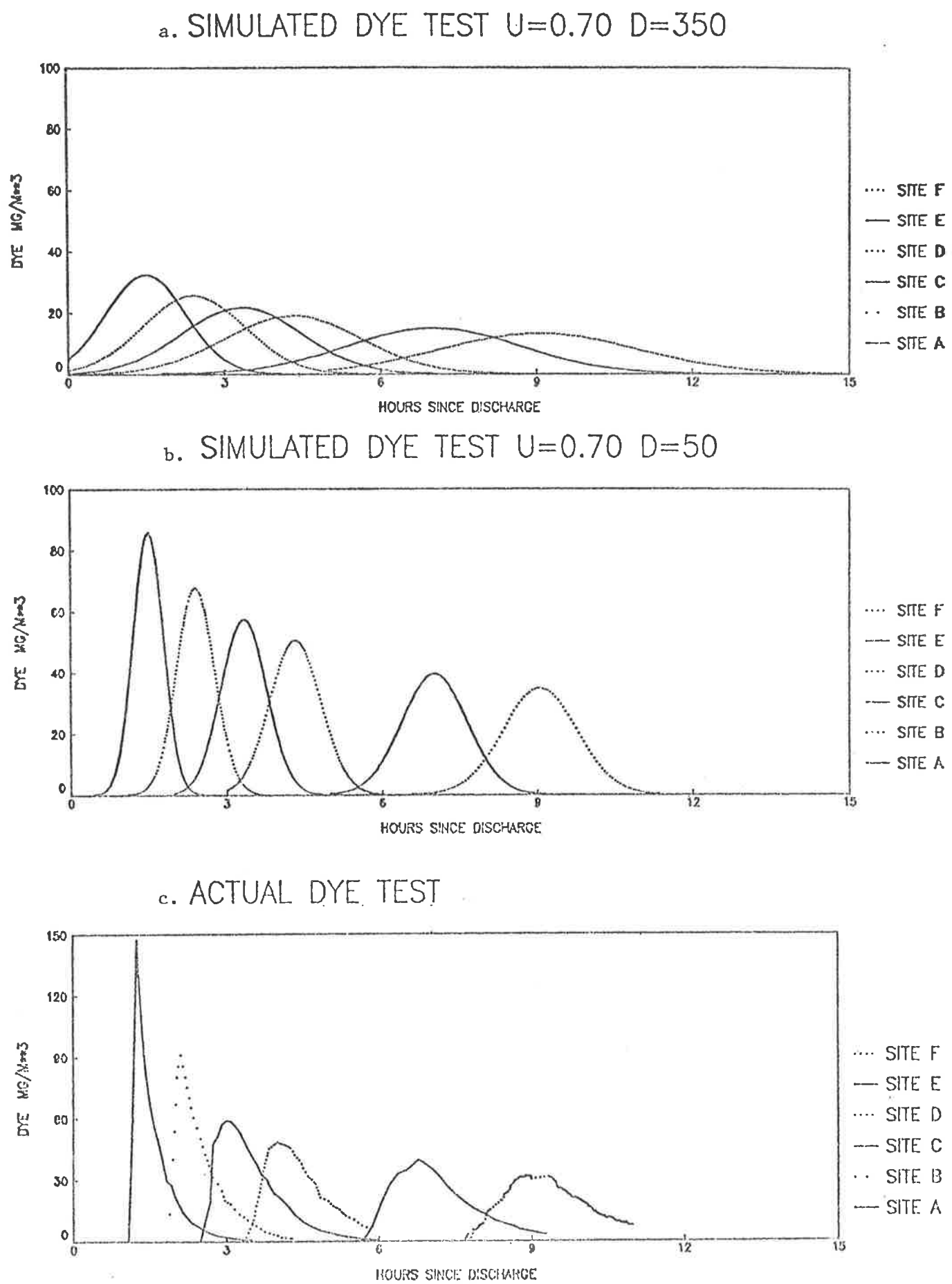



Figure 7 Waikato dye test : Simulated and observed

Figure 8 compares observed dye profiles with the best fit predictions. A reasonable fit was obtained for concentrations near the peak, but there was a persistent mismatch between observed and predicted concentrations in the rising and falling limbs. Tests indicated that this was not an artifact of the "frozen cloud" approximation, but reflected the inability of the Fickian dispersion model to predict the behaviour of tracer slugs at low concentrations.

In order to derive the design data required by the consultants, a hybrid approach was developed. Since it gave a satisfactory fit of high concentrations, the Fickian dispersion model was used to predict the size and timing of peak and half-peak concentrations. Then an empirical similarity form obtained from the dye test data as suggested by Day and Wood (1976) was used to reconstruct the entire profile. Details are given in Rutherford *et al.* (1980).

## CONCLUSION

For the problems of modelling the flushing of slug loads of pollutant along a river channel, accurate simulation of longitudinal dispersion is important. The existing Fickian model can be used to make reasonable estimates of pollutant concentrations near the peak of profiles provided reliable estimates of U and D are available.

The Fickian model is not very accurate for low concentrations in the leading and trailing limbs of profiles. However, improved models for longitudinal dispersion are being developed (see Valentine *ibid*) which may enable reliable predictions to be made of concentration in the rising and falling limbs of tracer profiles.

# WAIPA RIVER MILK SPILLS

## INTRODUCTION

On 2 March 1979 industrial action commenced at six factories of the NZ Cooperative Dairy Company (NZCDC) situated in the Waikato and Hauraki Plains district. Over a five day period some 2 million litres of milk was discharged into the Mangapiko Stream from the NZCDC factory at Te Awamutu whence it flowed into the Waipa River (see Fig. 9). High BOD concentrations and severe deoxygenation resulted, and there was a fish kill. The opportunity was taken to study the response of the river to the shock pollutant loadings.

## METHODS

Three longitudinal surveys of the Waipa River were made from boats.

Run 1 1947 - 2355 on 7.3.79
Run 2 0620 - 1026 on 8.3.79
Run 3 1730 - 2135 on 8.3.79

Measurement of river DO were made *in situ* using YSI meters and samples were taken for subsequent laboratory analysis of $BOD_5$.

A mathematical model for BOD and DO was developed. The model was based on one-dimensional transport equations like equation (1) to which were added source/sink terms to account for BOD exertion, physical reaeration and tributary inflows. BOD exertion was assumed to stop when DO dropped to zero (Gundelach and Castillo 1976) and a semi-empirical equation was used to estimate the reaeration rate (O'Connor and Dobbins 1958).

A novel numerical method was developed to solve the model equations (McBride in prep.), which has proved to be very accurate for predicting the movement of steep concentration fronts along non-uniform river channels under steady flow.

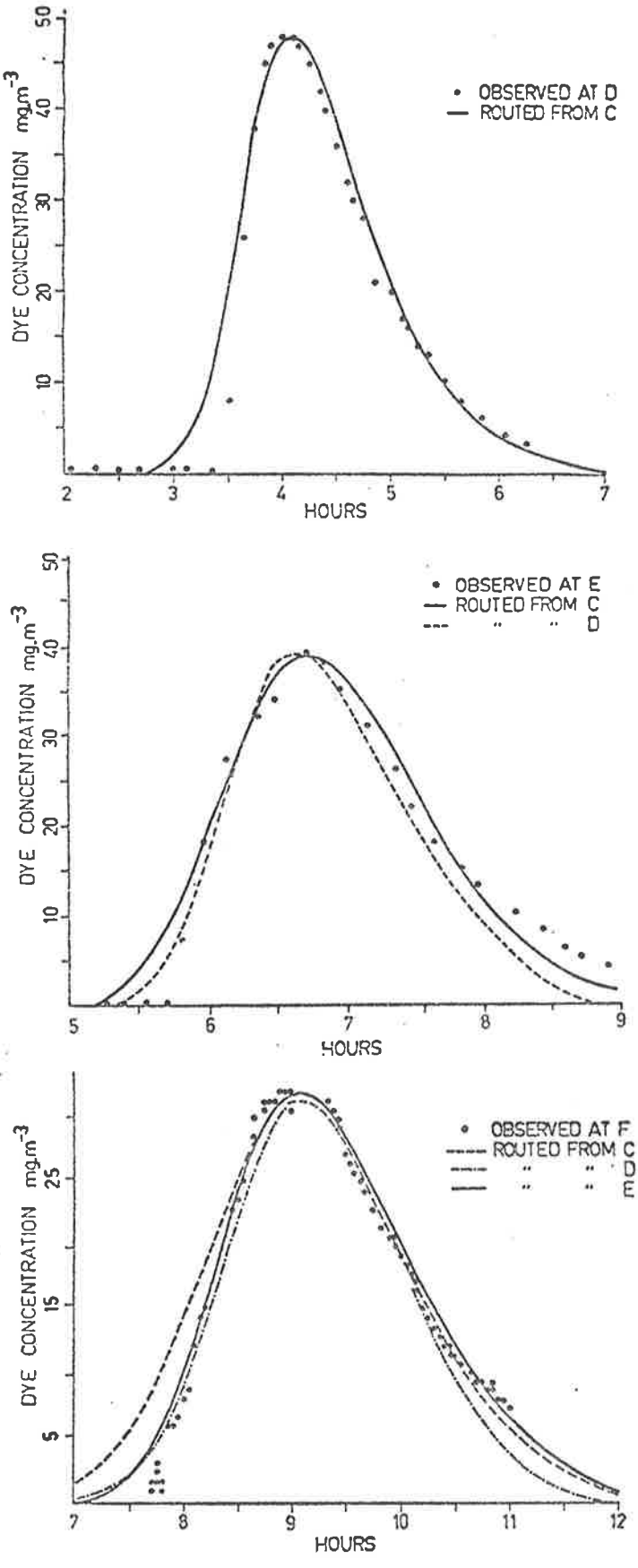



Figure 8 Waikato Dye Test : Observed and predicted dye profiles

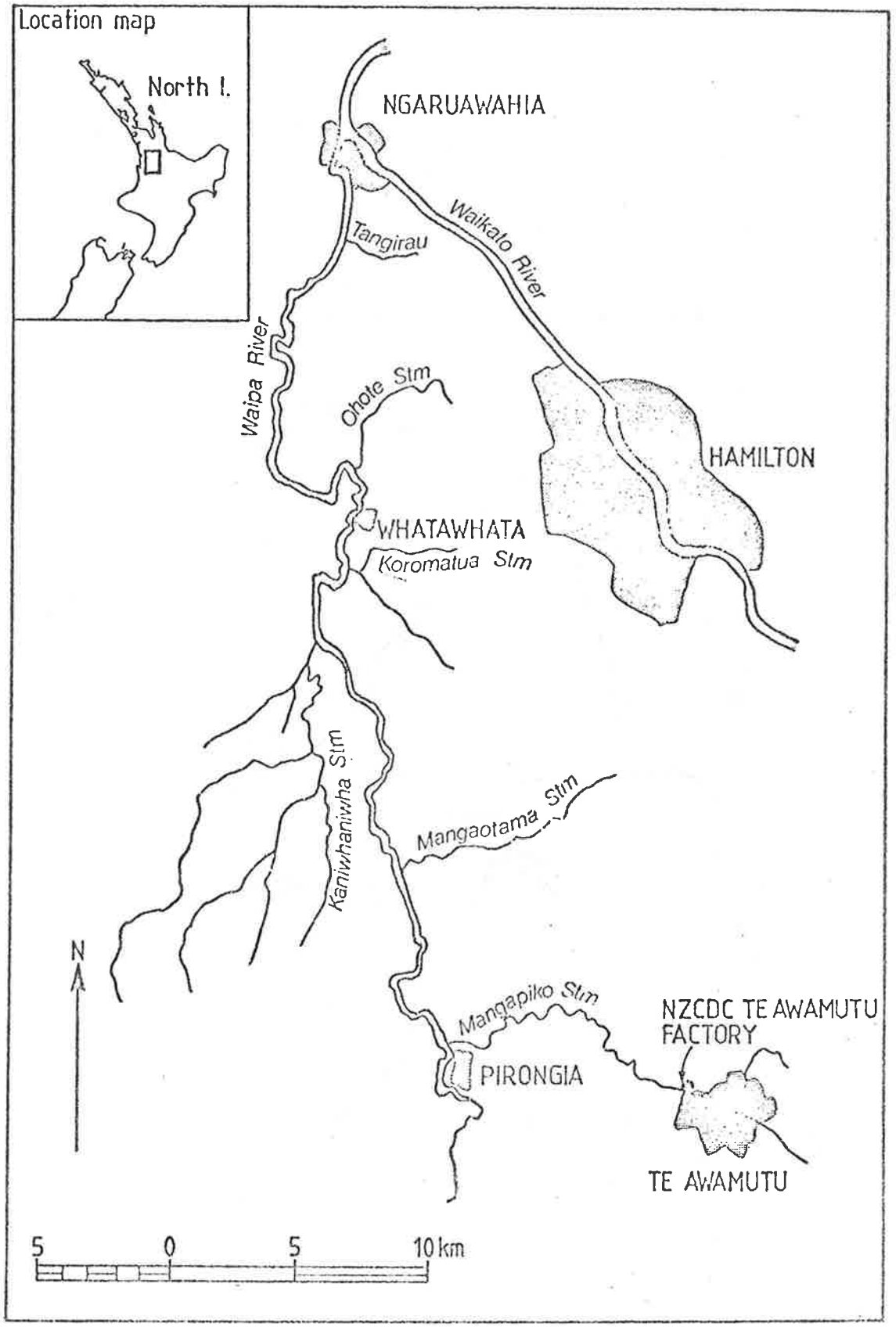



Figure 9 Waipa Study : Location Map

Further details of methods are given by McBride and Rutherford (1982).

## RESULTS

Mean velocity was estimated from the times of passage of peaks and troughts of BOD and DO. Velocity decreased as the river approached Ngaruawahia and could be described fairly well by

$$U = \begin{cases} .0005x + 0.1 \text{ for } 0 < x < 20 \\ .0105x - 0.1 \text{ for } 20 < x < 60 \end{cases}$$

where x = km above Ngaruawahia and U = velocity in $m.s^{-1}$.

Two patches of anoxic water containing high BOD concentrations were found in the river. Each was about 20 km long and were separated by about 10 km of water containing low BOD and fairly high DO concentrations (see Fig. 10). This reflected the known intermittent nature of the milk discharge.

Figure 11 compares BOD and DO observed during Run 3 with predictions made, using Run 2 data as the initial condition, for various values of longitudinal dispersion coefficient and BOD decay rate.

Values of D in the range 0-20 $m^2.s^{-1}$ would be expected in the Waipa River. Model results show that increasing the value of D attenuates the peaks in BOD concentration near km 16, km 24 and km 30. One might also have expected increasing D to decrease the slope of the BOD front near km 4. It does so only slightly for two reasons. Firstly, the velocity decreases markedly as you approach Ngaruawahia and this acts to "pile up" pollutant and sharpen concentration gradients.

Secondly, the absence of data near the downstream end of Run 2 and Run 3 prevents an accurate assessment of the concentration gradient and the boundary condition imposed by the numerical scheme may be partially responsible for this behaviour.

Increasing the value of D attenuates peaks in DO concentration near km 10 and km 27. One might have expected that increasing D would have decreased the slope of the DO front between km 30 and km 40. The fact that it does so slightly reflects the influence of reaeration. As DO drops the rate of reaeration increases and counteracts the effect of longitudinal dispersion. The upstream boundary condition may also have lessened the effect of increasing D close to km 40.

Generally these results accord with our expectation that increasing the rate of longitudinal dispersion "smears" BOD and DO concentration peaks.

Values of k between 0.6 and 5 $day^{-1}$ have been observed in NZ rivers (McBride and Rutherford 1982). Simulations show that variations of k between 0.25 and 1.0 $day^{-1}$ markedly influence BOD and DO concentrations between km 25 and km 40. In this region there are fairly high BOD concentrations waiting to exert their oxygen demand and non-zero DO concentrations enable bacterial metabolism to proceed.

By comparison, between km 16 and km 24 there are high BOD concentrations, but DO is zero so no exertion can occur and the influence of varying k is small. Between km 8 and km 14 BOD concentrations are low so there is little to exert and the influence of k on DO is small.

Variations of the reaeration rate also influence DO and BOD predictions markedly (McBride and Rutherford 1982). Overall it can be seen that:

1 Dispersion had no influence where BOD or DO concentration were uniform.

2 Dispersion noticeably smeared peaks and troughs of BOD concentration. Where BOD decay occurred, this augmented the effects of dispersion.

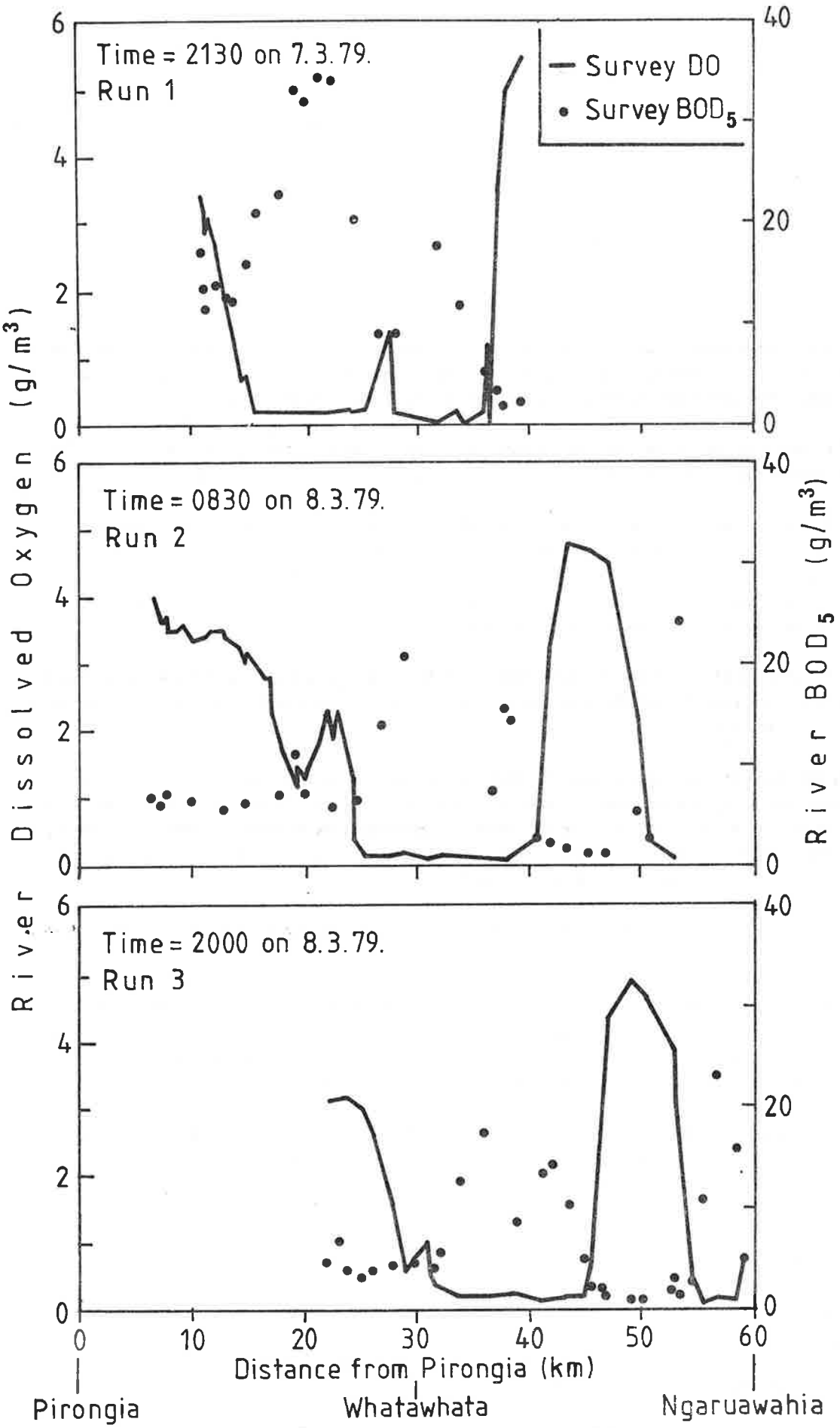



Figure 10 Waipa Study : Observed BOD and DO profiles

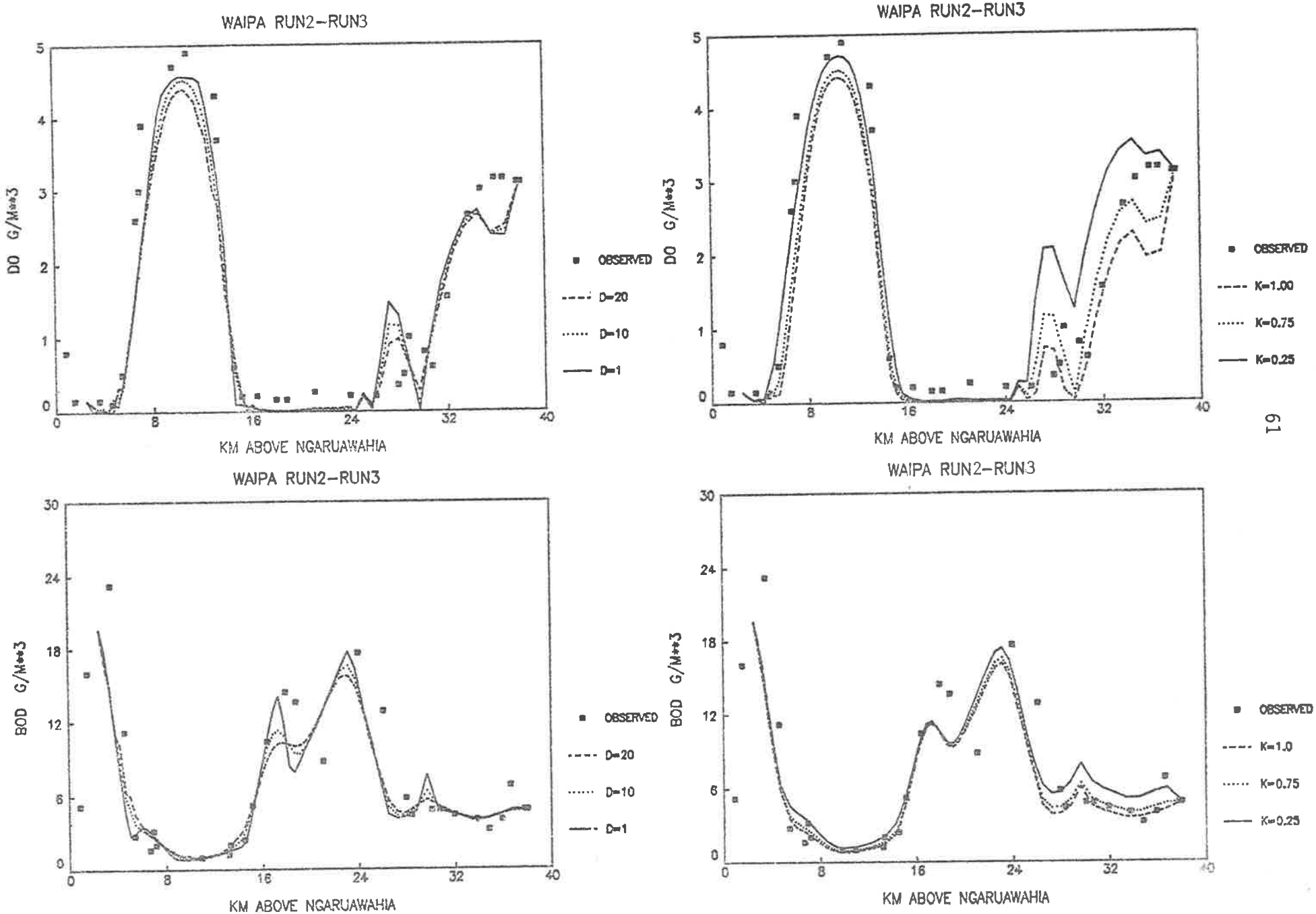



Figure 11 Waipa Study : Observed and predicted BOD and DO

3 Where the velocity decreased markedly, near Ngaruawahia, the tendency for dispersion to "smear" a BOD front was counteracted by the tendency for the decreasing velocity to cause BOD to "pile up".

4 Dispersion also smeared peaks and troughs of DO, and through its influence on BOD, affected DO uptake rates. However, reaeration counteracted the effects of dispersion and generally dispersion influenced DO less than it influenced BOD.

5 BOD decay rate influenced BOD and DO markedly in regions where DO exceeded zero. Overall, predicted BOD and DO profiles were more sensitive to variations of decay rate over the range of anticipated values than to variations of dispersion coefficient.

## CONCLUSIONS

1 Dispersion strongly influences the concentration of conservative pollutants originating from instantaneous point injections. (Dye test results.)

2 In a channel where area increases with distance downstream, dispersion is counteracted by decreasing velocity. (Waipa modelling studies.)

3 Dispersion also influences the concentration of non-conservative pollutants (e.g., BOD) originating from rapidly varying sources, although decay exerts a strong influence which augments and may dominate the effect of dispersion. (Waipa modelling studies.)

4 Dispersion has only a minor influence on the concentration of non-conservative pollutants whose rate of injection varies slowly, while advection and decay exert a strong influence. (Manawatu study.)

5 Dispersion influences DO concentrations below rapidly varying sources of BOD, but generally decay and reaeration exert a stronger influence. (Waipa modelling studies.)

6 Dispersion is neglible when predicting DO concentration in a river where BOD exertion is of minor significance compared with aquatic plant metabolism. (Waikato DO study.)

## REFERENCES


Currie, K.J. 1977: Water quality management report - Manawatu River below Palmerston North. Manawatu Reg. Wat. Bd. unpublished report.

Currie, K.J.; Rutherford, J.C. 1982: Management of BOD in the Lower Manawatu River. In "Aquatic Oxygen Seminar Proceedings, Hamilton, November 1980". Water & Soil Misc. Pub. No. 29, MWD, Wellington.

Day, T.J.; Wood, I.R. 1976: Similarity of mean motion of fluid particles dispersing in a natural channel. Wat. Res. Res. Vol. 12 No 4, 655-66.

Fischer, H.B. 1966: "Longitudinal dispersion in laboratory and natural channels". California Institute of Technology Report No KH-R-12, Pasadena.

Fischer, H.B. 1973: Longitudinal dispersion and turbulent mixing in open-channel flow. Ann. Rev. Fluid Mech. Vol 5, 59-78.

Gundelach, J.M.; Castillo, J.E. 1976: Natural stream purification under anaerobic conditions. J. Wat. Poll. Con. Fed. Vol. 48, No 7, 1753-8.

McBride, G.B.; Rutherford, J.C. 1982: Waipa River. In "Aquatic Oxygen Seminar Proceedings, Hamilton, November, 1980". Water & Soil Misc. Pub. No. 29, MWD, Wellington.

McBride, G.B. in prep: A Lagrangian Scheme for River Pollutant Transport.

O'Connor, C.J.; Dobbins, W.E. 1958: Mechanism of reaeration in natural streams. ASCE Trans., Vol 123, 641.

Rutherford, J.C. 1977: Modelling the effects of aquatic plants in rivers. ASCE Vol. 103, No EE4, 575-91.

Rutherford, J.C.; Taylor, M.E.U.; Davies, J.D. 1980: Waikato river pollutant flushing rates. ASCE Vol. 106 No EE6, 1131-50.

Strachan, C (Ed) 1979: Waikato River - A water resources study. Water & Soil Tech. Pub. No. 11, MWD, Wellington.

ACKNOWLEDGEMENTS

The cooperation of the Manawatu Catchment Board, Manawatu River Technical Committee, Waikato River Technical Committee, Kingston Reynolds, Thom and Allardice, Binnie and Partners (London), Waikato Valley Authority, NZ Cooperative Dairy Company and Ministry of Works and Development is acknowledged. Many of the graphs were drawn using a program written by R A Hoare and run on the University of Waikato PDP 11/70 computer.

DISCUSSION

P N McFarlane As you measured dye concentrations and BOD concentrations simultaneously, did you observe any BOD inhibition effects due to the presence of dye?
Gilliland Rhodamine WT was chosen for its documented low toxicity in the aquatic environment. I believe inhibition of BOD to be unlikely at the levels observed at the river sites during the study.

P Tortell Given the low dispersion characteristics of the Waipa River, had the milk discharge been in one slug rather than in three parts, would the impacts have been the same regarding BOD, DO levels and impact on river organisms?
Rutherford Had all the milk been discharged over say 8 hours, I expect we would have seen BOD concentrations about three times higher than observed but of course DO could not have been depleted further than the anoxia which was observed. Hence organisms would have been exposed to anoxic conditions for about a third of the time, i.e., about 8-12 hours, which may still have been long enough to inflict severe damage.

M E U Taylor Would discharging all the milk in one slug have caused problems further downstream?
Rutherford Possibly, although the Waipa River flows into the Waikato River at Ngaruawahia where considerable dilution takes place. If, however, the Waipa slug and the slug from the Te Rapa Dairy Factory (on the Waikato) had coincided, then the Waikato River could well have been stressed more severely than was observed.

J D Boyle At what stage does a Fickian dispersion model cease to be appropriate in a pool and riffle system like the Manawatu? Would a stirred tank/plug flow model be more appropriate? It may be able to explain some of the long tails.
Rutherford This is something I would like to look at but have not yet done. There is a recent publication on this subject and it has some attractions.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre* *November 17-18, 1981*

# LONGITUDINAL DISPERSION AND THE EFFECTS OF DEAD ZONES

*E M Valentine, Central Laboratories, MWD, Lower Hutt*

## ABSTRACT

A model of the longitudinal dispersion process which includes the effects of dead zone trapping is outlined. A set of field experiments which demonstrate these effects is described and conclusions drawn about the nature of the process in rivers.

## INTRODUCTION

It has been well demonstrated that for steady flow in pipelines the one-dimensional Fickian-type diffusion equation derived by Taylor (1921) is valid. That is, in describing the distribution of a solute in terms of the moments of the concentration distribution, a finite quantity of solute tends to become normally distributed with a variance growing at a rate proportional to time.

Thus a Fickian process is characterised by a constant diffusion coefficient, K, that is independent of the concentration and the time.

This equation, of the form $J = - K \frac{\partial c}{\partial x}$ (1)

(where J is the one-dimensional flux across a reference plan perpendicular to x, per unit time, per unit area of a diffusing substance with concentration c) has been applied to natural rivers and in some cases the observations suggest that the theory is not applicable (Nordin and Sabol 1974).

The importance of the non-uniform velocity distribution on the dispersion of a pollutant in natural rivers has long been recognised. More recently, interest has been focused on additional mechanisms in an effort to explain why the one-dimensional solution does not appear to apply to rivers (Valentine and Wood 1977, 1979 a, b; Valentine 1978).

### The dead zone mechanism

It seems probable that, apart from the non-uniformity of flow cross-section and velocity distribution dealt with by Fischer (1966), the most important mechanism which is present in rivers and not in pipes is the trapping and release of parcels of fluid by peripheral dead zones. Until a few years ago this mechanism received scant attention.

The boundary roughness of a stream consists of elements which cause the production of trapped, circulating eddies. It has been demonstrated in numerical and experimental studies that this dead zone trapping of an instantaneously injected, neutrally buoyant material, greatly increased the dispersion coefficient and delayed the period where the variance increases linearly with time (i.e., where equation (1) applies).

### Dispersion equation

The Eulerian dispersion equation for turbulent flow can be written for three dimensions.

$$\frac{\partial c}{\partial t} + U\frac{\partial c}{\partial x} = \epsilon_x \frac{\partial^2 c}{\partial x^2} + \frac{\partial}{\partial y}\left(\epsilon_y \frac{\partial c}{\partial y}\right) + \frac{\partial}{\partial z}\left(\epsilon_z \frac{\partial c}{\partial z}\right) \quad (2)$$

where $c(x,y,z)$ = concentration of a dispersant at a point,
$t$ = dispersion time,

$x,y,z$, = distance coordinates in longitudinal, vertical and lateral directions,
and $\varepsilon_x, \varepsilon_y, \varepsilon_z$, = local eddy diffusivities in x, y and z directions.

Equation (2) describes dispersion in the flow zone. An equation which describes concentration in the idealised dead zones shown in Fig. 1 is

$$\frac{w}{t} = \frac{KU}{d}(c-w) \qquad (3)$$

where w = concentration of dispersant in eddy traps,
K = eddy entrainment coefficient, evaluated from experiment,
U = cross-sectional (mainflow) average of velocity,
and d = depth of dead zone.

That is, the mass transfer across the main flow/dead zone interface is assumed to be proportional to the concentration difference across it, and to the mean velocity in the flow zone.

These two simultaneous partial differential equations may be solved numerically for up to three-dimensions by considering the moments of the concentration distribution. This is useful, as it is in this way that experimental results can also be expressed. However, the numerical results must be expressed in the spatial domain (i.e., as a function of distance x) and experimental results are generally expressed in the temporal domain (as a function of time t).

## EXPERIMENTS

The experiments which are described were designed to test the model for a case where trapping was important, but without the usual complications of sinuosity. A 4000 m reach of a water race on the Canterbury Plains was used. This channel is straight, has a generally uniform slope, a rectangular cross-section, and constant flow. The cross-section is small, therefore a great dimensionless distance is represented. The bed materials consist of the armouring fraction of the underlying alluvial deposits and the side boundaries of weed-covered soil.

The experiments basically consisted of introducing a slug of salt tracer to create a sudden plane source, and monitoring the conductivity levels produced in the flow at chosen sites along the channel.

Two series of tests were performed; the first set where the channel was in an overgrown state (the 'rough' series), and the second set following the clearing of the weed covered channel sides (the 'cleared' series). That is, one set had a relatively high dead zone volume and the other a very much reduced trapping potential.

In modelling the flow cross-section an evaluation of the flow section variables must be made. This is straightforward with the flow zone and idealised dead zones but more difficult for the irregular dead zones of a natural stream. However, measurement of the depths and areas of the dead zones in a representative sample of the boundary roughness has permitted a reasonable approximation of dead zone volume.

## RESULTS

The comparison of numerical and experimental results is accomplished by an approximation called the 'frozen cloud' which allows both sets of results to be expressed in the temporal domain. The approximation does not create too many difficulties in comparing the effects of the different dead zone volumes upon the dispersion process. The important statistical parameters which describe the concentration distribution are now presented.

Displacement and velocity

Figure 2 shows graphs of displacement of the tracer cloud mean from the injection point as a function of time for the 'rough' and 'cleared' conditions. Both sets of experimental data indicate that the mean velocity of the cloud, $U_c$, reduces from an initial maximum to a constant minimum velocity as predicted by the model. Despite the scatter, due primarily to small discharge differences between occasions when tests were performed, it is apparent that for the greater roughness there is a slower final velocity.

Variance

Graphs of temporal variance, $\sigma_t^2$, versus the mean travel time are shown in Fig. 3.

There is a remarkable difference in the growth of variance between the respective roughness conditions, i.e., the greater the dead zone volume the larger the growth of variance. It would be impossible to account for this difference merely by considering a change of velocity profile. The curves show an initial period when the rate of growth of variance increases with time followed by a region where the growth of variance is linear. In these cases it took 1500 m (rough) and 600 m (cleared) to reach the Taylor period. In view of the difficulty of estimating dead zones, the agreement between experiment and theory must be considered satisfactory.

Decay of peak concentration

As the peak of the concentration distribution is subject to relatively large errors due to slightly differing flow rates, these have been removed by adjusting the data to constant values of the zeroth moment (i.e., $\int_0^\infty c\,dt$).

For the Taylor type of solution to apply, the peak concentration must decay as a function of $t^{-\frac{1}{2}}$ which appears to be true in Fig. 4, for large dispersion times. Taken with the form of the displacement and variance data these data are consistent with the eventual Fickian description of the longitudinal dispersion process.

Skew

This is a sensitive parameter. It is also directly affected by the assessment of the 'tail' of the concentration distribution. Even if the spatial distribution were Gaussian, the temporal distribution would exhibit positive skew due to the continuing dispersion process as it passes the probe. Thus, if the process becomes Fickian, the skew coefficient obtained can be expected to become asymptotic to some positive value. This appears to be satisfied, as shown in Fig. 5.

The decay of skew in these data becomes very slow. Also, where the dead zone volume is greater, the values of skew at early dispersion times are much larger.

CONCLUSIONS

1 The dispersion process with dead zones can eventually be described as behaving in agreement with the Taylor hypothesis.

2 There is a delay, due to the trapping mechanism, in the arrival at this period.

3 There is an enhanced growth of variance. That is, for a given flow, the dispersion coefficient is greatly increased.

4 The mass transport velocity of the pollutant is reduced below the mean flow velocity.

It appears that river channels have generally been too short to reach the diffusive (Taylor) period and

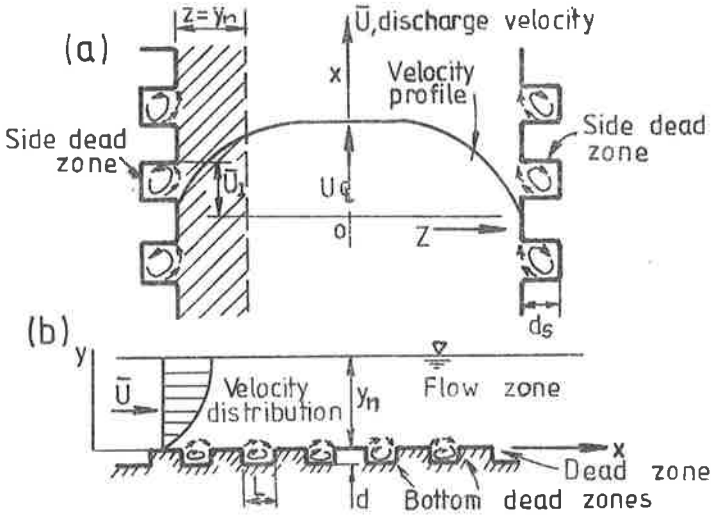



FIG.1 Idealized Three-Dimensional Model:
(a) Plan at Mid-Depth
(b) Longitudinal Section

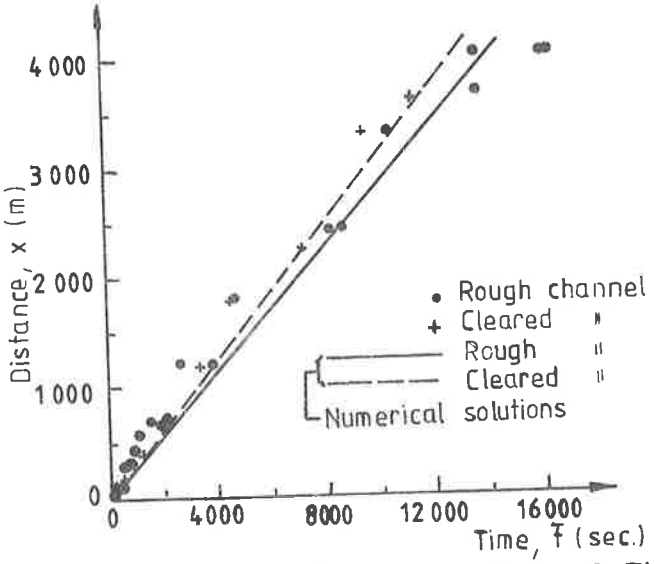



FIG.2 Cloud Displacement as Function of Time for Field Data and Numerical Comparisons

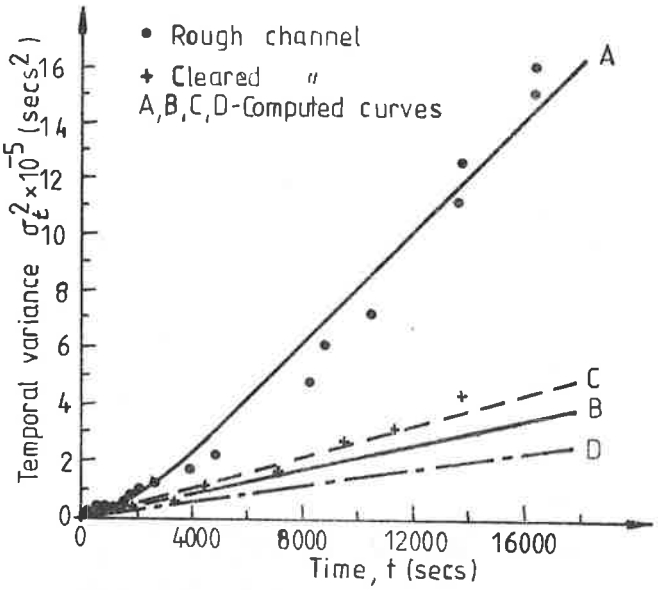



FIG. 3 Cloud Temporal Variance as Function of Time for Field Data and Numerical Comparisons

KEY: A = rough, B = rough, no dead zones
C = clear, dead zones, D = clear, no dead zones

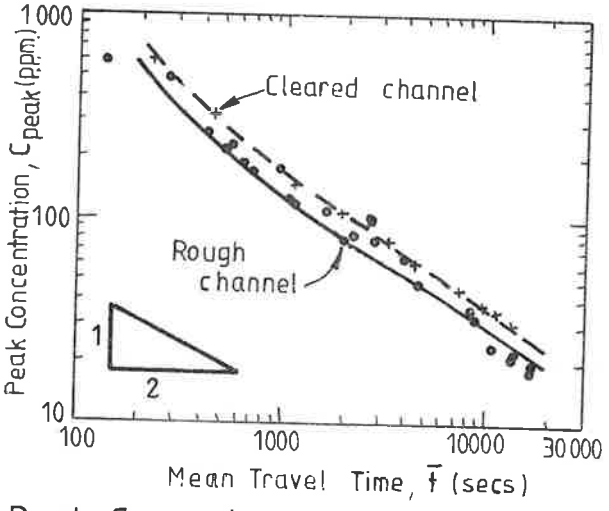



FIG. 4 Peak Concentration as Function of Time for Field Data

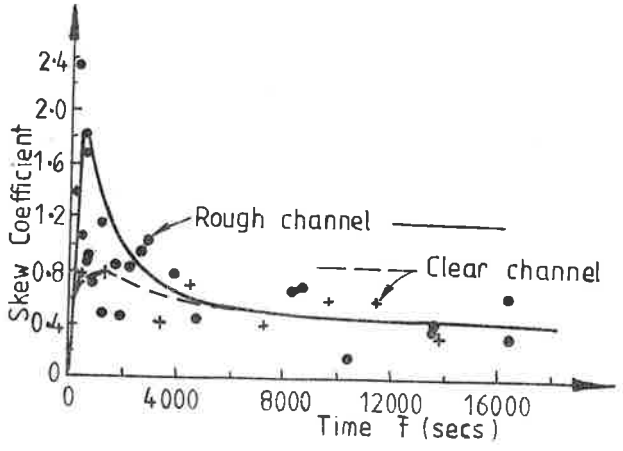



FIG. 5 Temporal Skew Coefficient as Function of Time for Field Data

to indicate a reducing skew coefficient. If the major dispersive mechanisms are accounted for, then a constant dispersion coefficient is applicable only after considerable distances. Indeed, as the final 'equilibrium period' is rarely established, the initial period where the rate of growth of variance is increasing with time must be regarded as the most important in some rivers.

ACKNOWLEDGEMENTS

The writer was supported by the Ministry of Works and Development for the duration of this study.

REFERENCES

Fischer, H.B. 1966: "Longitudinal Dispersion in Laboratory and Natural Channels". California Institute of Technology, Report No KH-R-12, Pasadena.

Nordin, C.F.; Sabol, G.V. 1974: "Empirical Data on Longitudinal Dispersion in Rivers". United States Geological Survey, Water Resources Investigations 20 - 74.

Taylor, G.I. 1921: Diffusion by continuous movements. Proc. London Maths Soc. Series 2, 20, 196-212

Valentine, E.M. 1978: "The Effects of Channel Boundary Roughness on Longitudinal Dispersion". Research Report 78/16, Dept. of Civ. Eng., University of Canterbury, Christchurch, New Zealand.

Valentine, E.M.; Wood, I.R. 1977: Longitudinal dispersion with dead zones. ASCE Vol. 103, No HY9, 975-990.

Valentine, E.M.; Wood, I.R. 1979a: Experiments in longitudinal dispersion with dead zones. ASCE Vol. 105, No. HY8, 999-1016.

Valentine, E.M.; Wood, I.R. 1979b: Dispersion in rough rectangular channels. ASCE Vol. 105, No. HY12, 1537-1553.

DISCUSSION

A J Sutherland Could you please elaborate on the meaning of the key associated with Fig. 3 of the written paper and in particular the correspondence between 'A = rough' and 'B = rough, no dead zones'?
Valentine 'A' represents the numerical solution for both the 'flow zone' and 'dead zones' included from field measurement. 'B' represents the numerical solution which considered only the 'flow zone' as above but ignored the dead zone mechanism. This was done to demonstrate that the flow profiles alone could not produce the sort of dispersion which was actually measured in the field.

McBride In Equation 3, how was K calibrated? How sensitive is the model to changes in K and d?
Valentine K was computed from experimental work in a 'dead zone' in the bed of a laboratory flume. If the equation describing instantaneous dead zone concentrations

$$\frac{dw}{dt} = \frac{KU}{d}(c - w)$$

is employed, and material tracer is injected into the dead zone cavity and allowed to 'bleed off' downstream, the concentration in the main flow, $c_1 = 0$. Therefore for this case

$$\frac{dw}{dt} = -\frac{KU}{d}(w).$$

By monitoring the rate of decay of w in the cavity, knowing d and U, K can be evaluated.

The model appears to be relatively insensitive to values of K and this did not vary significantly for differences in channel dead zone geometry. However, the rate of growth of variance is sensitive to d and the area of dead zones which together describe dead zone volume.

A G Barnett (1) Referring to Fig. 4, while I accept that the slope of the curve might eventually reach the Taylor period with a 1:2 slope it would appear that this has barely happened within the experimental reach, especially for the rough channel. This reinforces the conclusion that this period is unlikely to to be reached in many practical cases.

(2) The comment that the curve never becomes Gaussian in a timewise sense while becoming Gaussian spacewise is not absolutely correct as this relies on the assumption of the Taylor solution. There is an alternative asymptotic solution, attributable to Hayami, which becomes Gaussian in time but not in space. Both the Taylor and Hayami solutions converge at large times.
(3) The mean velocity as measured by the asymptotic behaviour of a dye cloud is the true mean taking dead zone volume into account.
Valentine (1) I agree that one of the most important conclusions from this paper must be that the Taylor period of the dispersion process will not be reached in many practical cases, especially where dead zones are present. Specifically, however, the data in Fig. 4 is the most sensitive parameter when comparing different experiment runs on different days. These data suffer from tracer losses and the data presented were collated at levels of only a few times the ambient conductivity. It is therefore necessary to view these data along with the other parameters.
(2) I am not yet familiar with the Hayami solution. However, I agree that both spatial and temporal distribution will tend towards the Gaussian at large time. Theoretically for the Taylor solution the spatial distribution will achieve this. However, in a physical situation the tendency for the temporal distribution to become Gaussian must be very slow or indeed asymptotic to some negative value of skew.
(3) The asymptotic values of U and $U_{cloud}$ will be equal for the usual definitions. However, the definition of the dead zone geometry as employed in the model in fact removes this from the 'flow zone'. In order to provide input data for the model, flow-zone velocity measurements were taken to arrive at a velocity characterisation and a value of U (in the manner which this would normally be done). This means that U represents the mean average transport velocity in the flow zone only for standard methods of gauging.

J C Rutherford Can you solve the 1-D dispersion equation with dead-zones directly without recourse to moments? Can you take the moment solutions and get concentration versus time or concentration versus distance profiles.?
Valentine I am not aware of a 1-D solution with dead zones other than that developed by Hays (1966) and based on frequency analysis.

It would appear to be impossible to take moment solutions and get back to C-t or C-x curves. However, by using overall dispersion coefficients predicted by moment solutions it should be possible to calculate approximate values of peak concentration or durations above certain concentration levels.

ADDITIONAL REFERENCES

Hays, J.R. 1966: "Mass transport mechanisms in open channel flow". PhD thesis, Vanderbilt University, Nashville, 138 p.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre* *November 17-18, 1980*

# DISPERSION OF A CONTINUOUS POLLUTANT SOURCE IN NON-UNIFORM FLOW

*A J McNulty, University of Canterbury, Christchurch*

## ABSTRACT

A new approach to diffusion of a continuous pollutant source in two-dimensional flow has enabled non-uniform flow conditions to be examined. A model has been developed to calculate vertical concentration variations in a natural river flow situation. The model indicates for the extreme case an increase of 25% in the mixing length. The method used is described and some results given.

## INTRODUCTION

In order to assess the potential impact of the release of a pollutant in a river, it is necessary to determine the pollutant concentration downstream of the release point. Previous studies have found the concentration distribution for the release of a slug in varying velocity distributions and a continuous point release in a uniform velocity with a uniform diffusivity. The case of a continuous release in the more realistic logarithmic velocity and parabolic diffusivity distributions has not yet been approached.

In this investigation, an algorithm has been developed in two-dimensional turbulent shear flow (velocity varying with depth) to obtain the complete downstream concentration distribution. This has enabled the previous assumptions: that there are negligible longitudinal concentration gradients, that the velocity does not vary with depth and that the diffusivity does not vary with depth, to be examined.

## OUTLINE OF PROBLEM

The governing equation in the diffusion equation (or continuity of concentration equation) for a two-dimensional flow is (see appendix for notation)

$$\frac{\partial c}{\partial t} + U \frac{\partial c}{\partial x} = \epsilon_x \frac{\partial^2 c}{\partial x^2} + \frac{\partial}{\partial y}\left(\epsilon_y \frac{\partial c}{\partial y}\right) \qquad (1)$$

The concentration distribution is therefore dependent on the velocity and the diffusivity at any point. In this case it is assumed that there is a local homogeneity of diffusion coefficients. That is, the longitudinal diffusivity is equal to the vertical diffusivity at any point. Figure 1 shows the boundary conditions for the problem. Since this is two-dimensional, water will flow in the x-direction only. The important boundary conditions are: the input conditions at $x' = 0$, that there is an equilibrium concentration a long way downstream, and that there is no loss of concentration across the upper and lower flow boundaries.

Whereas previous work has been done with a uniform velocity this investigation involves a non-uniform velocity and in particular we have chosen a logarithmic profile. This is shown in Fig. 2, along with the corresponding diffusivity profile.

The diffusivity profile is derived directly from the velocity distribution as is standard practice. For means of comparison, the method was also used with a uniform velocity distribution and a standard solution was obtained following Carslaw and Jaeger (1959).

FIGURE 1. Section down channel

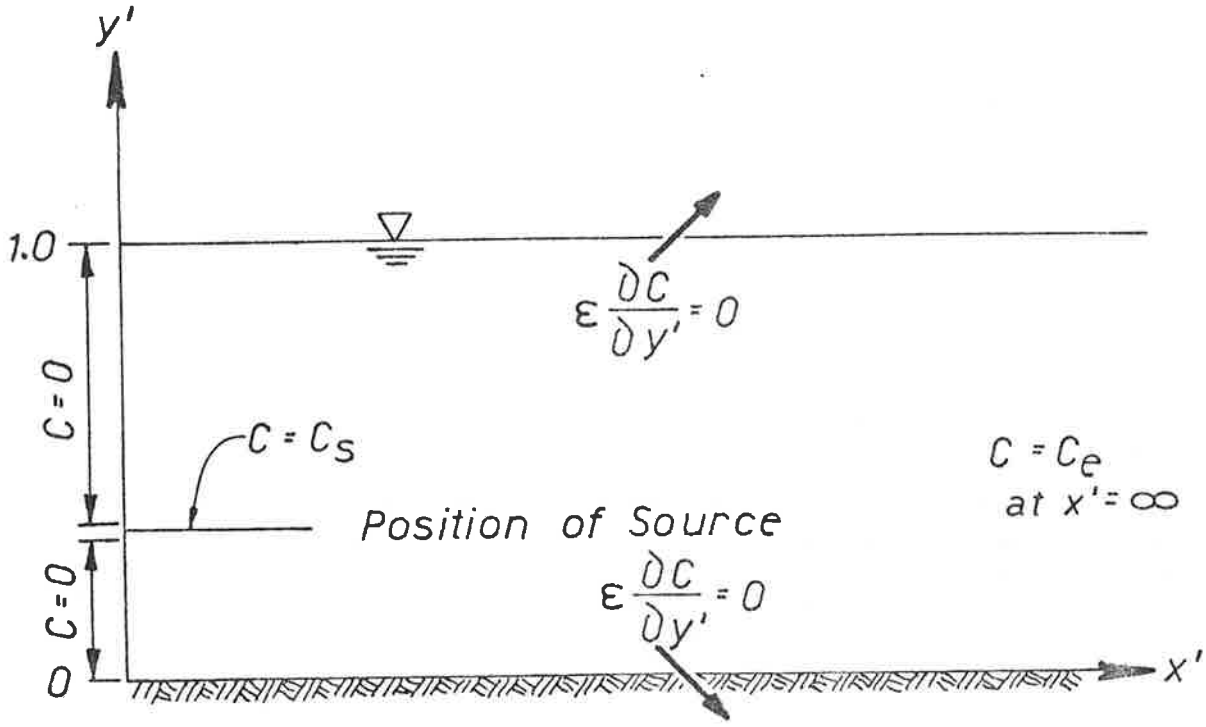
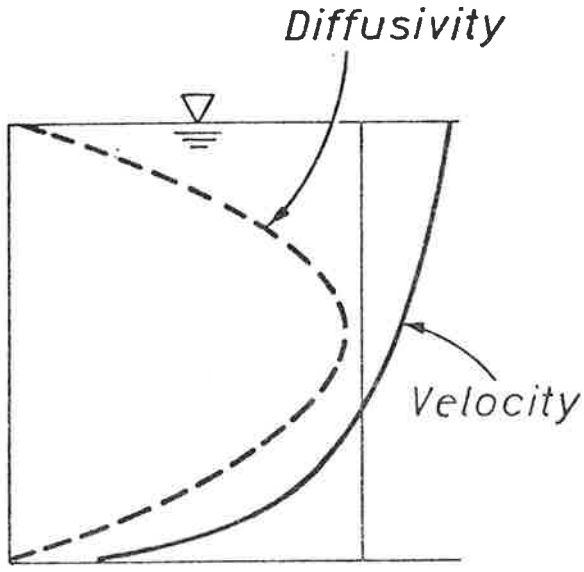



FIGURE 2. Velocity and diffusivity distributions



## METHOD OF SOLUTION WITHOUT THE RESTRICTIVE ASSUMPTIONS

For the sudden release of a slug of pollutant Aris (1956) multiplied the diffusion equation by $x^p$ and integrated from x = 0 to infinity and thus obtained a set of equations from which the moments of the solution (i.e., $\int_o^\infty cx^p\,dx$) could be obtained for all values of p. It is apparent that with the problem set up in its present form this method will lead to infinite moments. However, if a new variable $c_{d*}$ is defined by

$$c_{d*} = c/c_e - 1 \tag{2}$$

then the diffusion equation remains unchanged but the boundary conditions must be modified. Now we have new input conditions and $c_{d*} = 0$ a long way downstream from the source. This enables finite moments to be obtained for this "concentration deficit". Indeed, if we define the moment of the concentration deficit distribution as

$$c_{dp} = \int_o^\infty c_{d*}\, x'^p\, dx' \tag{3}$$

then we can generate an ordinary differential equation for any $c_{dp}$ which requires either initial conditions or lower order moments. These equations can therefore be solved numerically for all the moments required. The method of solution is numerical integration, a standard solution technique which gives good results. It remains to obtain from these moments and the boundary conditions a plot of $c_{d*}$ versus distance downstream at any height in the flow. This will be discussed in terms of the far field and the near field.

## THE FAR FIELD SOLUTION

Far from the source of the pollutant the longitudinal concentration gradients are very small. The exclusion of this term enables a solution of the form

$$c_{d*} = F(x').G(y') \tag{4}$$

to be obtained by the method of separation of variables. Then for y' = constant, that is, choosing any height in the flow, we expect an exponential form of solution for $c_{d*}$. To solve for the coefficients of the exponential series we can take moments of the series. At large distances from the source, the dominating term will be the first term of the exponential series. This will be of the form

$$c_{d*} \simeq A_0(y')\, e^{-\alpha_0 x'} \tag{5}$$

Then from the moments we can solve for both coefficients $A_0(y')$ and $\alpha_0$.

## THE NEAR FIELD

An attempt has been made to determine the near field concentration profile using the computed moments and assuming that the solution form is that of the far field case when longitudinal concentration gradients are neglected, given by

$$c_{d*} = A_0(y')\, e^{-\alpha_0 x'} + A_1(y')\, e^{-\alpha_1 x'} + A_2(y')\, e^{-\alpha_2 x'} + \ldots \tag{6}$$

The coefficients $\alpha_i$ are assumed to be the same as for the uniform velocity - uniform diffusivity case. That is, $\alpha_i = \alpha n_i$, where $\alpha$ is a constant (as determined for the far field solution) and $n_i$ is of the form $n_i = (i + 1)^2$, i = 0, 1, .... Again, moments of the exponential series can be obtained and in addition to these equations, the value of $c_{d*}$ at x' = 0 is known, thus

$$c_{d*}\big|_{x'=0} = A_0 + A_1 + A_2 \ldots \tag{7}$$

Away from the source (at $x' = 0$) all derivatives with respect to $x'$ are zero yielding further equations in the coefficients only.

It remains then, to determine some criteria as to how many moment equations and how many derivative equations to combine with eqn (7). Once this is decided, a matrix can be generated and solved for the appropriate number of coefficients.

Problems with numerical stability prevents the use of all the moments, so the number of moments used varied from two to five and conversely the derivatives were varied from eight to zero. The actual number of moments and derivatives used at any point depends on the distance from the source.

## RESULTS

In order to assure the validity of obtaining a complete concentration distribution by the methods of moments, the standard series solution was compared with that obtained from the "method of moments", using uniform velocity-diffusivity distributions. In both cases the basic diffusion equation (1) was used, with assumptions of uniform velocity, constant diffusivity and negligible longitudinal concentration gradients. Figure 3(a) and (b) are respectively the series solution and the moment solution obtained for a source at 0.998 to 1.00. Figure 4(a) and (b) are similar plots for a source at 0.499 to 0.501. Identical solutions are obtained remote from the source (say 5 depths downstream from the point of release). Closer to the source the agreement is reasonable and certainly sufficiently accurate for engineering purposes. From this we can conclude that in engineering applications, the first term of our exponential series, eqn (4), will describe the distribution adequately. This term is relatively simple to obtain.

The methods of moments was then used for the case of the more correct logarithmic velocity and parabolic diffusivity distributions. The maximum variation between this case and the uniform case occurs when the source is at the bottom of the channel. This is shown in Fig. 5, with the dashed lines representing the distribution for the uniform velocity profiles. It is noticed that for this source location, the low velocity and diffusivity at the channel bed is significantly retarding the dispersion and away from the source there is up to 25% increase in the mixing length with the logarithmic velocity.

The inclusion of trapping mechanisms in the flow has a different effect in the continuous release case than the previous instantaneous slug releases. With the slug releases these mechanisms released trapped material back into the main flow after the pollutant cloud has moved downstream, whereas in the steady case, the trapping regions act as storage elements with no net transfer to or from the main flow. Thus, effectively, the trapping regions will reduce the amount of pollutant in the main flow, enabling equilibrium to be reached at a smaller distance from the source.

Experiments are presently being performed to verify the calculations made for the realistic logarithmic velocity distribution. Some preliminary results from these experiments are shown in Fig. 6. It can be seen quite clearly from these plots that the numerical solution, the solid line, is incorrect close to $x' = 0$. The method is being extended to give a better approximation in the near field, which is of considerable importance in atmospheric diffusion calculations.

## REFERENCES

Aris, R. 1956: On the dispersion of a solute in a fluid flowing through a tube. Proc. Royal Soc. of London, Vol. 235A, 67-77

Carslaw, M.S.; Jaeger, J.C. 1959: "Conduction of heat in solids". Oxford University Press, London.

FIGURE 3. Series (a) and moment (b) solutions for source at surface : uniform velocity and diffusivity

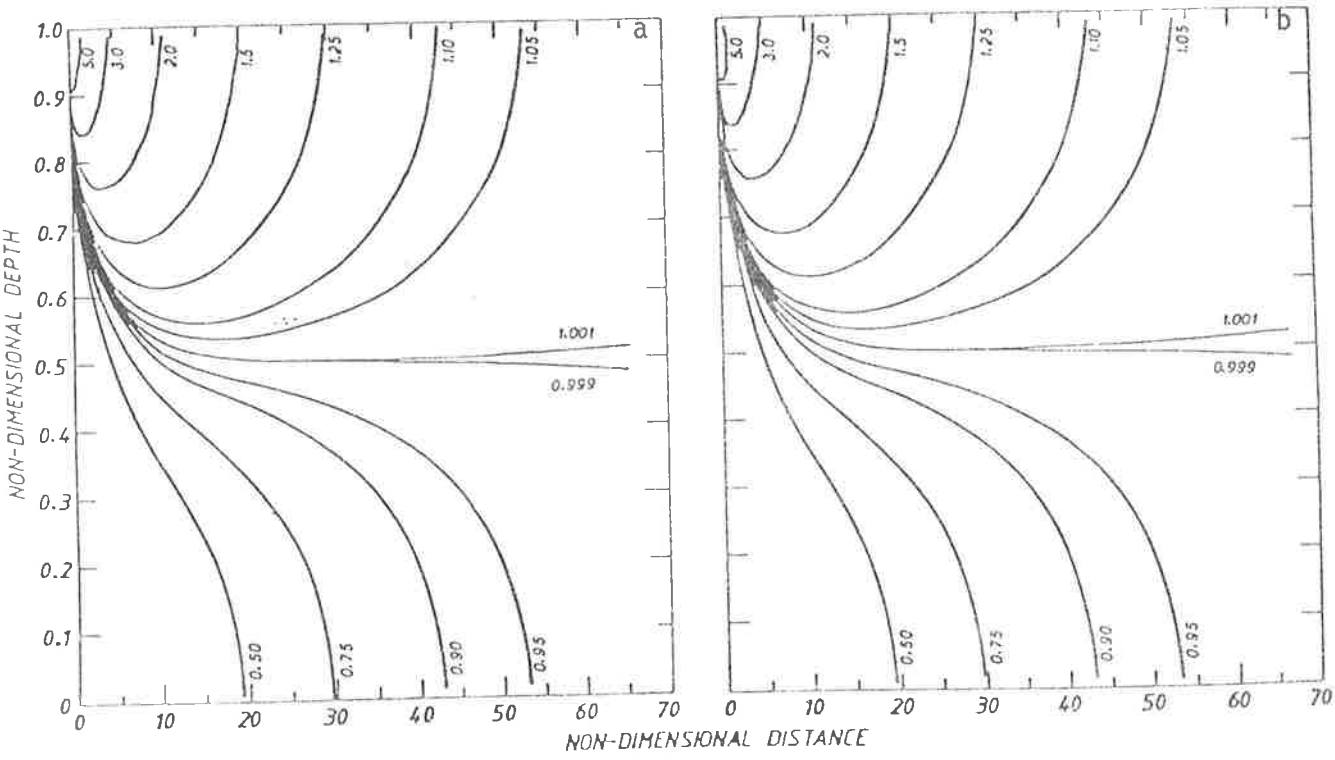
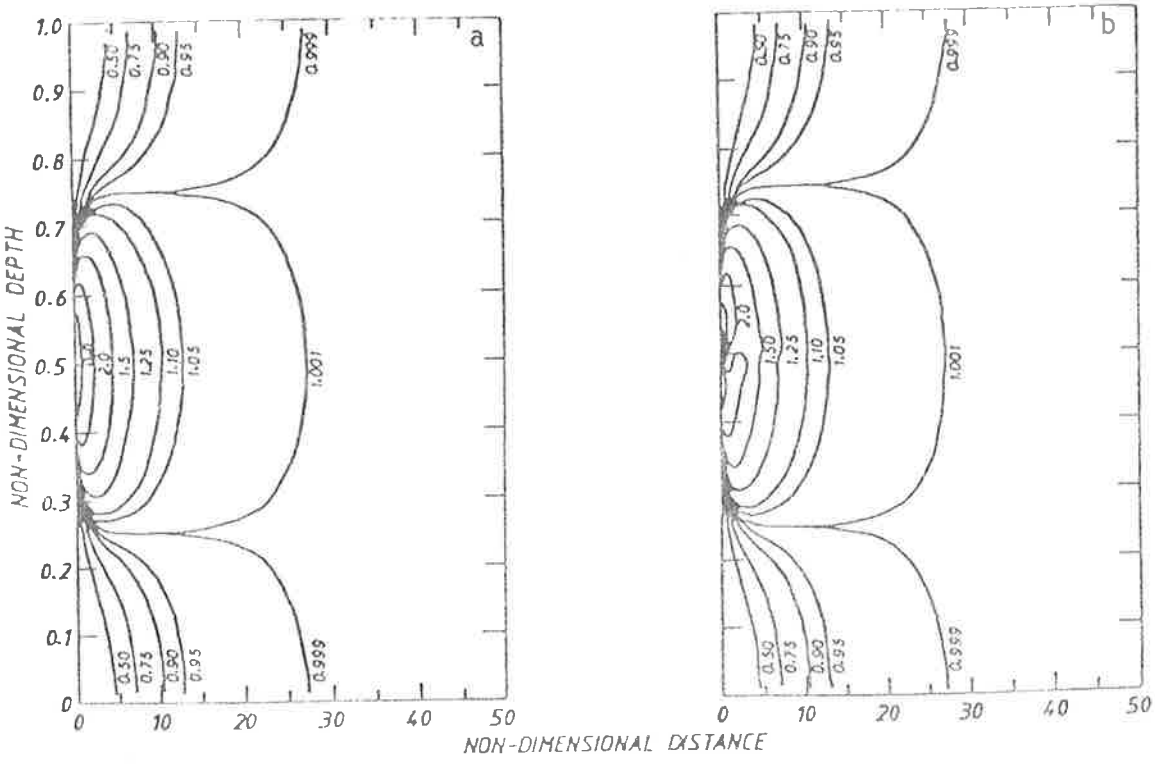
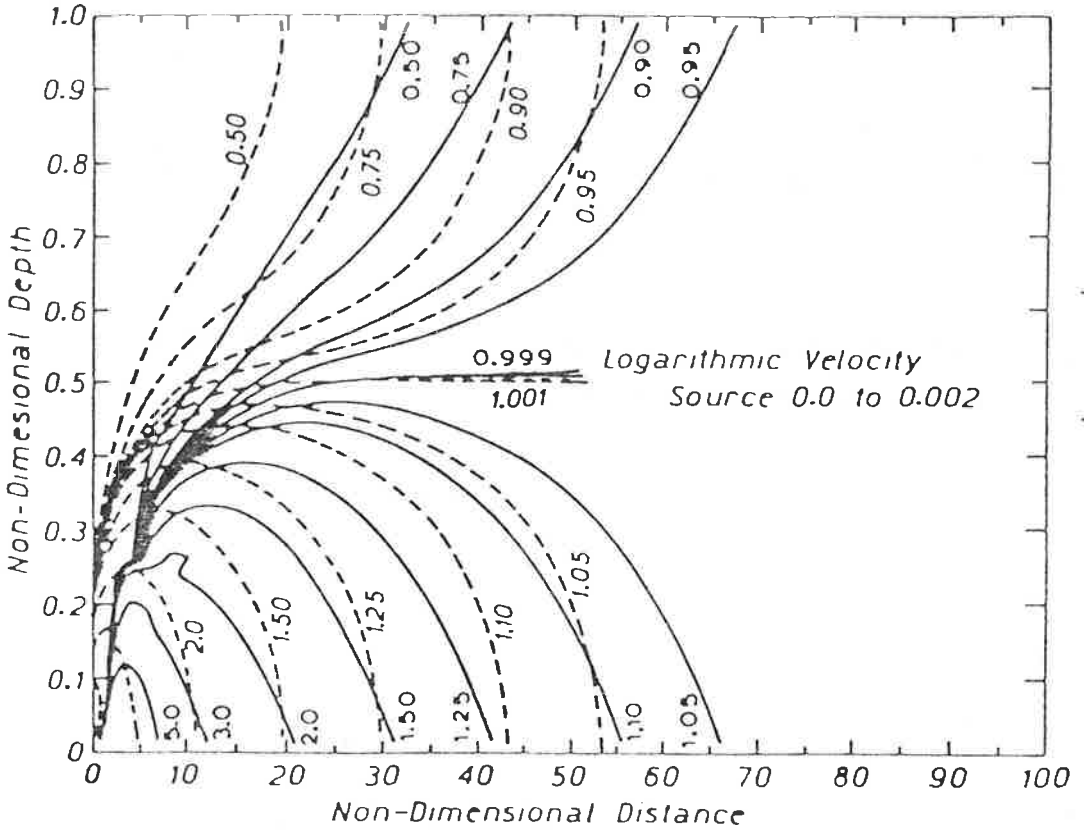


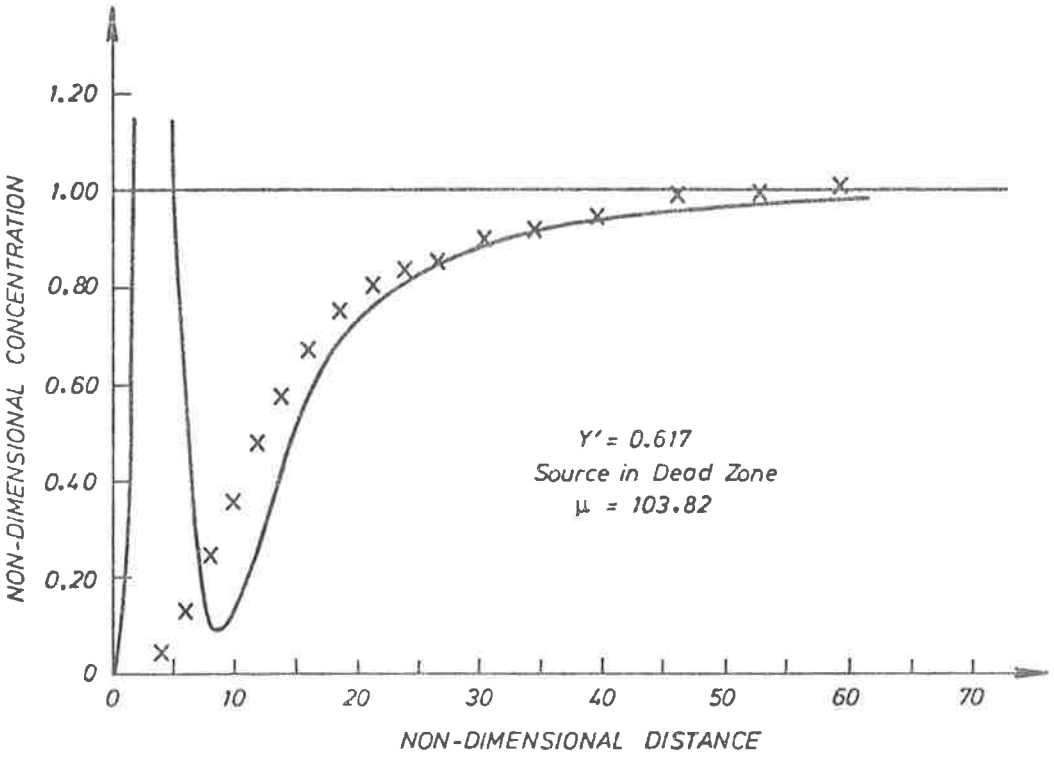

FIGURE 4. Series (a) and moment (b) solutions for source at mid depth : uniform velocity and diffusivity




Lines of equal $C/C_e$

FIGURE 5. Moment solutions for source at bed : uniform and non-uniform velocity and diffusivity






X observed

—— predicted

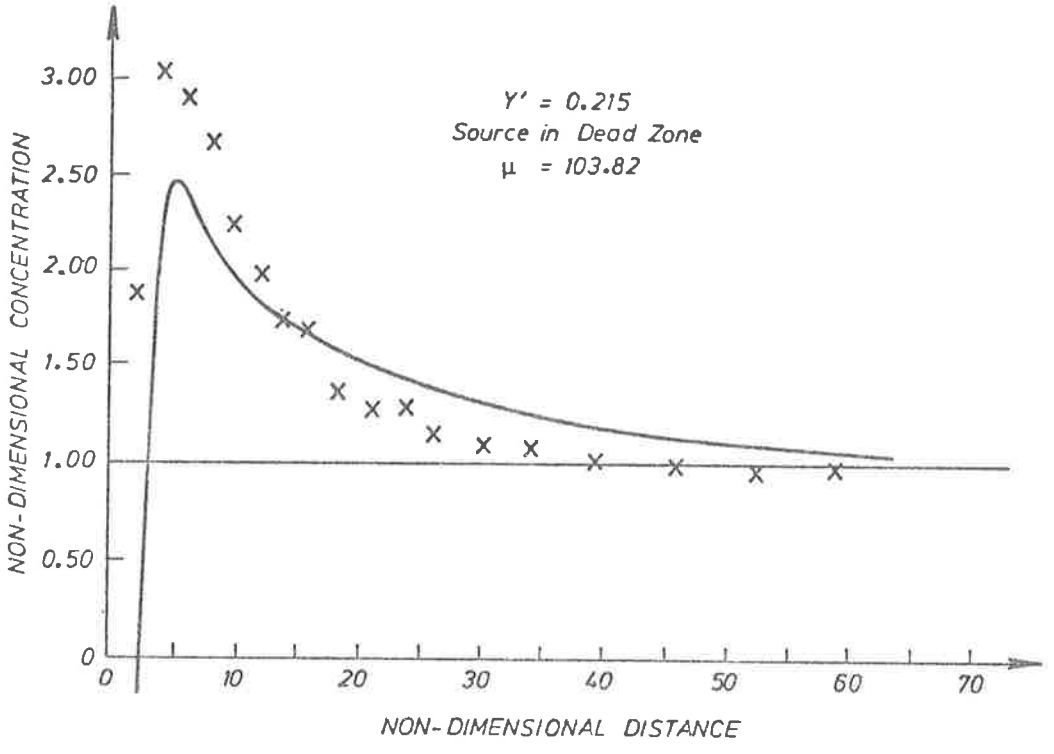



FIGURE 6 Comparison of observed and predicted concentrations for $y_n$ = 75.6 mm.

APPENDIX - notation

| | | |
|---|---|---|
| $A_0(y')$, $A_1(y')$... | = | coefficients of the exponential series solution |
| $c$ | = | concentration distribution of pollutant, a function of x, y and t |
| $c_{dp}$ | = | $p^{th}$ moment of the concentration deficit |
| $c_{d*}$ | = | dimensionless concentration deficit $\frac{c}{c_e} - 1$ |
| $c_e$ | = | fully mixed concentration |
| $c_s$ | = | source concentration at x = 0 |
| $e$ | = | exponential constant |
| $F(x')$ | = | general function of x' |
| $G(y')$ | = | general function of y' |
| $n_i$ | = | $(i + 1)^2$ for i = 0, 1, ... |
| $p$ | = | moment number, p = 0, 1, 2 ... |
| $t$ | = | dispersion time |
| $u$ | = | flow velocity |
| $U$ | = | mean flow velocity |
| $u_*$ | = | shear velocity |
| $x$ | = | longitudinal displacement from the source |
| $x'$ | = | dimensionless distance, $(x/y_n)$ |
| $y$ | = | vertical displacement above bed |
| $y'$ | = | dimensionless depth, $(y/y_n)$ |
| $y_n$ | = | mean water depth |
| $\alpha_0$, $\alpha_1$, $\alpha_2$ ... | = | exponent coefficients for the series solution |
| $\varepsilon$, $\varepsilon_x$, $\varepsilon_y$ ... | = | turbulent diffusivities; homogeneous, longitudinal and vertical values respectively |
| $\mu$ | = | $(\bar{u}/u_*)^2$ |

DISCUSSION

J C Rutherford In Fig. 5, U and D values do not appear explicitly. Were similar values of U and D used in the uniform and non-uniform cases?

McNulty Velocity is defined non-dimensionally in the model as $u = U\chi(y)$. U is exactly the same for both the uniform flow solution and the non-uniform (logarithmic) flow solution. For uniform flow $\chi(y) = 1$ for all y, and clearly in the non-uniform case, by the definition of U, $\int_0^1 \chi(y)\, dy = 1$.

The diffusivity is similar, that is $E_x = E_y = E = D\psi(y)$ where $D = U_*^2 Y_n/U$ and $\psi(y) = (1-y)/\frac{\partial x(y)}{\partial y}$ which depends entirely on the choice of velocity profile. Again the integral $\int_0^1 \psi(y)\, dy = 1$ for both velocity distributions.

M J O'Sullivan Do your results apply to large dead zones, e.g., back-waters in rivers?

McNulty These results may be applied to large irregular trapping elements which are random in their location and number. In this case it would be a matter of examining each "dead zone" to determine the volume of water it contained and hence the effective volume of "smeared" eddy required to allow for each dead zone. It would also be necessary to estimate the effect of any such zone on the mainflow velocity. The computer solution will then allow for a non-zero mainflow/dead zone interface velocity rather than taking out the dead zones and having the velocity zero everywhere on the mainflow boundary.

E M Valentine When considering larger pockets, such as embayments or backwaters, the dead zone model should still apply if the parameters which describe the dead zone volume are known. However, for large embayments a large non-dimensional distance or time would be required to reach an equilibrium state.

McNulty This is true for the instantaneous release because pollutant will be released back into the mainflow region after the cloud has moved downstream. With a continuous release the dead zones in fact remove pollutant from the mainflow. This effectively increases the volume available for mixing as compared to the same mainflow region with no dead zones. Thus the pollutant has a shorter mixing time.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre November 17-18, 1981*

# MODELLING ADVECTION AND LONGITUDINAL DISPERSION

*G B McBride, Water & Soil Science Centre, MWD, Hamilton*

## ABSTRACT

The performance of two well-known and two recently developed numerical schemes for modelling advection and longitudinal dispersion in river and estuary channels is described. Performance has been assessed by applying each scheme to an idealised test problem in steady uniform channel flow and making a qualitative visual assessment of degree of false smearing/oscillation produced. Recommendations on scheme selection for river and estuary problems are given.

## INTRODUCTION

After a slug of tracer is mixed across the cross-section of a channel, for example during a dye test, the subsequent downstream distribution of the tracer is brought about by the simultaneous processes of advection and longitudinal dispersion. A mathematical model of these processes can be constructed to assess the mixing characteristics of the channel. For problems in which the flow is approximately steady and uniform, analytical solutions to the model equation may be found. Rutherford (1981) has given nomograph solutions for this case. For unsteady or non-uniform flows an approximate numerical solution to the model equation must be sought using a "numerical scheme". However, these schemes often introduce errors in the form of false smearing and oscillations near the front and tail of the tracer patch.

After reviewing the nature of advection and longitudinal dispersion processes, this paper compares the performance of two well-known numerical schemes that have been used in river and estuary pollution studies. Performance has been assessed by visual comparison, using computer graphics, of scheme results for an idealised test problem. The performance of two recently-developed higher accuracy schemes is also included, and conclusions are drawn as to the criteria that should be used to select a scheme for a particular study.

## ADVECTION AND LONGITUDINAL DISPERSION PROCESSES

Advection refers to the process of the downstream transport of the tracer patch by the mean channel cross-section velocity U, where

$$U = Q/A \tag{1}$$

and Q is the channel water rate of flow and A is the channel cross-section area.

Longitudinal dispersion refers to the spreading of the tracer patch along the river channel. It arises because lateral and vertical variations in longitudinal velocity cause tracer to be transported downstream faster in the main stream than near the banks and bed. This "trapped" tracer is eventually mixed with the mainstream flow by lateral and vertical currents resulting in a longitudinal spreading of the tracer; hence the term longitudinal dispersion.

Figure 1 typifies the lateral variation of longitudinal velocity that gives rise to longitudinal dispersion.

On Fig. 1 the cross-section AA' is advected at the mean velocity U. The departure of the actual velocity u from the mean is u', i.e.

$$u = U + u' \tag{2}$$

FIGURE 1 Lateral variation of longitudinal velocity

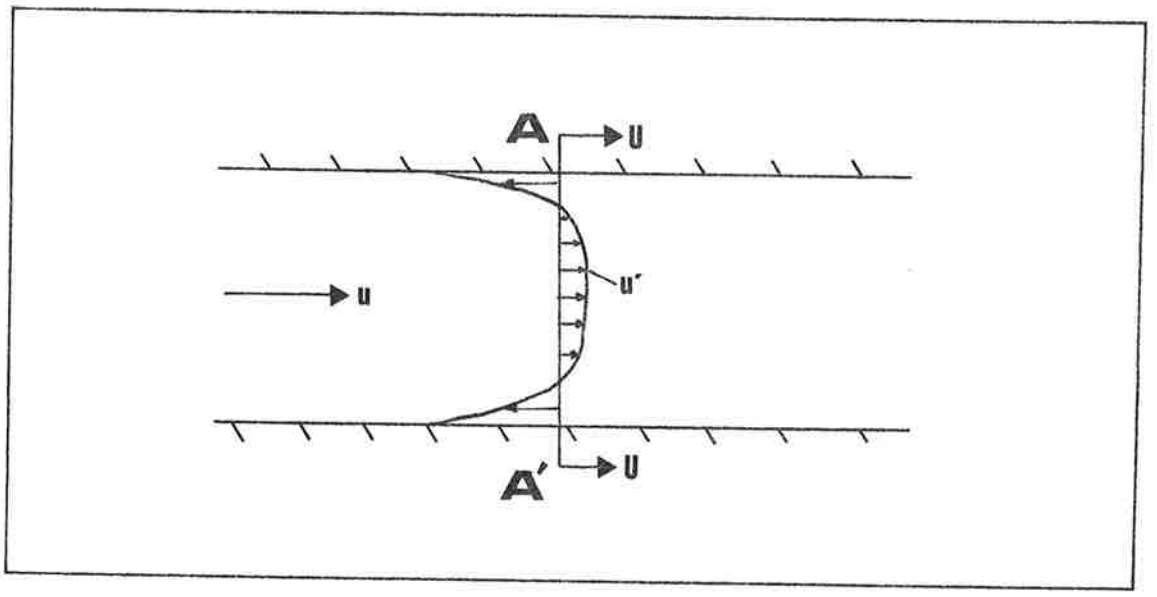
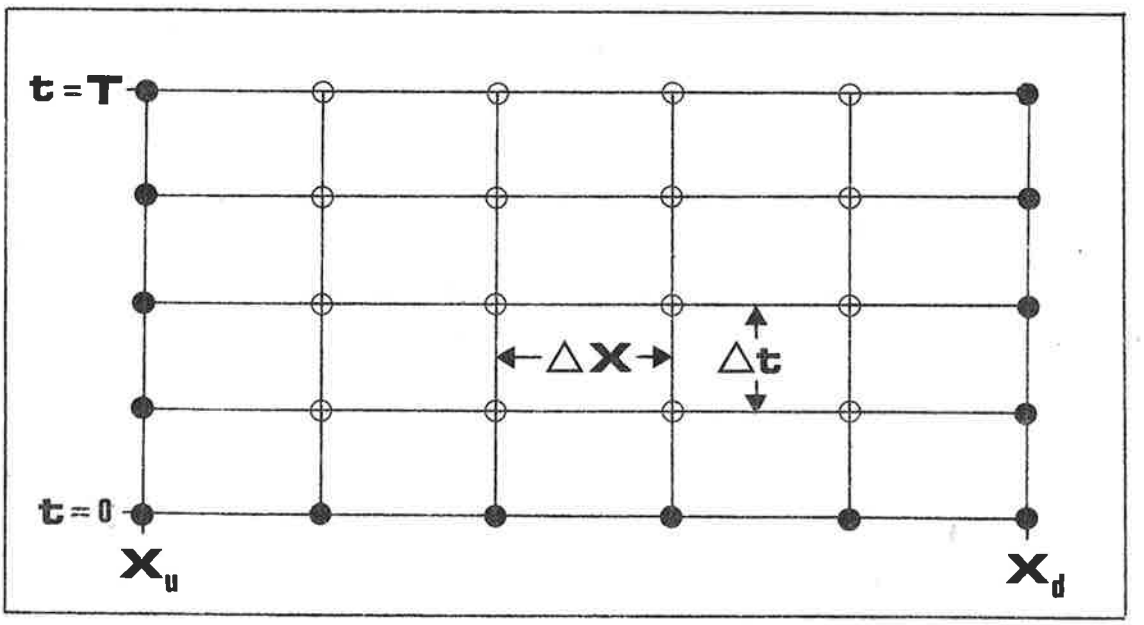



FIGURE 2 Example computational grid



It is the lateral (and vertical) variations in u' that give rise to longitudinal dispersion.

It has been found, as reviewed in Fischer *et al.* (1979), that the rate at which mass of the tracer is dispersed through AA' is proportional to the *longitudinal* gradient of tracer concentration. The constant of proportionality is called the longitudinal dispersion coefficient.

Typical values of the coefficient are 10 $m^2.s^{-1}$ for the Waipa River (McBride and Rutherford 1982) and 50-200 $m^2.s^{-1}$ for the Waikato River (Rutherford *et al.* 1980). Strictly this description of longitudinal dispersion should not be applied over an "advective zone" extending some distance downstream from an inflow; the extent of this zone is a topic of current research (McQuivey and Keefer 1976; Beltaos 1980). However, in modelling studies it is usually assumed to hold over the whole channel being modelled.

## NUMERICAL SCHEMES

When using a numerical scheme, one seeks to compute values of tracer concentration at various points along a channel reach at a number of times. One therefore speaks of a "computational grid" being imposed on the channel upon which those calculations are to be made. A simple example is shown in Fig. 2 in which we wish to solve for values of tracer concentration at four points between $X_u$ and $X_d$ at four times between time = 0 and time = T.

This is a uniform grid since the grid time and distance intervals ($\Delta x$ and $\Delta t$) are constant. By using the scheme difference equation the unknown concentrations are solved for at the interior nodes of the grid (open circles) given known values of the concentration at t = 0 (the so-called "initial conditions") and at boundary nodes of the grid (known from so-called "boundary conditions"). These known values are shown as solid circles. In practice grids contain hundreds, if not thousands, of nodes to obtain good resolution. Computers are used to carry out the calculations.

The schemes discussed in this paper are:

A The QUAL1 scheme developed by the Texas Water Development Board (1971). This scheme has been widely used for steady non-uniform river flows (Grenney *et al.* 1978).

B The Stone and Brian (1963) scheme. This scheme has been used for unsteady flows in rivers (Keefer and Jobson 1978) and in estuaries (Thatcher and Harleman 1972).

C The QUICKEST scheme of Leonard (1979). This is a recently developed scheme that can be applied to unsteady non-uniform flow in rivers and estuaries.

D The LAMBDA scheme of McBride (*in prep*). This is also a recently developed scheme. It applies to steady non-uniform river flows and has been used successfully for the Waipa "milk spill" data (McBride and Rutherford 1982).

Difference equations for these schemes are given in the Appendix.

A notable feature of schemes C and D is that for some choices of $\Delta x$ and $\Delta t$, the downstream boundary concentration (the solid circles vertically above $X_d$ on Fig. 2) need not be specified in advance. This is especially useful if calculation of tracer concentration at the distance is the object of the modelling exercise. To use schemes A and B for such a case one must extend the downstream boundary to a point where it can reasonably be expected that the effect of the boundary condition on upstream concentrations is small. At that point a zero concentration gradient is commonly imposed, e.g., Texas Water Development Board (1971).

SCHEME PERFORMANCE

Schemes A-D have been used for an idealised problem: the "Top Hat Test". This is now a common test for numerical schemes, albeit a severe one. In this test we consider a channel of length 100 $\Delta x$. At time zero the concentration everywhere is 5 units, except between nodes 10 and 19 where it is 20 units. This is shown as the dotted line on Fig. 3. We simulate advection/longitudinal dispersion in this channel for the length of time required for the mass of tracer to advect downstream by $36\Delta x$. This test is carried out using each of Schemes A-D. The upstream boundary conditions is taken as 5 units at node 0 for all time, and at the downstream boundary (when required) a zero concentration gradient is assumed at node 100.

Processes of advection and longitudinal dispersion modelled on a particular grid are very conveniently categorised by the following dimensionless numbers

Courant number, $\alpha = U\Delta t/\Delta x$
Dispersion number, $\sigma = D\Delta t/\Delta x^2$

where D is the longitudinal dispersion coefficient. By using schemes A-D for various combinations of $\alpha$ and $\sigma$, we can quickly build up an understanding of scheme accuracy.

The most severe test for these schemes is that of pure advection where D=0, i.e., $\sigma = 0$. In Fig. 3 we show the schemes' predictions for pure advection, with $\alpha$ chosen as unity (since the LAMBDA scheme is only defined for that value). Some immediate conclusions can be drawn.

+ The QUICKEST and LAMBDA schemes (schemes C and D) both reproduce the exact solution to the problem for this Courant number ($\alpha = 1$), i.e., the tophat is simply transported downstream by a distance $36\Delta x$.

+ The Stone and Brian scheme (scheme B) generates some smearing and substantial oscillations at the tail and peak of the tophat.

+ The QUAL1 scheme (scheme A) severely disperses and damps the true concentration wave. This is because of a phenomenon known as "numerical dispersion", i.e., the scheme used to simulate advection also produces a false longitudinal dispersion*.

At the Courant number used for this test ($\alpha = 1$) the QUICKEST and LAMBDA schemes make use of the fact that from one time step to another tracer is simply advected from one node to the adjacent downstream node. In fact the LAMBDA scheme is set up on a grid that deliberately forces the local Courant number to be unity, even for non-uniform flow. The other schemes are not so constrained and it is instructive to compare their performance at other Courant numbers. No further testing of the QUAL1 scheme is necesary; it will always introduce a substantial amount of false dispersion.

Figure 4 shows the pure advection test for the QUICKEST scheme (scheme C) for $\alpha = 0.6$. Although there is a little smearing and oscillation, these results are very accurate when compared to the results of many other advection schemes (McBride 1980). The results for schemes A and B (not shown) are similar to those for $\alpha = 1$ shown in Fig. 3, i.e., they are generally unsatisfactory.

Inclusion of longitudinal dispersion ($\sigma > 0$) generally improves the accuracy of these schemes, since it provides a mechanism for damping out oscillations if they occur. Figure 5 shows the predictions of schemes B and C for $\alpha = 0.6$ with a very low dispersion number, $\sigma = 0.01$. Also shown on the figure are the results for the LAMBDA scheme (scheme D), which closely approximate the exact solution. As can be seen, by comparison with Figs. 3 and 4, inclusion of even a very small amount of longitudinal dispersion gives much improved results, especially for the Stone and Brian scheme (scheme B).

---

* In fact the numerical dispersion coefficient for the QUAL1 scheme can be shown to equal $\frac{U}{2}(\Delta x + U\Delta t)$.

FIGURE 3

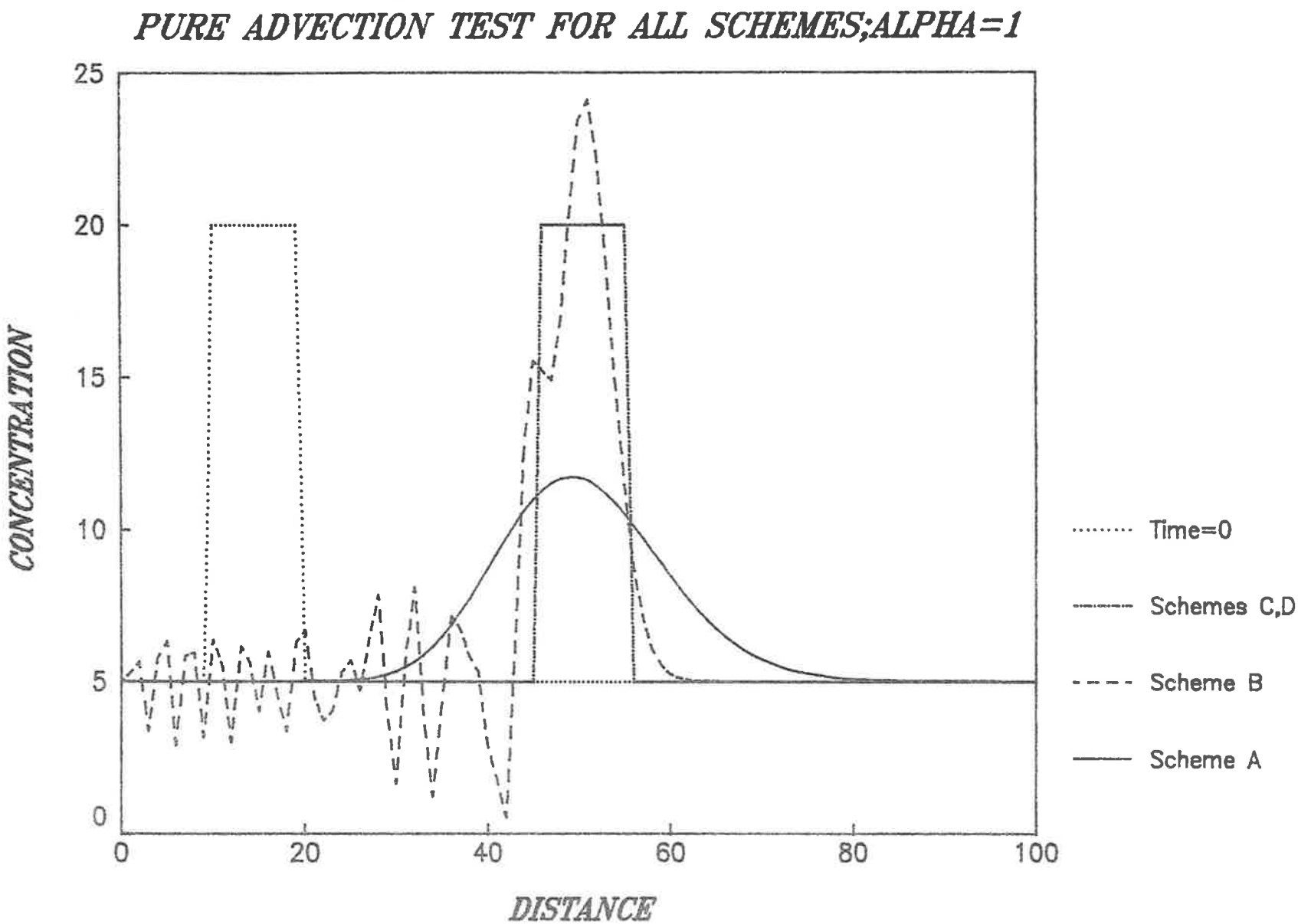

PURE ADVECTION TEST FOR ALL SCHEMES;ALPHA=1
CONCENTRATION
25
20
15
10
5
0
0
20
40
60
80
100
DISTANCE
Time=0
Schemes C,D
Scheme B
Scheme A

FIGURE 4

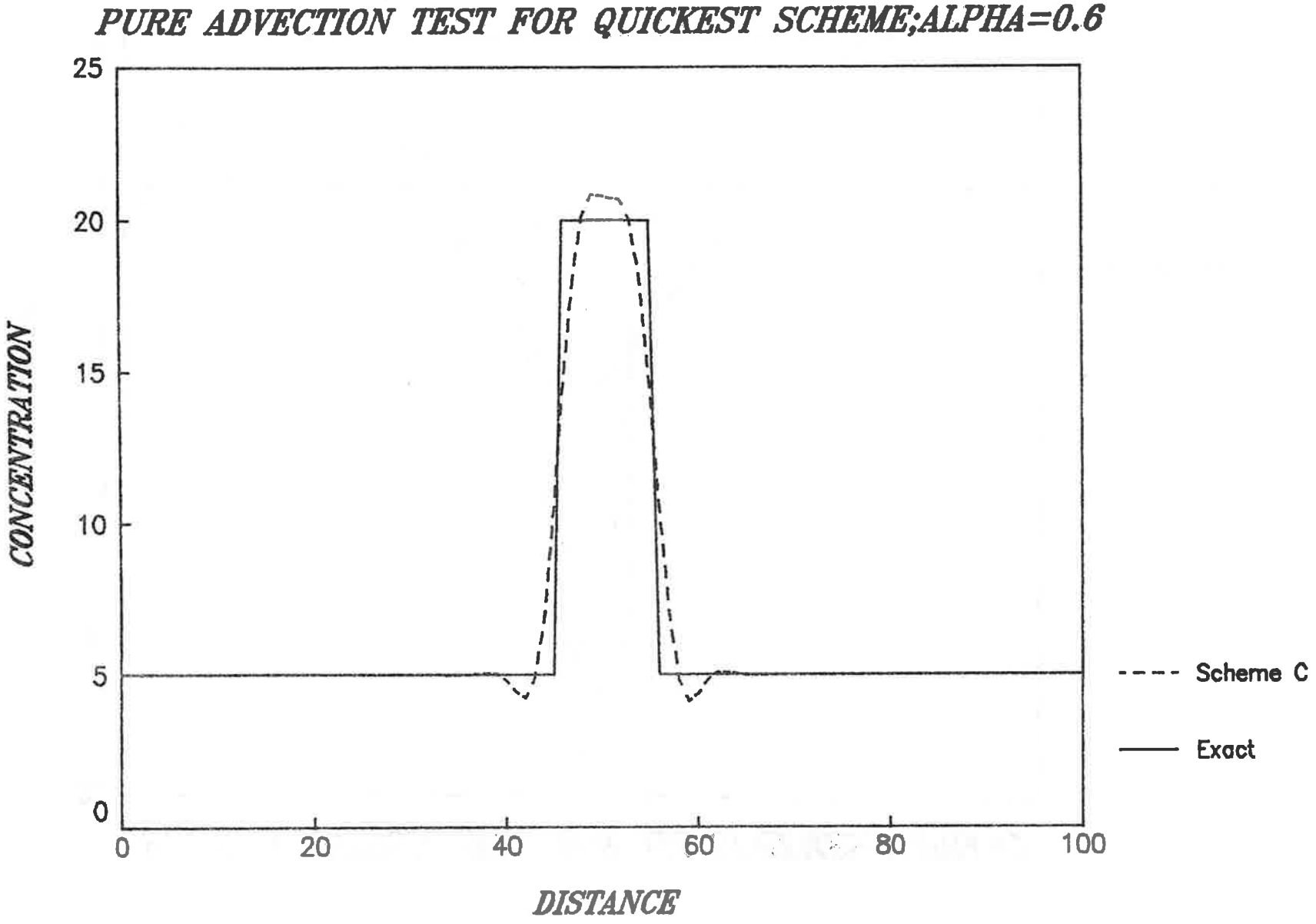
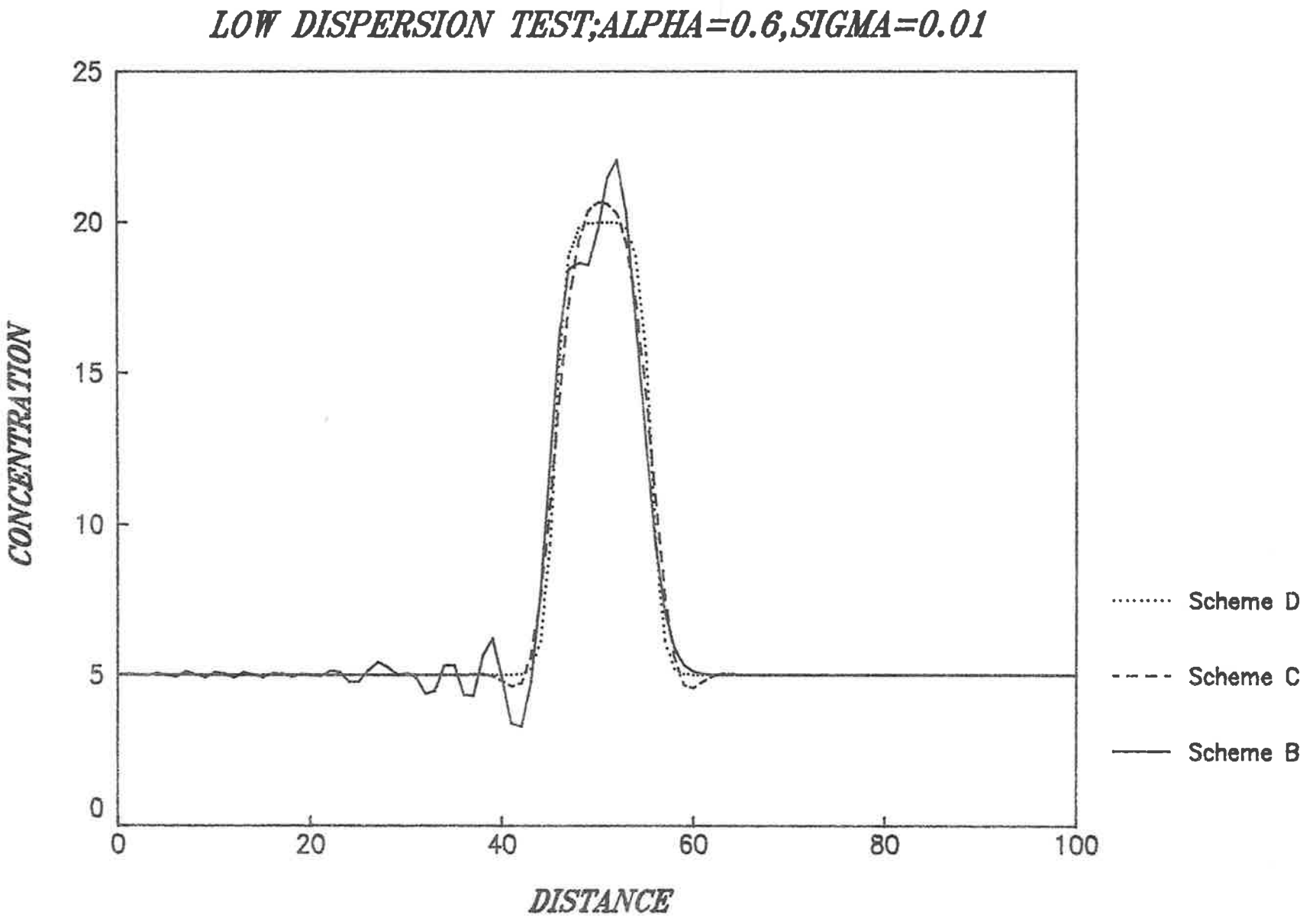

PURE ADVECTION TEST FOR QUICKEST SCHEME;ALPHA=0.6
CONCENTRATION
25
20
15
10
5
0
0
20
40
60
80
100
DISTANCE
Scheme C
Exact

FIGURE 5


LOW DISPERSION TEST;ALPHA=0.6,SIGMA=0.01
CONCENTRATION
25
20
15
10
5
0
0
20
40
60
80
100
DISTANCE
Scheme D
Scheme C
Scheme B

On Fig. 6 is shown the effect of increasing the Courant number on the performance of the Stone and Brian scheme. The figure shows that as $\alpha$ is raised above 1 significant upstream oscillations occur.

The above results suggest that it would be helpful to examine scheme accuracy for a variety of values of $\alpha$ and $\sigma$. This is not necessary for the LAMBDA scheme since it has already been shown (McBride, in prep.) that this scheme is very accurate for all $\sigma$, at least up to $\sigma = 1$. A large number of tests for the Stone and Brian and QUICKEST schemes have been carried out for $0 \leq \alpha \leq 2.4$ and $0 \leq \sigma \leq 1$, thus covering the range of these parameters likely to obtain in modelling studies. A qualitative assessment has been made as to the accuracy of the schemes' predictions by visual inspections of each simulation using computer graphics. Results were adjudged accurate if oscillations did not exceed 1 unit in amplitude. Figures 7 and 8 indicate the combinations of $\alpha$ and $\sigma$ for which accurate results have been obtained using these two schemes for the "Top Hat Test". The shaded area on each graph indicates the region where significant oscillations and smearing occurred. The Stone and Brian scheme is stable for all values of $\alpha$ and $\sigma$, though inaccuracies occur increasingly as $\alpha$ increases. The QUICKEST scheme has been shown by Leonard (1979) to be stable only for certain combinations of $\alpha$ and $\sigma$, as shown on Fig. 8. However, it is considerably more accurate than the Stone and Brian scheme at low values of $\sigma$.

## CONCLUSIONS

The following conclusions can be drawn from the above results.

1 The QUAL1 scheme is inappropriate for estimating longitudinal dispersion in channels because of induced numercial dispersion.

2 For steady non-uniform channel flow the most accurate results are obtained using the LAMBDA scheme.

3 For unsteady flows, such as in estuaries, both the Stone and Brian and QUICKEST schemes can give accurate results, with the QUICKEST scheme performing better at low dispersion numbers.

It should be pointed out that these conclusions are based on the "Top Hat Test" only. In the author's opinion there is no substitute for carrying out an accuracy analysis such as the above for a test depicting the type of problem to be modelled. This is not an onerous task once basic skills have been built up, especially if one uses modern computer graphics facilities. This testing ought to include several schemes and the effect of including other processes, i.e., lateral inflows or internal sources/sinks.

## ACKNOWLEDGEMENTS

Fruitful discussions have been had with Drs J C Rutherford and A G Barnett. Dr R A Hoare supplied the excellent computer graphics software (used on the University of Waikato PDP 11/70 machine).

## REFERENCES

Beltaos, S. 1980: Longitudinal dispersion in rivers. ASCE Vol. 106, No. HY1, pp 151-172.

Fischer, H.B.; Imberger, J.; List, E.J.; Koh, R.C.Y.; Brooks, N.H. 1979: "Mixing in Inland and Coastal Waters". Academic Press. New York, 483p.

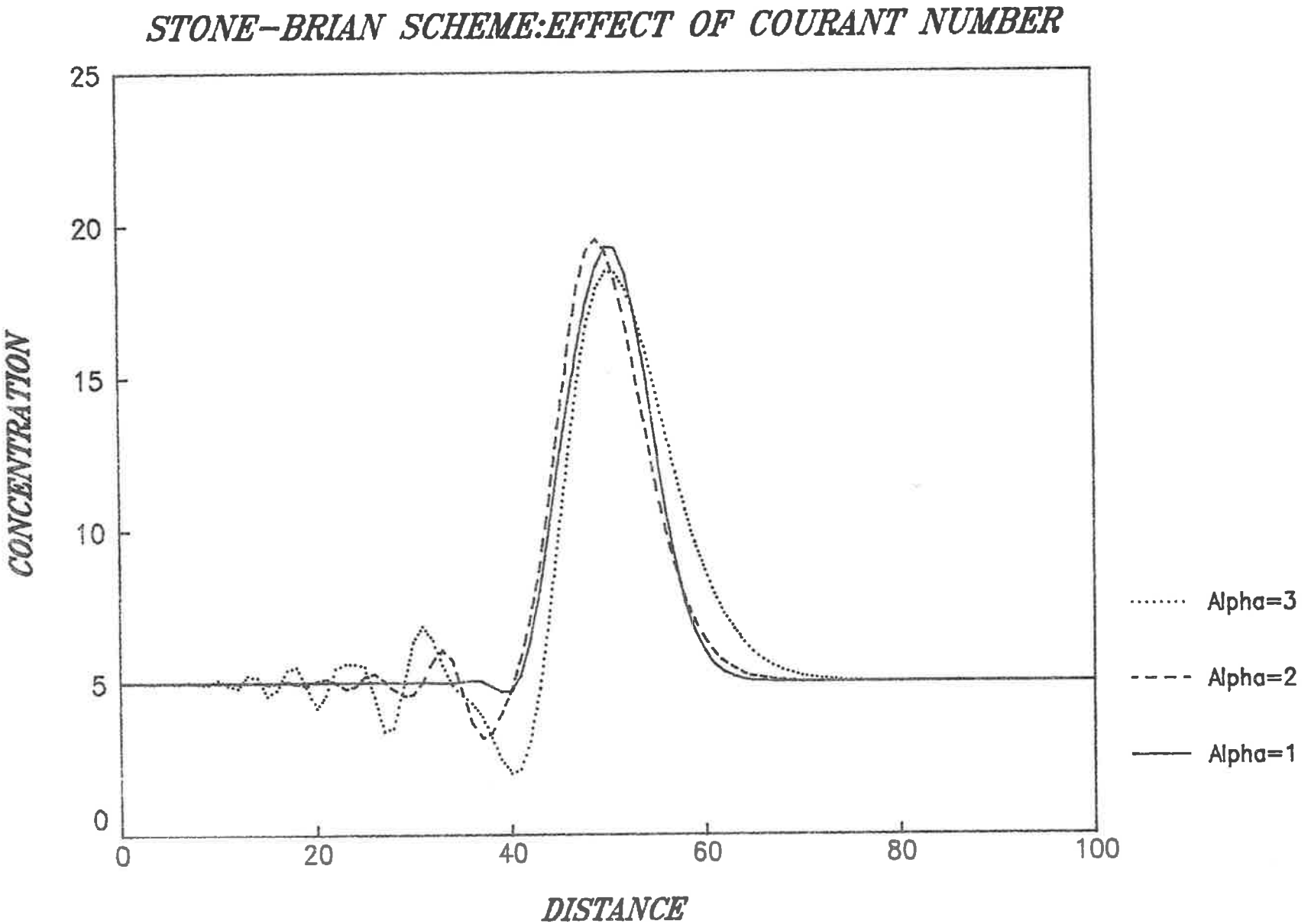
Grenney, W.J.; Teuscher, M.C.; Dixon, L.S. 1978: Characteristics of the solution algorithms for the QUAL1 river model. Wat. Poll. Con. Fed. Vol. 50, 151-157.

Keefer, T.N.; Jobson, H.E. 1978: River Transport modelling for unsteady flows. ASCE Vol. 104, No HY5, 635-647.

Leonard, B.P. 1979: A stable and accurate modelling procedure based on quadratic upstream interpolation. Computer Methods in Applied Mechanics and Engineering Vol. 19, 59-98.

McBride, G.B. 1980: Results of Numerical Experiments into the Behaviour of Numerical Methods for Modelling Advection Dominated River Transport. Unpub. Int. Rep. No. 80/12, Water & Soil Science Centre, MWD, Hamilton.

FIGURE 6


STONE-BRIAN SCHEME:EFFECT OF COURANT NUMBER
CONCENTRATION
25
20
15
10
5
0
0
20
40
60
80
100
DISTANCE
Alpha=3
Alpha=2
Alpha=1

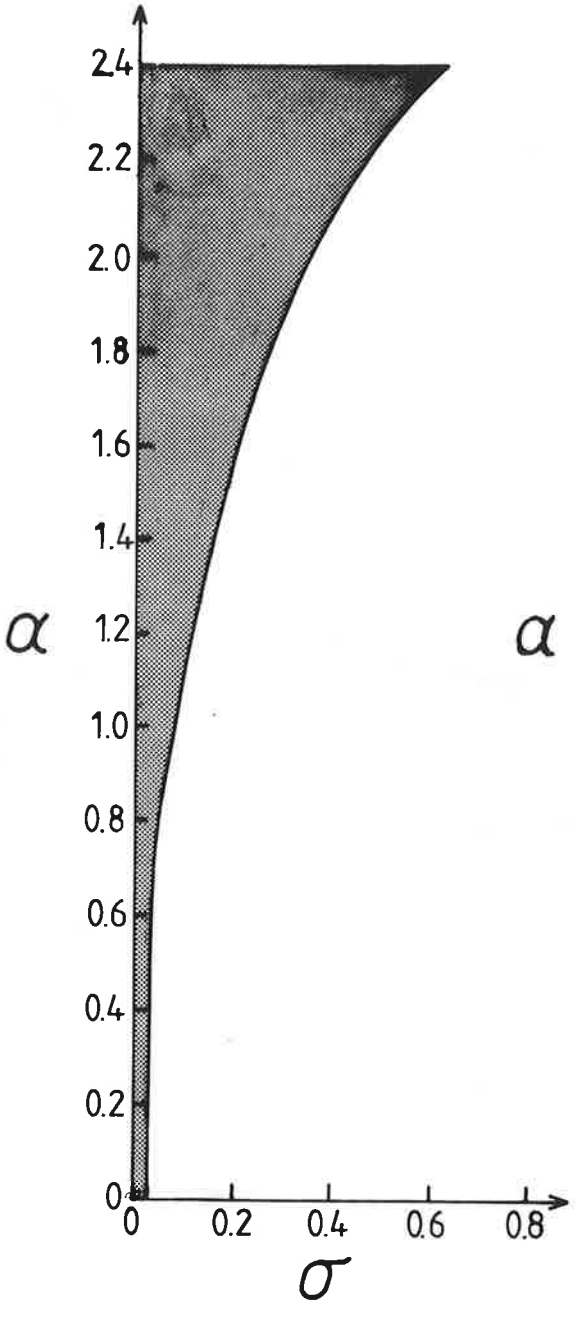



FIGURE 7 Accuracy plot for Stone-Brian scheme

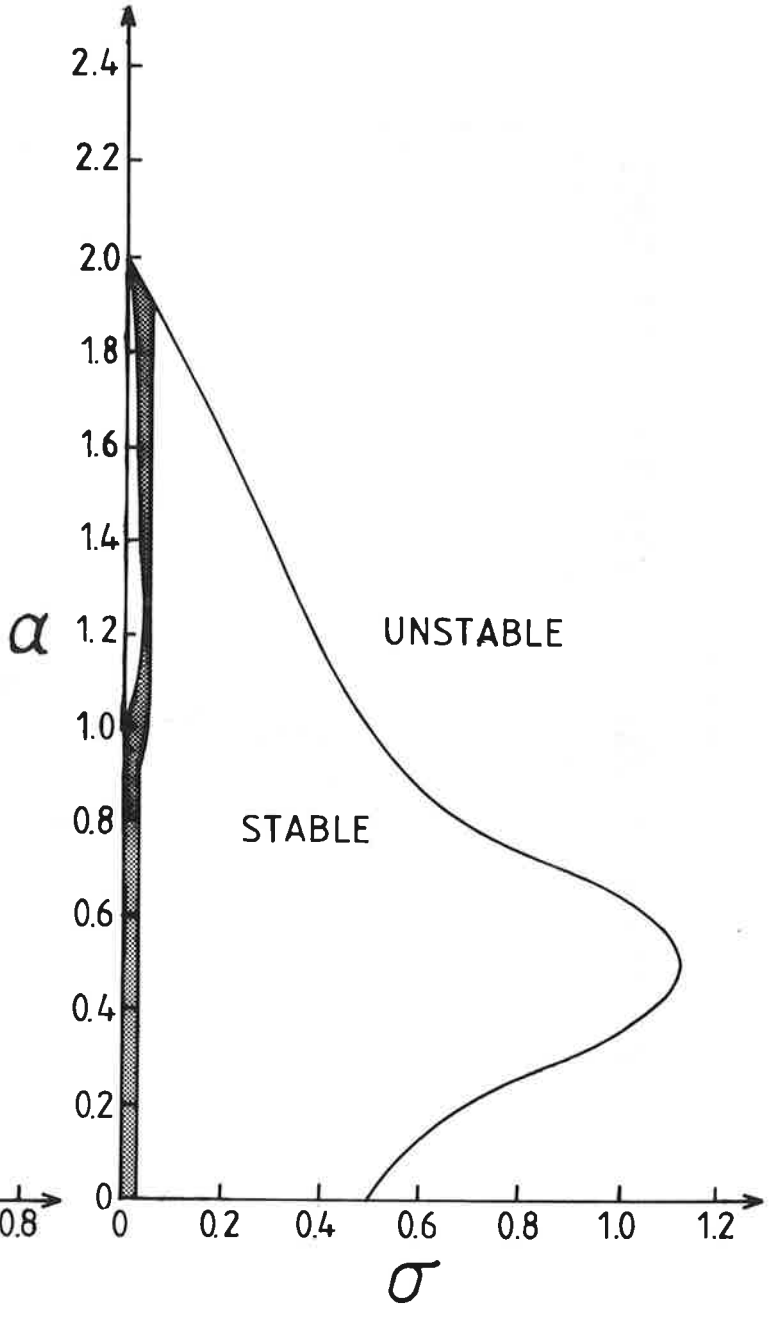



FIGURE 8 Accuracy plot for QUICKEST scheme

McBride, G.B. (in prep.): A Langrangian Scheme for River Pollutant Transport.

McBride, G.B.; Rutherford, J.C. 1982: Waipa River. In "Aquatic Oxygen Seminar Proceedings, Hamilton, November, 1980". Water and Soil Misc. Pub. No. 29, MWD, Wellington.

McQuivey, R.S.; Keefer, T.N. 1976: Convective model of longitudinal dispersion. ASCE Vol. 102, No. HY10, 1409-1424.

Rutherford, J.C. 1981: Handbook on Mixing in Rivers. Water & Soil Misc. Pub. No. 26, MWD, Wellington.

Rutherford, J.C.; Taylor, M.E.U.; Davies, J.D. 1980: Waikato River pollutant flushing rates. ASCE Vol. 106, No. EE6, 1131-1150.

Stone, H.L.; Brian, D.L.T. 1963: Numerical solution of convective transport problems. American Institute of Chemical Engineers Journal Vol. 9, No. 5, 681-688.

Texas Water Development Board 1971: "Simulation of Water Quality in Streams and Canals. Theory and Description of the QUAL1 Mathematical Modelling System", Report 128, Texas Water Development Board, Austin, Texas.

Thatcher, M.; Harleman, D.R.F. 1972: "A Mathematical Model for the Prediction of Unsteady Salinity Intrusion in Estuaries". R.M. Parsons Laboratory Report No 144, Massachusetts Institute of Technology, Cambridge, Massachusetts.

## APPENDIX : SCHEME DIFFERENCE EQUATIONS

The difference equations that follow apply to a uniform grid of uniform channel flow. The dimensionless numbers are : $\alpha = U\Delta t/\Delta x$; $\sigma = D\Delta t/\Delta x^2$. The sub- and superscript notation is $C_j^n = C(j\Delta x, n\Delta t)$, where C is concentration.

Scheme A QUAL1 Scheme

$$-C_{j+1}^n + (1+\alpha+2\sigma)C_j^n - (\alpha+\sigma)C_{j-1}^n = C_j^{n-1}$$

This scheme is unconditionally stable.

Scheme B Stone and Brian Scheme

$$(2+3\alpha-6\sigma)C_{j+1}^n + (8+12\sigma)C_j^n + (2-3\alpha-6\sigma)C_{j-1}^n$$

$$= (2-3\alpha+6\sigma)C_{j+1}^{n-1} + (8-12\sigma)C_j^{n-1} + (2+3\alpha+6\sigma)C_{j-1}^{n-1}$$

This scheme is unconditionally stable.

Scheme C QUICKEST Scheme

$$C_j^n = \left[\tfrac{\alpha}{6}(1-\alpha)(\alpha-2)+\sigma(1-\alpha)\right]C_{j+1}^{n-1} + \left[(1-\tfrac{\alpha}{2})(1-\alpha^2)+\sigma(3\alpha-2)\right]C_j^{n-1}$$

$$+ \left[\alpha(1-\tfrac{\alpha}{2})(\alpha+1)-\sigma\ (3\alpha-1)\right]C_{j-1}^{n-1} + \left[\tfrac{\alpha}{6}\ (\alpha^2-1)+\sigma\alpha\right]C_{j-2}^{n-1}$$

The stability region for this scheme is shown on Fig. 8. If $\alpha = 1$ no downstream boundary condition is required.

Scheme D LAMBDA Scheme

$$-\theta\sigma C_{j+1}^n + (1+2\theta\sigma)C_j^n - \theta\sigma C_{j-1}^n = \bar{\theta}\sigma C_j^{n-1} + (1-2\bar{\theta}\sigma)C_{j-1}^{n-1} + \bar{\theta}\sigma C_{j-2}^{n-1}$$

The grid for this scheme is constructed subject to the constraint that $\alpha = 1$. The parameter $\theta$ is a time-weighting coefficient between 0 and 1, and $\bar{\theta} = 1-\theta$. For $\theta = 0$ the difference equation is explicit, stable for $\sigma \leq \frac{1}{2}$, and no downstream boundary condition is required. For $\theta=1$ the difference is fully implicit and unconditionally stable.

DISCUSSION

T R Healy Is the modelling of advection as given in the paper really applicable to two layer flow in estuaries? Most NZ estuaries at their headwaters or in freshes exhibit two layer flow, and this might be a major problem for the numerical modellers in the future.
McBride The models discussed in this paper are all steady and one-dimensional in space. Modelling unsteady and/or two-dimensional stratified flow in estuaries is a much more difficult problem, and one beyond the experience of this author. Development of a full understanding of modelling one-dimensional flow should be the starting point for the modelling of two-dimensional flows.
A G Barnett You have considerable difficulty modelling the stability at the interface in two layer flow.
M J O'Sullivan One of the greatest difficulties is when two layer flow breaks down to become one layer and vice versa.

M E U Taylor Have you tried any profiles other than the "Top Hat" to test the models and how did the various models perform with these profiles?
McBride Test problems were tried with a boundary condition variation and with a "Dunces Cap" (like the "Top Hat Test" but with width $2\Delta x$). A test with an inflow imposed at an interior node was also tried. The "Dunces Cap" and boundary condition change tests gave qualitatively the same results as the "Top Hat Test". The inflow test revealed that further problems, in the form of false upstream mass transport and oscillations upstream of the inflow node, can occur with schemes that use central differencing for the advection term of the transport equation. This is particularly apparent for schemes which are neutrally stable for pure advection.

Dr A J Sutherland (1) What is the source of numerical dispersion and could it be minimised by greater precision in the calculations?
(2) Is Fig. 3 a fair test of the schemes in that $\alpha$ has been optimised for schemes C and D and not for schemes A or B?
McBride (1) Numerical dispersion is not attributable to rounding errors, and thus cannot be decreased by increasing machine precision. Rather it is attributable to the scheme truncation error. The truncation error is the "amount" by which a difference equation, defined on a particular computation grid, departs from the differential equation for the same problem. If this "amount" contains a second order space derivative, then one speaks of the scheme possessing numerical dispersion. The coefficient of this derivative (which will be in terms of U, $\Delta t$ and $\Delta x$) is the scheme "numerical dispersion coefficient". A truncation error analysis can be performed quite simply by attempting to recover the differential equation from the difference equation, using Taylor's series. Details are in most numerical analysis texts.
(2) Scheme B has been tested for a range of $\alpha$, as in Fig. 7. The effect of varying $\alpha$ on the performance of scheme A has been tested, but is not shown in the paper. Such testing shows that as $\alpha$ is decreased, the numerical dispersion decreases a little. This may be expected from the decreased numerical dispersion coefficient (given in the paper as $\frac{U}{2}(\Delta x + U\Delta t)$). This corresponds to a false dispersion number of $\frac{\alpha}{2}(1 + \alpha)$. Thus halving $\alpha$ from 1 to $\frac{1}{2}$ reduces this dispersion number from 1 to 3/8. However, to run the "Top Hat Test" for the latter case requires that the number of time steps be doubled. The net effect of reducing $\alpha$ and increasing the number of time steps is a small decrease in numerical dispersion.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre November 17-18, 1981*

# CLASSIFICATION OF NEW ZEALAND INLETS: SHORT-CIRCUITING ENVIRONMENTAL ASSESSMENT OF NEW ZEALAND COASTAL INLETS

*R A Heath & K R Grange, N Z Oceanography Institute, DSIR, Wellington*

## ABSTRACT

In Heath (1976) it was reasoned that it was not possible to study immediately all the New Zealand coastal inlets. Rather it was advocated that detailed studies should be carried out in one inlet within each group of inlets, the groups having been classified according to their physical properties. This idea is based on the premise that inlets can ultimately be classified within schemes such as the theoretical classification scheme of Hansen and Rattray (1966).

Our research in New Zealand estuaries, while in the long term hopefully leading to the goal of understanding the inlet environment, has also lead us to other questions. Accepting that much present-day research in New Zealand inlets is problem oriented involving environmental assessment with strong time and resource constraints, is there some way of short-circuiting this environmental assessment?

Our present (we are still finalising a manuscript which is at present in draft form) conclusions are that the main factors which control the inlet are : residence time of the inlet waters; the tidal excursion of coastal waters; the tidal energy dissipation; the freshwater inflow; width and depth.

From these factors a considerable amount can be predicted for the inlet biology and geology and therefore they can be used to indicate if there is likely to be an "environmental problem".

## REFERENCES

Hansen, D.V.: Rattray, M. 1966: New dimensions in estuary classification. Limnol. Oceanog. Vol. 11, No. 3, 319-326.

Heath, R.A. 1976: Broad classification of NZ inlets with emphasis on residence times. NZ Journal Marine & Freshwater Res., Vol. 10, No. 3, 429-444.

## DISCUSSION

J C Rutherford Were the residence times in Pelorus Sound estimated from tidal prism calculations and from salinity measurements comparable?
Heath The tidal prism method gives a value of 18 days, the salinity defect method 21 days. However, the close agreement is probably misleading because the presence of an internal tide probably decreases the residence time of the lower layer and makes the validity of the tidal prism method doubtful. The calculations based on Ketchum's modified tidal prism method for the near surface layer are probably more valid.

A G Barnett Have you considered a numerical model of the Pelorus Sound system?
Heath Not really. What would be needed is a model capable of changing the density structure with time after a storm event.

B L Williams Could you expand on your approach referred to in the abstract for the forthcoming paper on the short circuiting strategy for inlet environmental assessment?
Heath What we are saying is that people need to look carefully at broad physical indicators to see if there is an environmental problem.

B L Williams

Do you consider there is an advantage in 'physical' scientists and 'biological' scientists working alongside each other on estuarine problems?

Heath Yes very much so, for the ultimate problem rests with the biological system which is closely linked with physical and geological processes.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre November 17-18, 1981*

# THE USE OF FLUORESCENT TRACERS TO EXAMINE MIXING IN LUCAS CREEK, UPPER WAITEMATA HARBOUR

*K E Parnell, Geography Department, University of Auckland*

## INTRODUCTION

Bearing in mind that this meeting is, in essence, a workshop, this paper, as well as presenting results of a field experiment which was undertaken during March 1981 in Lucas Creek, reviews the fluorometric technique for use in surface water tracing. Special emphasis will be given to the effect of the morphology of Lucas Creek on mixing. The results in this paper are time specific for the conditions prevailing in Lucas Creek at the time the experiment was undertaken.

## THE STUDY AREA AND THE UPPER WAITEMATA HARBOUR CATCHMENT STUDY

The Albany Basin is 36 square km in area within the Upper Waitemata Harbour Catchment (Fig. 1.). Lucas Creek is a 6.5 km long estuarine arm of the Upper Waitemata Harbour with principle fresh water inputs from Lucas Stream, Oteha Stream and Te Wharau Stream. The head of tide is at Albany with well defined high tide limits on both Lucas Stream and Oteha Stream arms.

Following an indication in the Waitemata Harbour Plan (AHB and ARA 1975) that the Hobsonville Inlet and Lucas Creek were sensitive to pollution, and, therefore, that there was a need for further study to ascertain whether further land development would lead to serious environmental degradation, the Upper Waitemata Harbour Catchment Study (UWHCS) was promoted by the Auckland Regional Water Board. It was established as:

"A regional study with national significance designed to provide answers to pressing local problems of land and water management ... but with a responsibility to develop principles or guidelines for rational land development and water quality control that may be applied on a national level."

The general terms of reference for the study are:

"To carry out a study of the land and water resources of the Upper Waitemata Harbour and its catchment in order to facilitate the integrated management and protection of these resources and to promote their enhancement for the benefit of the local, regional and national communities."

The study focuses on six fields, these being

i Land resources
ii Fresh water hydrology
iii Estuarine hydrology and sedimentology
iv Water quality studies
v Biological studies
vi Guidelines formulation

The experiment reported here was undertaken in conjunction with a tidal gauging being conducted by the District Hydrology Group, MWD, Auckland.

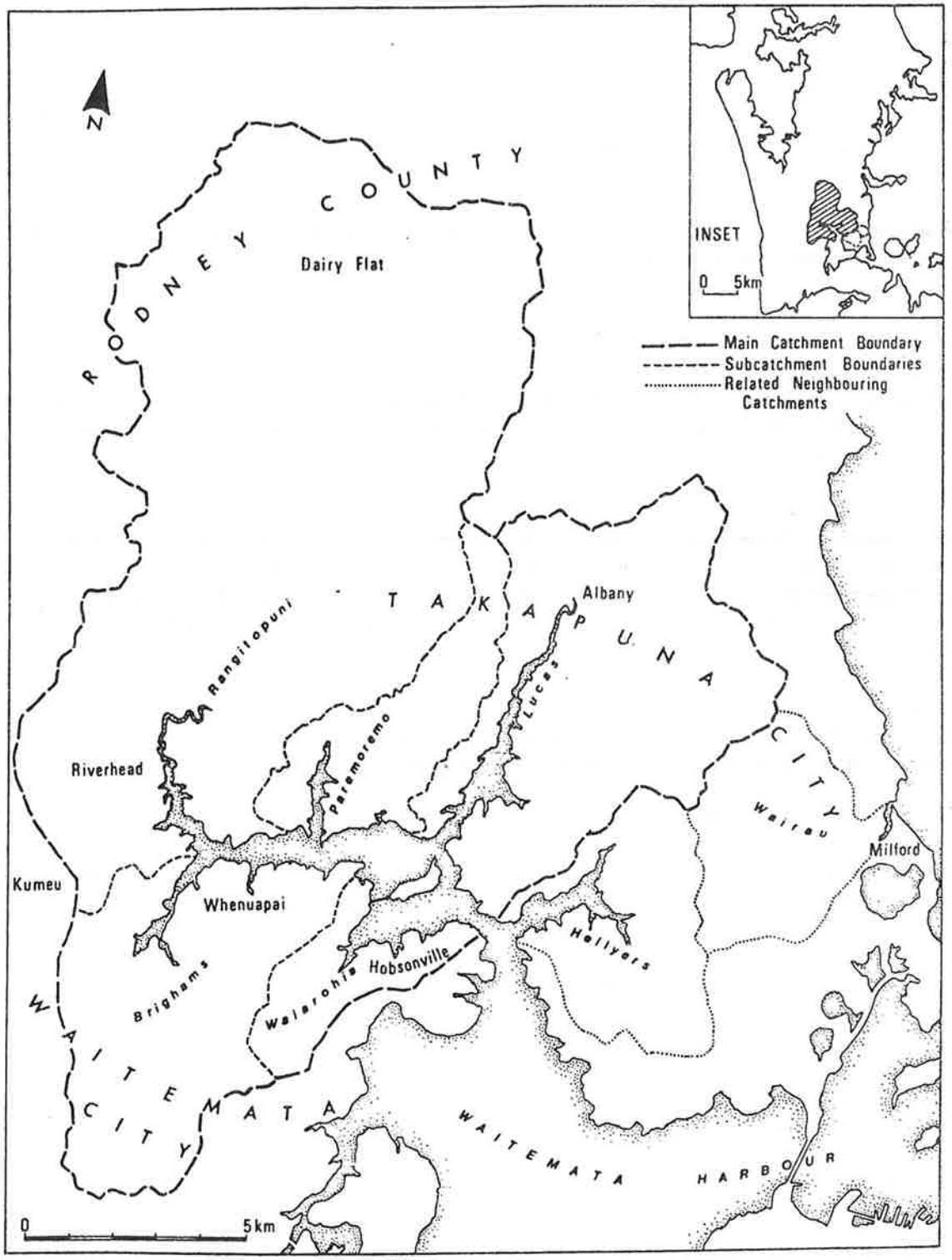



Figure 1 The Upper Waitemata Harbour Catchment.

## THE FLUOROMETRIC TECHNIQUE IN ESTUARINE RESEARCH

Fluorescent tracers have been proven in a number of dispersion experiments in the last 20 years (Pritchard and Carpenter 1960; Warner and Smith 1980). They are relatively inexpensive and both easily handled and analysed. They were chosen as the most practical tracers for the Lucas Creek study because there appeared to be no insurmountable problems with the tracer properties or appropriate analytical techniques.

An introduced tracer must be non-toxic to aquatic life. Smart and Laidlaw (1977) review the toxicity of fluorometric tracers and conclude that: "concentrations sufficiently high to be a problem are so transient under normal field applications because of rapid dilution following injection, that dye will not cause any ill effects to aquatic life."

Fluorometric substances absorb light energy causing excitation of some electrons and the emission of light energy at a higher wavelength. Dyes have different combinations of excitation and emission spectra, which makes the separation of some dyes possible.

Filter fluorometers utilise this property to give a relative measure of the intensity of light emitted by a sample containing a fluoroscent substance. Quantitative analysis is possible by calibration of the fluorometer with known concentrations of dye. Filters are fitted to the fluorometer which only allow specific wavelengths to pass, these being chosen as close to the absorption and emission peaks as possible. The most commonly used fluorometer is the Turner model III (Turner and Associates 1974). Wilson (1968) gives a detailed description of analytical techniques.

A number of factors which affect fluorescence must be considered in determining the appropriate tracer for use in a particular situation. The important factors are temperature, salinity, pH, background natural fluorescence levels, adsorption, photochemical decay and minimum detectable limits.

Fluorescent tracers currently available fall into three ranges of absorption-emission spectra, including the optical brightners which were rejected for this experiment because they are known to decay very rapidly in sunlight and because natural background levels are very high. Basic data for the dyes in the two ranges are given in Table 1.

TABLE 1 Excitation and emission maxima of the dyes considered and filter combinations

| Range | Common Name | CI Number | Dye Colour | Maximum Excitation (nm) | Maximum Emission (nm) | Primary Filter (Turner Code) | Secondary Filter (Turner Code) |
|---|---|---|---|---|---|---|---|
| Orange | Rhodamine B | 45170 | Basic Violet 10 | 555 | 580 | 110-832 | 110-833 |
| | Rhodamine WT | N/A | N/A | 555 | 580 | 110-823 | 110-833 |
| Green | Fluorescein | 45350 | Acid Yellow 73 | 490 | 520 | 110-815 | 110-818 |
| | Lissamine (FF) | 56205 | Acid Yellow 7 | 420 | 515 | 110-813 | |
| | Pyranine (Conc) | 59040 | Solvent Green 7 | 455 | 515 | | |

Although Feuerstein and Shelleck (1963) reviewed some of the properties of three tracer dyes, the first comprehensive statement was produced by Smart and Laidlaw (1977). The findings of these studies were taken as a basis from which suitable tracers were chosen for further examination.

In natural waters background fluorescence varies considerably. There is little background fluorescence in the orange range. However, the green range has high background fluorescence making low concentration quantitative analysis difficult. Feuerstein and Shelleck (1963) believe that the green range dyes can only be used as quantitative tracers in water of the highest quality.

Minimum detectable limits are based in part on background levels. Minimum detectable limits using the Turner III fluorometer and high sensitivity door are given in Table 2 (Smart and Laidlaw 1977, p. 18), calculated as 10% in excess of background or 1 scale unit, whichever is the greater, in deionized water.

TABLE 2 Minimum detectable limits

| Dye | Minimum detectability µg/l |
|---|---|
| Rhodamine B | 0.010 |
| Rhodamine WT | 0.013 |
| Fluorescein | 0.29 |
| Lissamine | 0.29 |
| Pyranine | 0.087 |

The fluorescence of Rhodamine WT and Rhodamine B is affected significantly by temperature. The green range dyes are not significantly affected. Standardisation can be achieved by allowing samples to stand in a room of constant temperature for some hours.

With the exception of Pyranine, all dyes are stable within the pH range of all but highly polluted estuarine water.

Smart and Laidlaw (1977) report that none of the green range dyes is affected by salinity. Rhodamine B is reported to be only slightly affected. However, two separate experiments on Rhodamine WT showed widely differing amounts of suppression of fluorescence by salinity. One experiment showed almost 100% suppression at 18 g/l chlorosity after a long period, whereas the other showed only 8% suppression. No explanation is offered for this difference except that the experiments "were conducted several years apart in different laboratories and on different batches of dye" (Smart and Laidlaw 1977, p. 21).

Rhodamine B is found to be of little use as a quantitative tracer because of its very poor resistance to adsorption, principally because of its cationic nature. Pyranine and Lissamine have high resistance to adsorption on both organic and inorganic substances. Rhodamine WT and Fluorescein have a moderate resistance to adsorption on both organic and inorganic material. Because of the relatively clean nature of estuary waters, if an anionic tracer is used little adsorption will occur (Smart and Laidlaw 1977, p. 27). Scott *et al.* (1969) conducted experiments on Rhodamine WT concluding that in the presence of fine material any loss of fluorescence will take place in a short time, that loss of fluorescence is independent of dye concentration and that loss of fluorescence may be appreciable for concentration of sediment greater than 500 mg/l.

Photochemical decay is determined in part by the amount of water sunlight can reach. Therefore, in shallow water there will be more decay than in a similar volume of deep water. Decay rates for Pyranine and Fluorescein are high, but: "the orange fluorescent tracers and Lissamine FF exhibit low photochemical decay rates such that no correction will be required for tests of up to one week in duration." (Smart and Laidlaw 1977, p. 34).

The decay of Rhodamine WT has been determined empirically by Warner and Smith (1980) using the segmented estuary approach. This was possible because the systems were large and no dye escaped. The studies suggested a decay rate of approximately 3% per day. As well as photochemical decay, possible causes of this decay are adsorption, salinity suppression of fluorescence and minimum detectable limits being reached at the dye front.

Initial evidence from the literature seemed to preclude all dyes for quantitative surface water tracing with the exception of Rhodamine WT and Lissamine, although Rhodamine B, Fluorescein and Pyranine were concluded to be suitable inexpensive tracers for a semi-quantitative pilot study.

It is recognised that photochemical decay and salinity may affect fluorescence, but there has been little quantitative or comparative research into these factors. The effect, if any, of salinity on photochemical decay has not been explored. It is recognised that the following experiments do not accurately simulate field conditions, but it is believed that the implications are applicable. Rhodamine B, Rhodamine WT, Pyranine and Lissamine were examined. Fluorescein was eliminated because of its known excessively fast photochemical decay rate.

Two experiments were conducted. Firstly, the dyes of interest, each at three different concentrations were diluted in both fresh and salt (35 ppt) water. Each was placed in sunlight, with a further sample of each dye at each concentration kept in total darkness as control. Secondly, the dyes at one concentration were diluted in water with eight different salt (NaCl) concentrations. These solutions were kept in darkness and tested for any reduction in fluorescence.

Silica-glass test tubes were used in the decay experiment. As a necessary preliminary to the experiment, a UV spectrum was run on the test tubes. It was found that the tubes passed almost all wavelengths of light reaching the earth's surface (above 292 nm) (Fig. 2). A silica-glass test tube containing deionized water was also tested. The results are given in Fig. 3. It is clear that at all wavelengths greater than 360 nm, only 50% of light is transmitted through 1 cm thickness of water (the diameter of the tube). Consequently, in perfectly clear water, 50% of any photochemical decay will take place in the upper 1 cm of water. The implications of this are encouraging for the use of dye in estuarine research as photochemical decay is likely to be non-existent in most of the water.

Figures 4 and 5 show photochemical decay of dye in both fresh and salt water for one of the concentrations examined. A general pattern is obvious. Salt water suppresses photochemical decay. The effect is less with Lissamine and Pyranine than with Rhodamine dyes, but is still significant. These results are encouraging for the use of dyes in sea water. For example, Rhodamine WT decays to 50% initial fluorescence after 180 hours in sunlight (mixed weather conditions in spring). Therefore in the top 1 cm of water, with light reaching the sample from all angles, there would be a decay rate of only 4.5% per day. As decay only occurs in the top few centimetres, and if there is no concentration of dye in the top layers (as might be expected in a highly stratified estuary), total photochemical decay would be negligible.

Rhodamine WT and Lissamine were found from the laboratory experiment to be considerably more stable in sunlight than Pyranine and Rhodamine B.

Because any suppression of fluorescence by salinity would not be readily distinguishable from photochemical decay, therefore, under the experimental conditions, the slowing of photochemical decay by the presence of salt must be more significant than suppression of fluorescence by the presence of salt.

It has been noted that Smart and Laidlaw (1977) presented conflicting evidence on salinity suppressing fluorescence of Rhodamine WT. Similar conflicting evidence was found in my laboratory experiments. The first experiment using eight salt concentrations showed suppression (Table 3) falling between the two findings of Smart and Laidlaw (1977). However, an experiment using the batch of dye ultimately used in the field test showed a different level of suppression (Table 4). No explanation can be offered for these differences. It is apparently necessary to test each batch of dye to be used in salt water research.

The literature, confirmed by further laboratory experiments, indicated that Rhodamine WT and Lissamine, were the only ones that could be reliably used simultaneously for long term quantitative analysis. Thus, they were selected for use in the main experiment. Rhodamine B and Fluorescein, being cheap, were considered suitable for a pilot study since quantitative analysis was not required, and their rapid decay characteristics enabled the main experiment to be conducted soon after the pilot study without fear of cross-contamination.

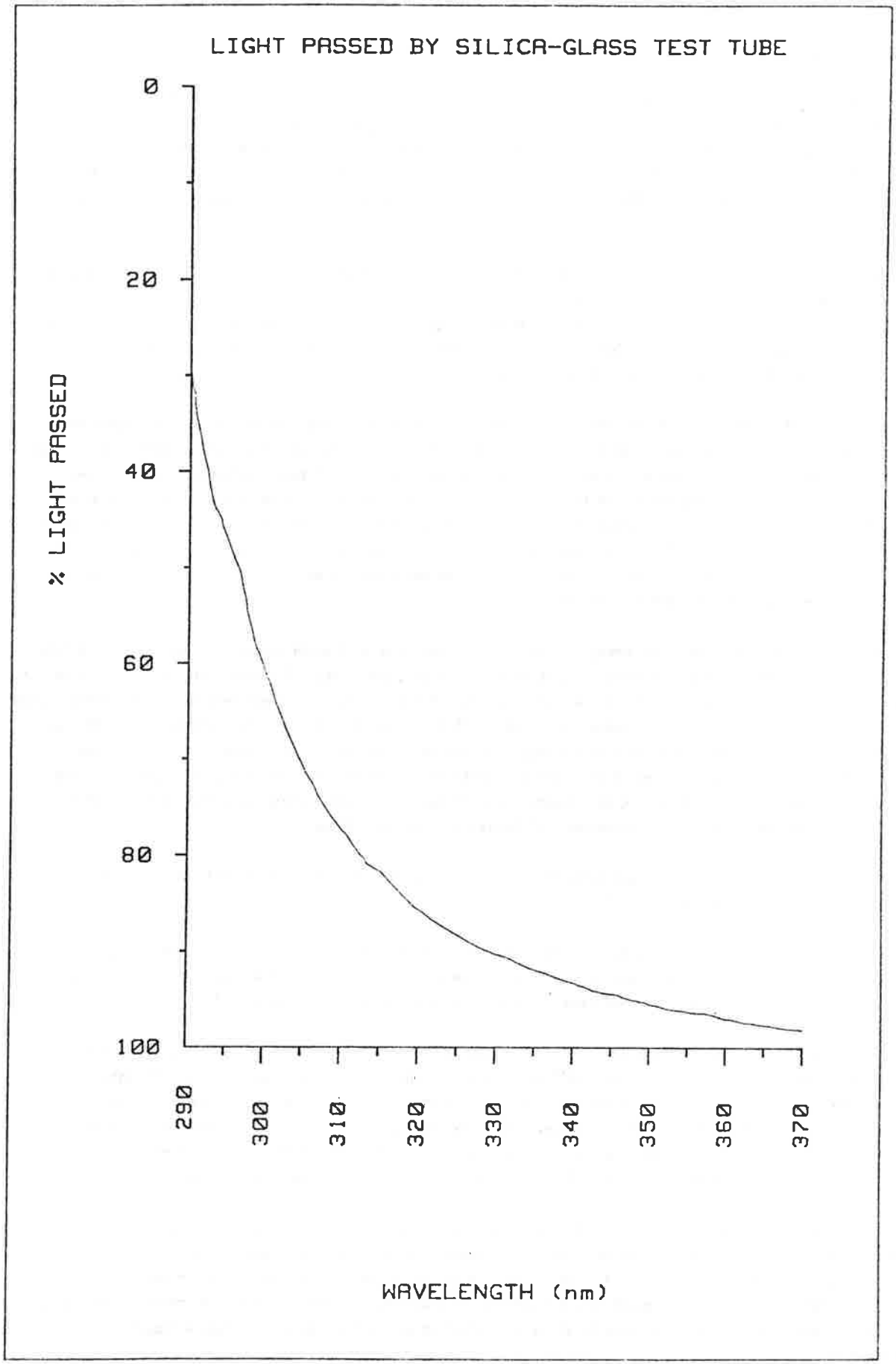



Figure 2 U.V. Spectrum showing the Light passed by a Standard Fluorometer Cuvette

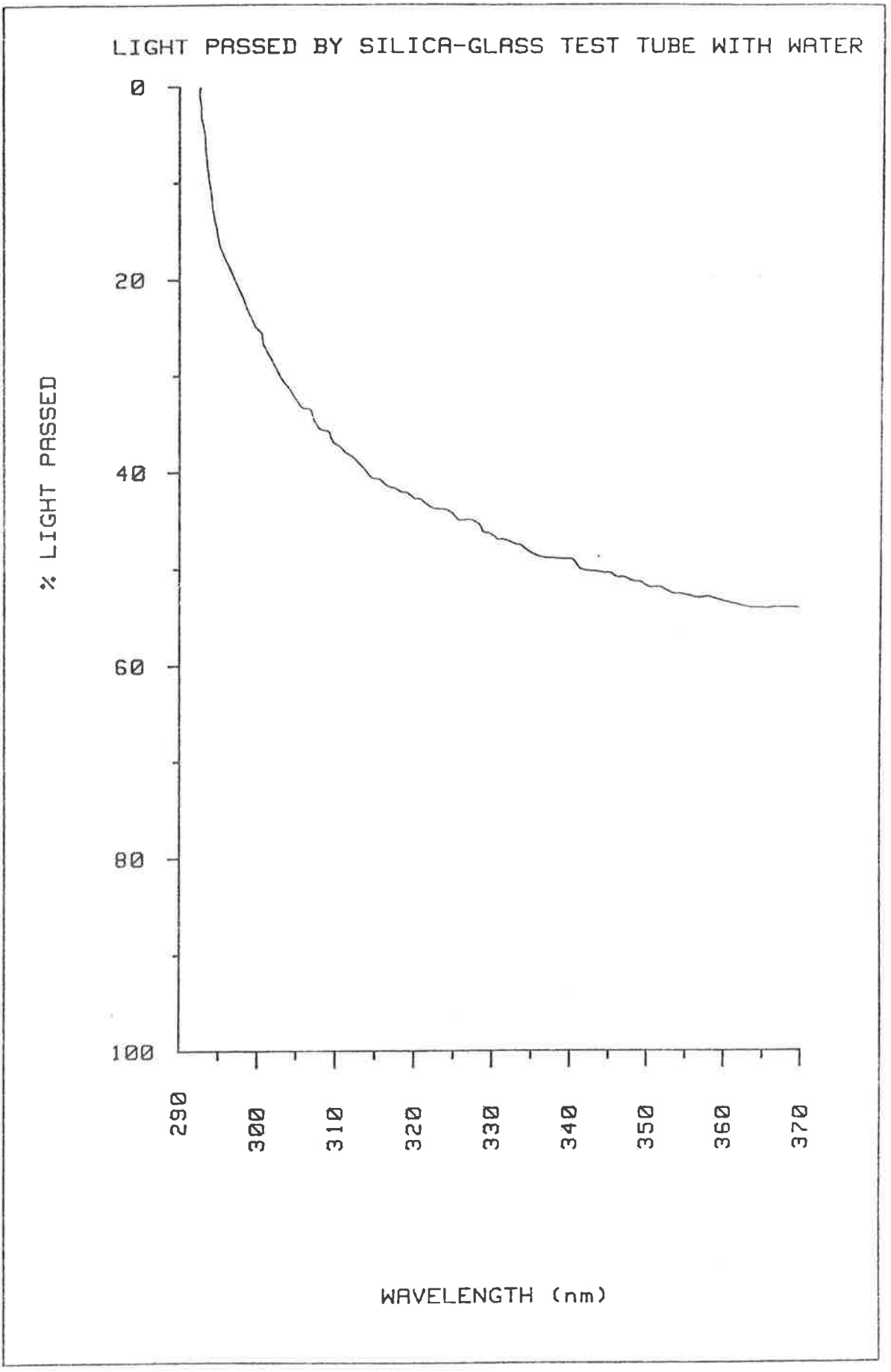



Figure 3 U.V. Spectrum showing the Light passed by a Standard Fluorometer Cuvette filled with Deionised Water

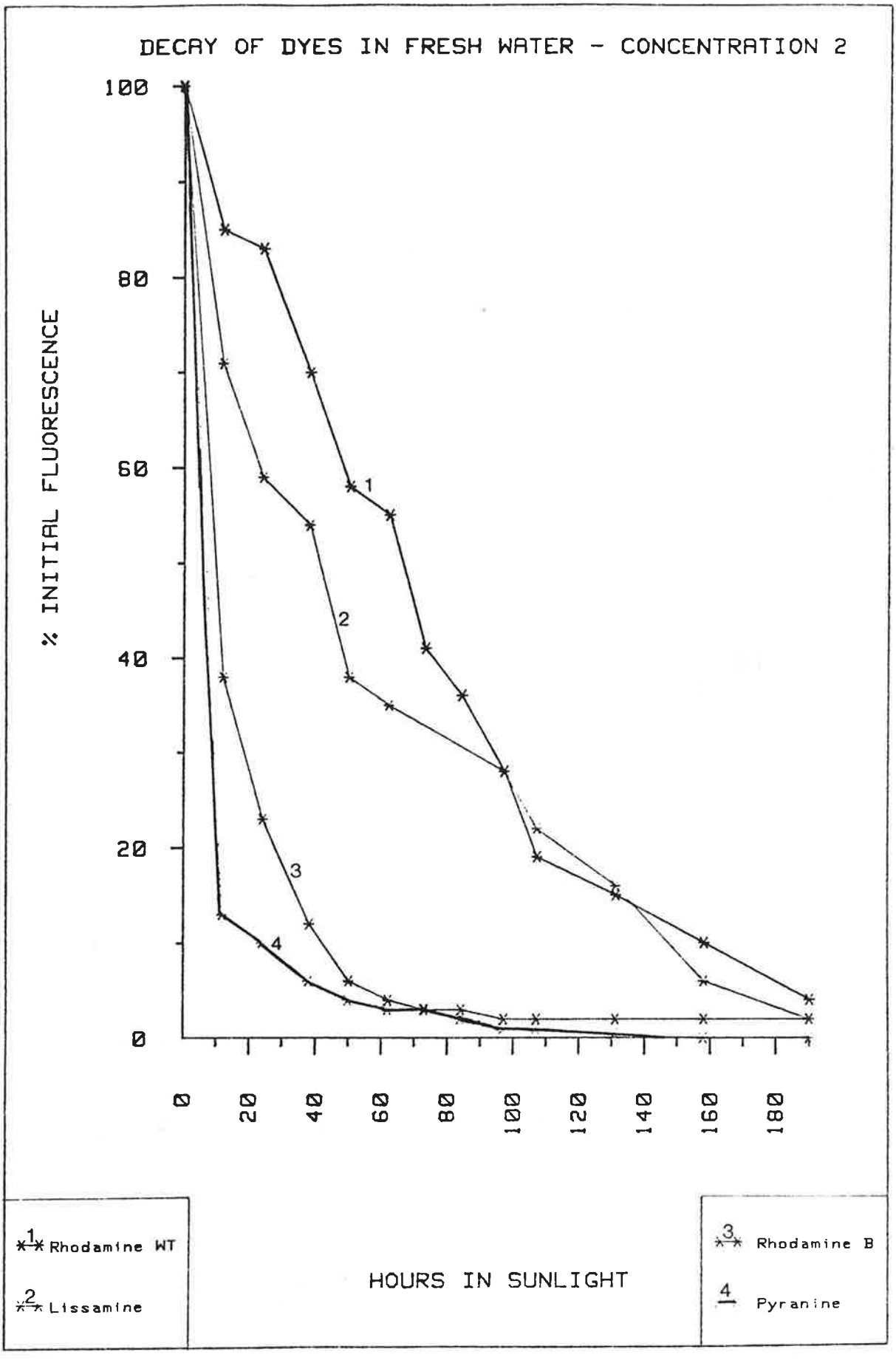



Figure 4 Decay of Dyes in Sunlight - Concentration 2

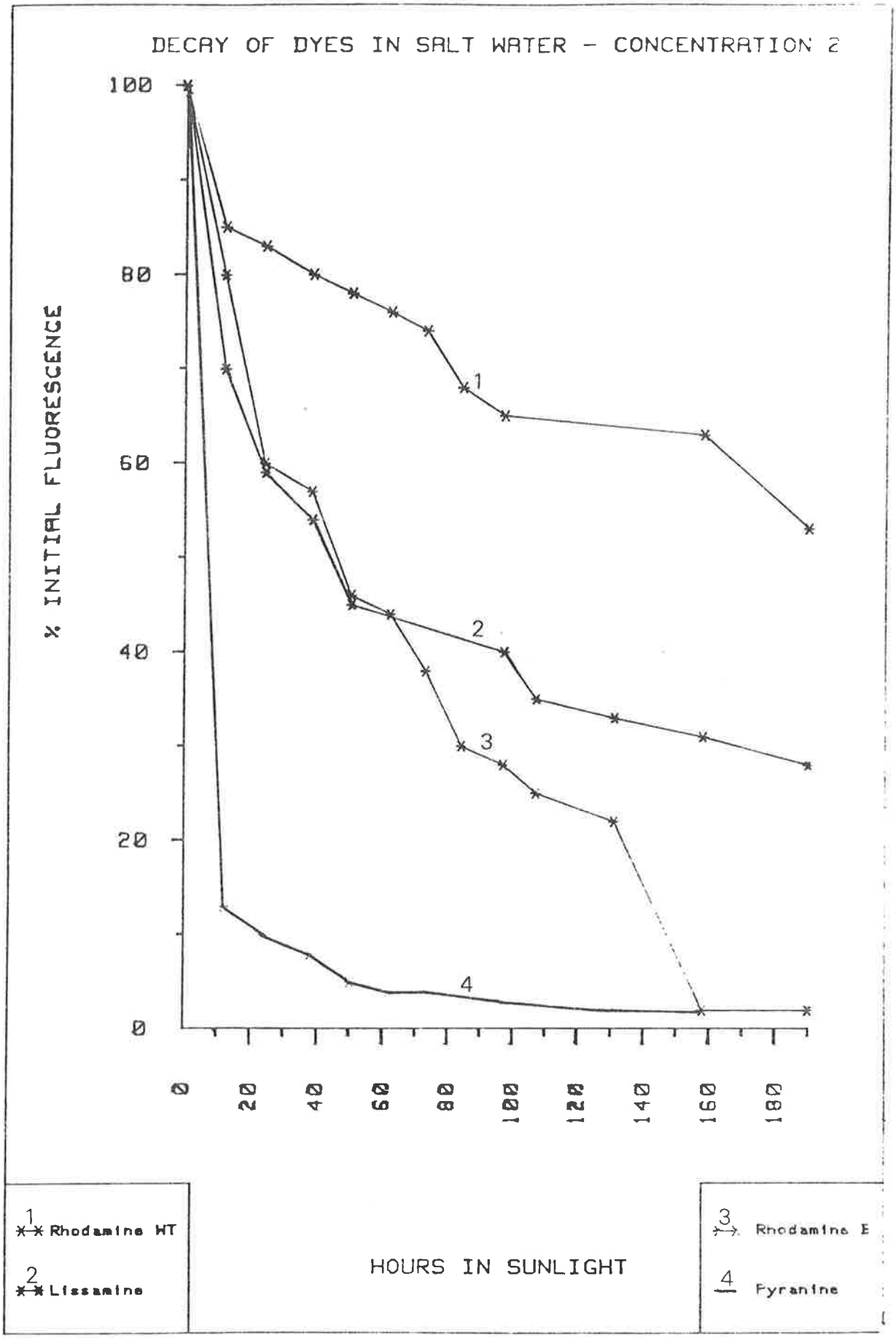

DECAY OF DYES IN SALT WATER - CONCENTRATION 2
% INITIAL FLUORESCENCE
100
80
60
40
20
0
0
20
40
60
80
100
120
140
160
180
1
2
3
4
HOURS IN SUNLIGHT
1 Rhodamine WT
2 Lissamine
3 Rhodamine B
4 Pyranine

Figure 5

TABLE 3 Experiment 1 : Effect of Sodium Chloride on Rhodamine WT at 0.25 µl/l

| | (% initial fluorescence) | | | | | |
|---|---|---|---|---|---|---|
| | Salt concentration g/l | | | | | |
| Time | 0.355 | 1 | 3.55 | 10 | 20 | 35.5 |
| 18½ hrs | 100 | 100 | 100 | 100 | 96 | 90 |
| 71½ hrs | 100 | 100 | 100 | 97 | 94 | 88 |
| 286 hrs | 100 | 100 | 97 | 94 | 90 | 84 |

TABLE 4 Experiment 2 : Effect of Sodium Chloride at 30 ppt on Rhodamine WT

| | (% initial fluorescence) | | | |
|---|---|---|---|---|
| | Dye concentration | | | |
| Time | 0.1 µl/l fresh water | 0.1 µl/l salt water | 0.01 µl/l fresh water | 0.01 µl/l salt water |
| 24 hrs | 100 | 100 | 100 | 100 |
| 72 hrs | 100 | 100 | 100 | 99 |
| 270 hrs | 100 | 90 | 100 | 94 |

In the course of the field experiments further insights were gained on the use of dyes in estuarine situations. Visual evidence showed that Rhodamine B adsorbed seriously onto all bed and bank materials, whereas Rhodamine WT did not leave any trace, even though concentrations of Rhodamine WT were much higher.

Fluorescein was visible for some hours after its injection. However, Lissamine disappeared almost immediately and was not detected fluorometrically after a few minutes, despite preliminary work that suggested Lissamine was suitable for field use. The reason for this is not known. When dissolving Lissamine powder in water for injection, some crystallisation was seen to occur after a few minutes. This has not been previously recorded and needs investigation.

Whereas both orange range dyes were suitable for their respective functions, there does not appear to be a suitable green range dye for estuarine research.

## FIELD EXPERIMENT DESIGN

The experiment was designed to simulate accurately a continuous inflow into Lucas Creek of a dissolved substance in the fresh water, from Lucas Stream. Injection of the tracer was at Albany because complete mixing of the tracer in the fresh water input was assured as a result of the stream flowing over a 3 m waterfall immediately downstream of the injection point. The bottom of the waterfall is the head of tide.

From 1050 h (high tide) to 1630 h (low tide) on 10 March 1981, 28.33 l of stock Rhodamine WT solution was injected into Lucas Creek at the head of tide, 100 m upstream of the State Highway 1 bridge at Albany. To give a suitable rate of constant flow, the manufacturer's stock solution was diluted to 30% by volume and injected at a rate of 277 ml/min using a constant head injection device, similar to a Mariotte Flask (Church 1974). Stock Rhodamine WT is supplied as a 20% solution by weight, with a specific gravity of 1.15, not 1.19 as reported by Wilson (1968). This was checked using a hydrometer. Therefore, 6.51 kg of Rhodamine WT was injected. On 11 March 1981, at 1130 h (high tide), 6 kg of Lissamine FF was injected at Greenhithe (Fig. 6).

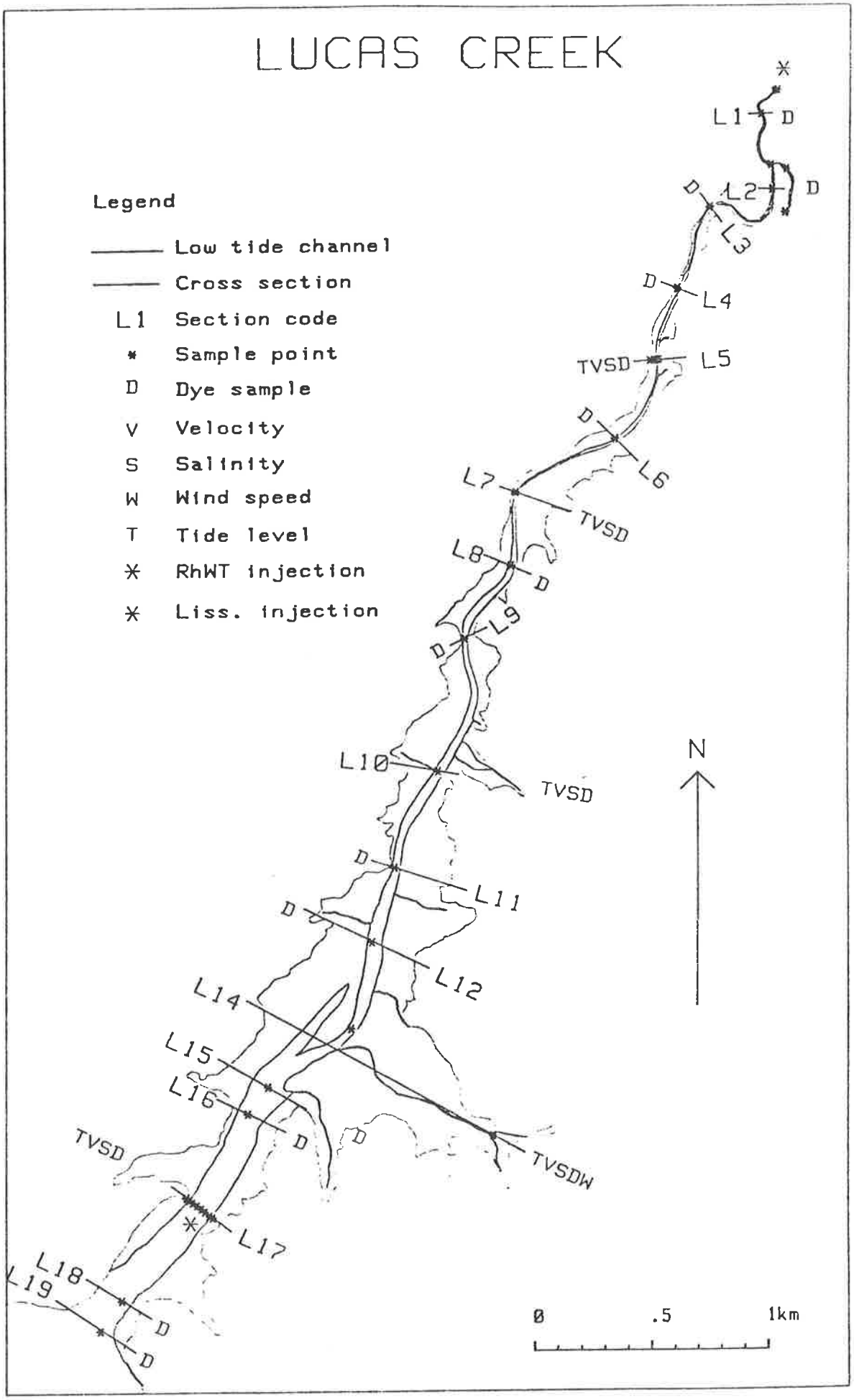



Figure 6 Experimental Design on 11 March 1981

Samples were taken from points on surveyed cross sections. These sites are indicated on Fig. 6. On injection day, and from 12 March to 18 March, water samples were collected periodically, but at least once a day. For the next two weeks samples were collected twice a week, and then two weekly until no trace of the dye was found on 7 May 1981. From 1030 h, 11 March, to 0100 h, 12 March comprehensive data were collected (Fig. 6).

## MIXING IN LUCAS CREEK

### Physical characteristics of Lucas Creek

Lucas Creek is a drowned river valley estuary which extends 6.5 km from the mouth at Greenhithe to Albany, splitting 6.2 km from the mouth into the Lucas and Oteha arms. Notable features are the deep pool at L9, the rock sills between L5 and L4 and upstream of L2, and the large side embayment to the Te Wharau Stream at L14 (Fig. 7).

### Vertical and lateral mixing

Differences in dye concentration in a cross-section can be caused by salinity stratification or by velocity differences, which make water in some parts of a cross-section move faster than elsewhere. The degree of vertical mixing was measured at L5 and L17 with intensive sampling on the surface and at 0.6 of the depth (0.6 D).

At L5 vertical mixing criteria are only important at some stages of tide, because rock sills restrict tidal flow upstream of that point. Significant differences in dye concentration in the vertical at L2 are illustrated in Table 5.

TABLE 5 L2 dye concentration, 1315 h 11 March

| Depth | Concentration µg/l |
|---|---|
| Surface | 136 |
| 1 m | 22 |
| 2 m | 13 |

However, at L5, below the rock sills there was a high degree of vertical mixing (Fig. 8). During the first half of the outgoing tide, surface concentrations are significantly less than concentrations at 0.6 D. Then as the tide goes out, the difference, especially as a percentage difference, becomes small. Similar patterns occur at all vertical profiles in the cross section. It is apparent that the rock sills between L5 and L4 cause considerable mixing. These features result in substantial mixing by turbulence, in a manner similar to rapids in rivers. When the water is deeper there is less mixing. It would be reasonable to expect that mixing would be less if the rock sills were not present.

Differences in the vertical are less on the incoming, than on the outgoing tide. Further mixing, which occurs in the already mixed water after passing L5, makes the water returning even more homogenous in the vertical than on the previous tide. Following this argument, it is apparent that differences in the vertical decrease with each succeeding tide.

At L17 vertical differences in dye concentration follow a clear pattern (Fig. 9). On the outgoing tide concentrations on the surface are greater than those at 0.6 D. On the incoming tide, however, there is a tendency for concentrations to be similar on the surface and at depth, and in some cases, at about mid-tide, higher concentrations occur at 0.6 D (Table 6).

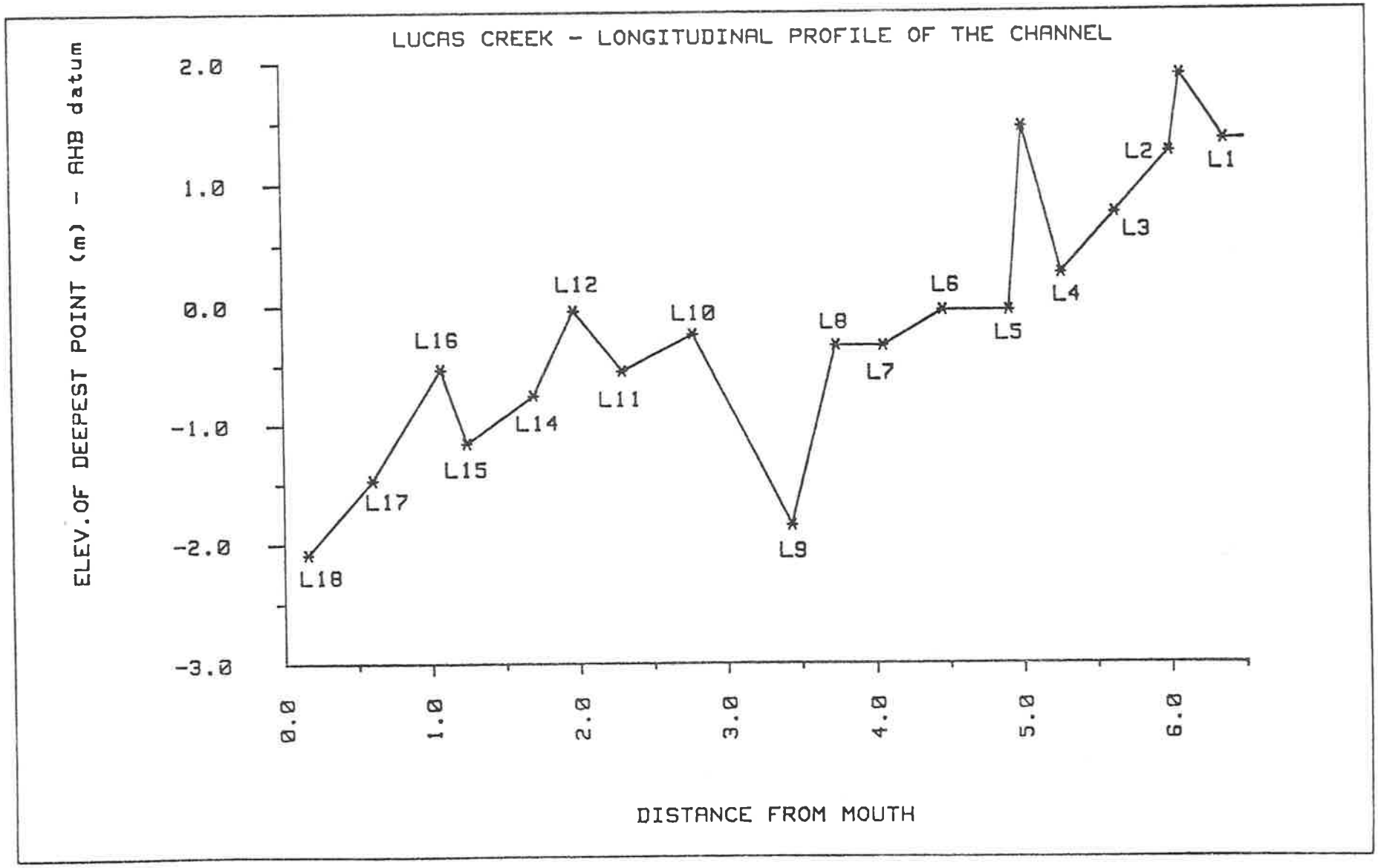



Figure 7 Longitudinal Profile of the Lucas Creek Channel

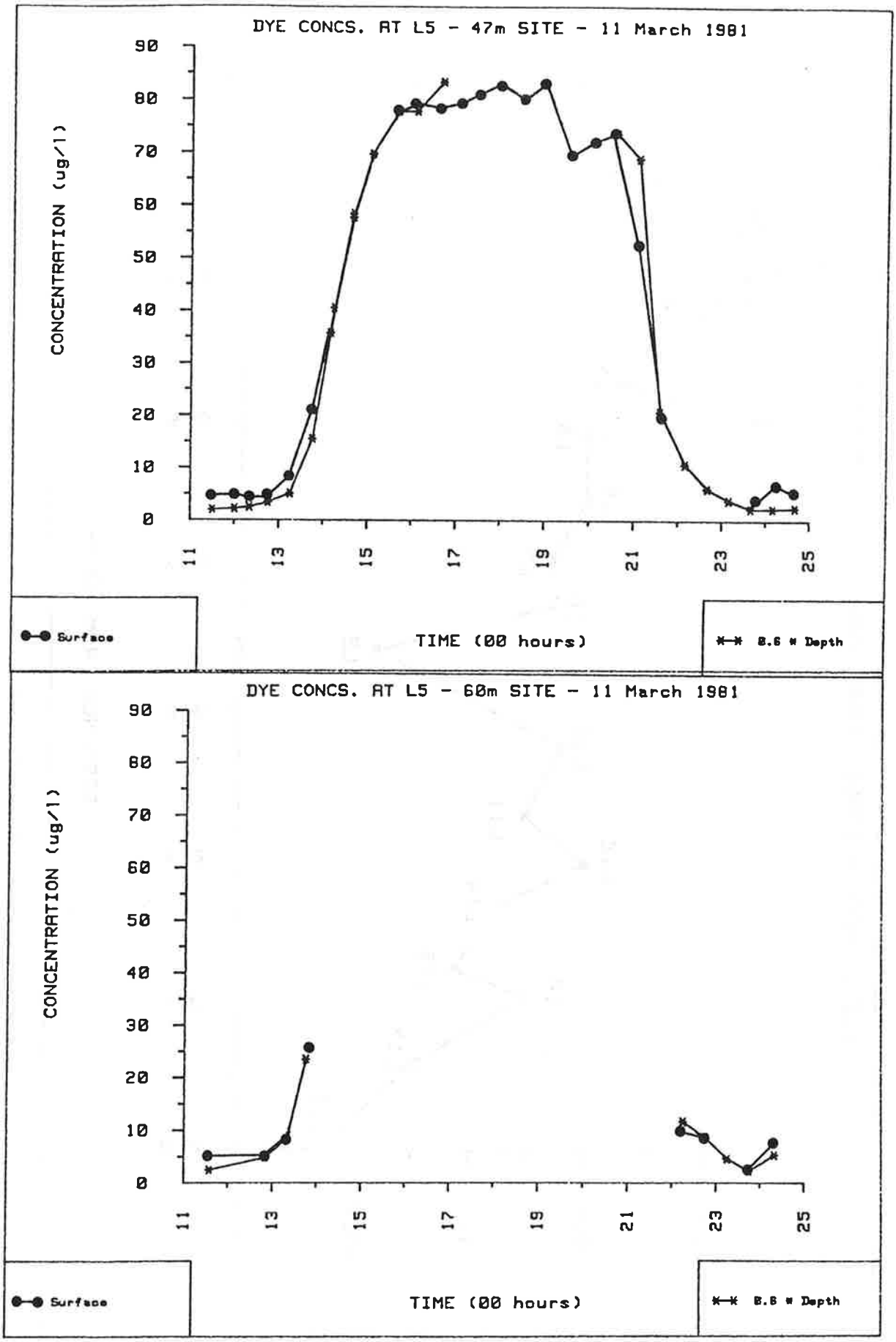



Figure 8 Dye Concentrations at L5 - 11 March 1981

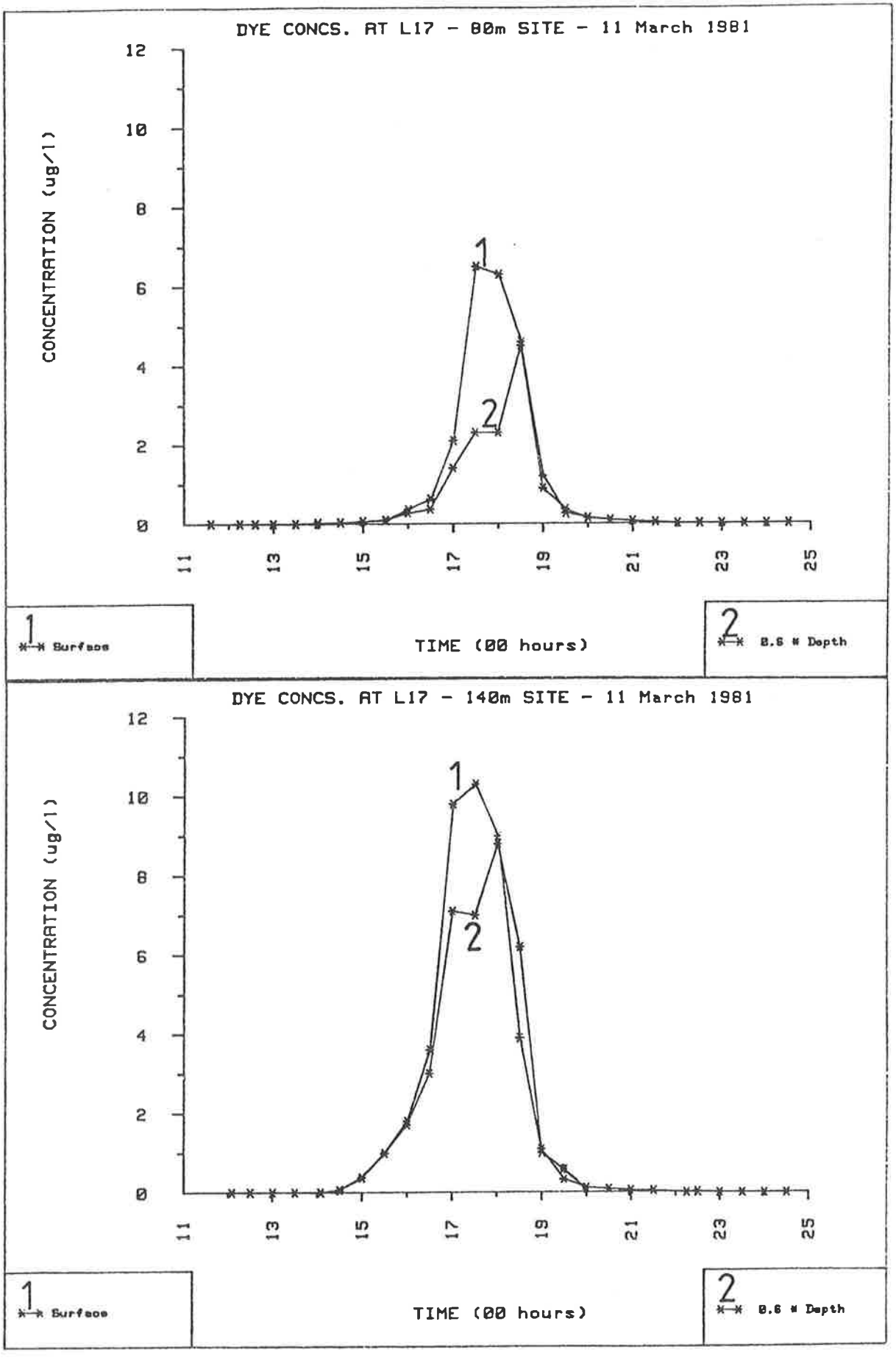



Figure 9 Dye Concentrations at L17 - 11 March 1981

TABLE 6 Dye concentrations at the 100 m vertical, L17 - 11 March

| Time | Concentration at surface (µg/l) | Concentration at depth (µg/l) |
|---|---|---|
| 1735 | 8.81 | 4.81 |
| 1803 | 6.61 | 6.81 |
| 1835 | 3.40 | 4.61 |
| 1903 | 0.30 | 0.36 |
| 1034 | 0.23 | 0.30 |
| 2003 | 0.10 | 0.10 |

On the outgoing tide the velocity profile causes higher concentrations at the surface, because surface velocities are greater, bringing high concentrations of dye from the upper reaches of the estuary. On the incoming tide velocities are again greater on the surface. Therefore, undyed water from the main harbour reaches the surface zone faster than the bottom zone.

The velocity distribution and channel features promote vertical mixing. The rock sills in Lucas Creek encourage extensive mixing at L5 and consequently a high degree of mixing is found in the estuary below L5.

Lateral differences in concentration show a distinctive pattern. To establish conditions at the beginning of the tidal gauging, samples were collected in the channel and near the bank at high tide at 1130 h on 11 March, one tidal day after the dye release began. In the upper reaches of the estuary there were considerable differences between bank concentrations and channel concentrations. Again these differences can be explained in terms of advective mean velocities. Water tends to linger near the banks because the shear effect on the banks is greater and, therefore, velocities in the channel are greater than those near the banks. On the incoming tide, water with lower dye concentrations will have moved faster in the channel than near the banks.

On the day following the dye injection, lateral variation in dye concentration at L5 was small, except at high tide when bank concentrations were higher than channel concentrations. When the 30 m and 60 m profiles were inundated there was a tendency for bank concentrations to lag behind channel concentrations.

This lag is also demonstrated at L10 (Fig. 10). Although the peak concentration at the bank did not reach the concentration attained in the channel, the time a given concentration is reached at the bank was about 30 minutes after than concentration had been reached in the channel.

The time lag between channel and bank concentrations at L17 was greater than that further up the estuary and was especially great on the ebb tide when velocities were highest. The lag is shown to be up to two hours (Fig. 11).

It is clear that differences in advective velocities are the major cause of vertical and lateral differences in tracer concentration after turbulent diffusion, caused by the rock sills, initially mixes the water in the cross section.

Longitudinal mixing

At 1130 h on 11 March most of the dye was located in the section of the estuary above L5. There had been considerable redistribution within this sector with a large quantity of dye moving into the Oteha arm of the estuary. Also, a considerable quantity had accumulated in the relatively deep section between L2 and L4. The visible dye front was between L6 and L7. Below this point there was only a small amount (4%) of dye.

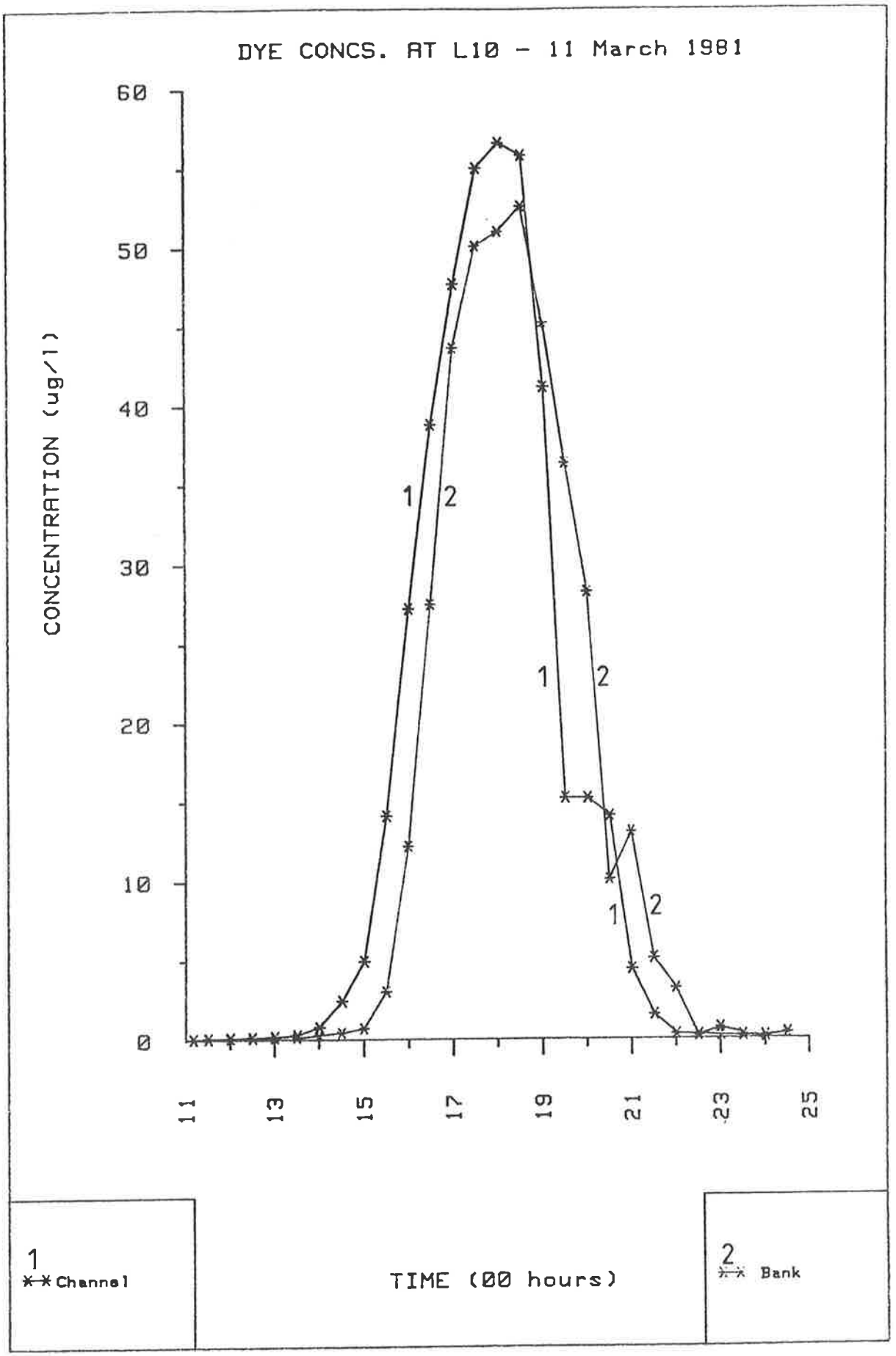



Figure 10 Dye Concentrations at L10 - 11 March 1981

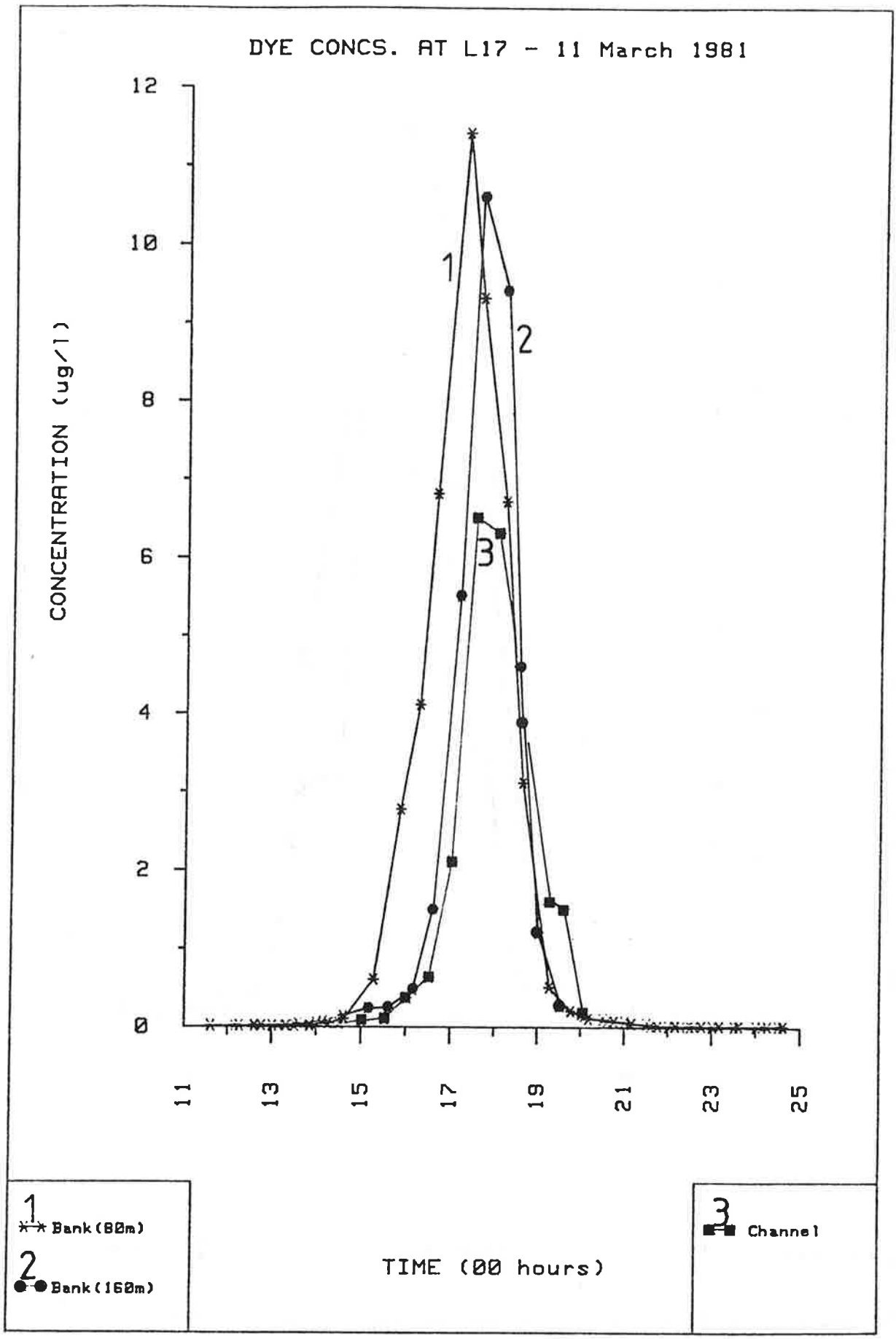



Figure 11 Surface Dye Concentrations at L17 - 11 March 1981

By low water there was a small but significant quantity of dye in the shallow section above L5, but concentrations remained high. Some storage between L2 and L4 is evident. Below L5, channel concentrations of dye ranged from 70 µg/l at L6 to 9 µg/l at the mouth. Four thousand grams of dye remained in the estuary at low tide. At the following high tide 4683 g were accounted for. Although some redistribution occurred, most of the dye (92% of that in the estuary at 2352 h) was upstream of L7.

The total volume of water in the estuary at LW spring tides equals the volume of water upstream of a point between L6 and L7 at HW spring tides. Therefore, if there were no mixing, but merely a replacement occurring as water enters the estuary from LW, the water downstream of the point between L6 and L7 would be water with its origin outside of Lucas Creek. The evidence from both high waters on 11 March shows that a marked change in concentration occurs between L6 and L7. This tends to suggest that the pushing of water in Lucas Creek at LW up to L6-L7 is an important process.

The influence of morphology on the removal of a contaminant

The flushing time, defined simply as "the time required to replace existing fresh water in an estuary at a rate equal to the river discharge" (Officer 1976) is generally calculated for an estuary using one of two methods. The method based on salinity is unsuitable for a harbour situation because much of the freshwater in an arm comes from outside that arm. For example, based on data in Johnson (1979), 40% of the freshwater in Lucas Creek at high tide on 11 December 1978 came from outside Lucas Creek. The other method commonly used, based on tidal prisms, is known to give an excessively short flushing time. This method (and its extension) gives estimates of between 1.1 and 3.2 tidal cycles for the experimental period. The fact that the half life of the dye in Lucas Creek was over 3 tidal cycles, and that dye was detected in the estuary for over 70 tidal cycles, shows that these methods underestimate flushing time considerably. The underestimate is not only a result of inherent errors based on the assumptions, but on the way they are used. The methods are usually applied to the entire estuary, even though more than one dispersion regime may exist.

All estuaries, at some stage in the tidal cycle, have sections which behave as rivers, where outflow from the section equals the inflow of fresh water. In Lucas Creek this section is well defined, being above a series of rock sills between L5 and L4 (see Fig. 7). For about 3½ h either side of high water the section above L5 acts as an estuary with 2-directional flow governed by tidal action and fresh water inflow. For the remainder of the tidal cycle the estuary behaves as a river, independent of tidal action. Thus, these rock sills considerably affect the dispersion of a substance entering the estuary above them.

If a substance is concentrated in the upper reaches of the system at HW, then, on the outgoing tide the substance in solution will travel at the same rate as the tidal flow until the rock bars are exposed, after which flow will be equal to the river inflow (about 30 l/s during the week following dye injection). During the first 3½ h of the ebb tide a proportion of the water in the upper system will pass the rock bars (remembering that a significant proportion of this water would have come from below L5 on the previous incoming tide) and will continue moving downstream until LW. After this, flow will be much slower. Because the nature of the system dictates that a significant quantity of water is ponded between L4 and L2, significant quantities of water from the upper sections with the highest contaminant concentration will remain above L5. An estimate of the volume of water above L5 at low tide is 4100 $m^3$.

When, on the incoming tide, the rock bars become inundated the water ponded behind them will be pushed into the top system, leaving "new" water which has come from further down the estuary in the section immediately above the rock sills. On the next ebb tide most of the water which gets over the sills before discharge falls to that of river input will be the relatively uncontaminated water. Consequently, only a small proportion of dyed water will escape before being pushed back for the cycle to begin again. Therefore, whereas water in the lower end of the system is rapidly dispersed, dispersion is much slower in the upstream end of the system unless there is considerable fresh water input.

Figure 12 shows the concentrations of dye in the pool at Albany from low tide to low tide on 18 March. There was a significant change in concentration between 1725 h and 1755 h. An interpretation of this is as follows.

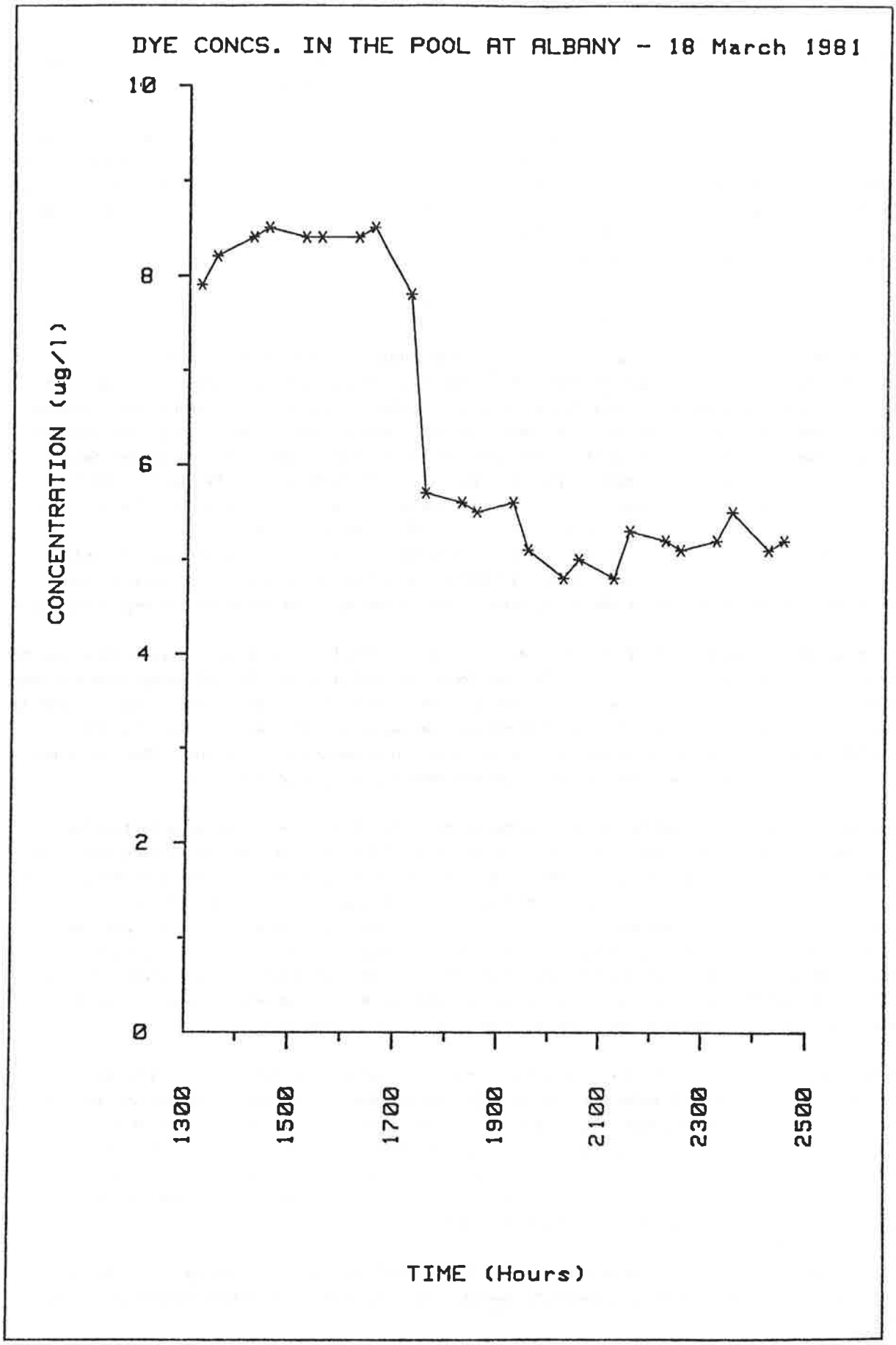



Figure 12 Dye Concentrations in the Pool at Albany - 18 March 1981

During the first part of the flood tide no water gets above L5 because the tide is below the rock sills. Then, for a short period water ponded above the rock sills is returned. Thus, new water is available in the pond for dilution for 2 h at the end of the rising tide. The water returning in this period has a constant concentration implying that it has a common source. From data collected on 17 March it is apparent that the water, or at least a considerable proportion of it, must have come from above L5 because at low tide the L5 concentration was 1.52 µg/l. As the volume of water above L1 at low tide is only about 5 per cent the volume at high tide, most of the water in the pool at high tide is not from the pool at the preceeding low tide. An estimate of the concentration of new water is (5.6 - 0.05 x 7.9)/0.95 = 5.5 µg/l, which must have originated above L5.

It is apparent that two distinct dispersion mechanisms operated in Lucas Creek under the tidal and meteorological conditions prevailing during the experimental period. These mechanisms were produced by two distinct characteristics which coincidentally almost overlap. Firstly, the rock bars cut off the top end of the system from tidal action during a significant proportion of the tidal cycle. Secondly, the amount of water in the estuary at LW is approximately equal to the volume above L6 and HW. While dispersion is rapid below L5 this is not the case above L5.

Two other features of the morphology of Lucas Creek can considerably effect dispersion. Side embayments separate parcels of water and a subtidal bar near the mouth affects the manner in which water leaves Lucas Creek.

The only significant embayment is the estuarine arm of the Te Wharau Stream. Concentrations in this arm never rose above 0.5 µg/l, which is small compared to the concentrations found in the lower section of the estuary at low tide. Because water does not inundate the mudflats of the Te Wharau arm until about half tide, any dye which gets in the embayment would have had its low tide origin outside the estuary, and would, therefore, contain a considerable proportion of undyed water.

At the mouth of Lucas Creek, the mixing and exchange that occurs will affect the amount of Lucas Creek water that leaves the Upper Harbour system, that gets redistributed around the Upper Harbour and that is returned to Lucas Creek.

At L19 the ebb tide distribution of dye in the cross section showed it to be concentrated close to the left bank (Fig. 13). This continued on the flood until between 1935 h and 2055 h (Table 7). While there was a substantial quantity of dye in the water outside Lucas Creek, concentrations on the flood tide were high near the left bank of Lucas Creek. Once the dyed water passed upstream, uncontaminated water followed, and hence lower concentrations occurred near the left bank.

Figure 14 shows where the dye was located in the cross section off the Herald Island jetty. It is clear that near low tide and for a substantial time following, dye was more concentrated near the left bank. Because of the nature of the branching system where Lucas Creek joins the main arm, it is likely that any contaminant located near the left bank off Herald Island will enter Lucas Creek on the flood tide.

TABLE 7 Dye location in the cross section L19 - 11 March

| Time | Left bank | Channel | Right bank |
|---|---|---|---|
| 1905 | 2.8 | 0.29 | 0.09 |
| 1830 | 0.297 | 0.23 | 0.16 |
| 1935 | 0.29 | 0.06 | 0.09 |
| 2055 | 0.09 | 0.13 | 0.16 |

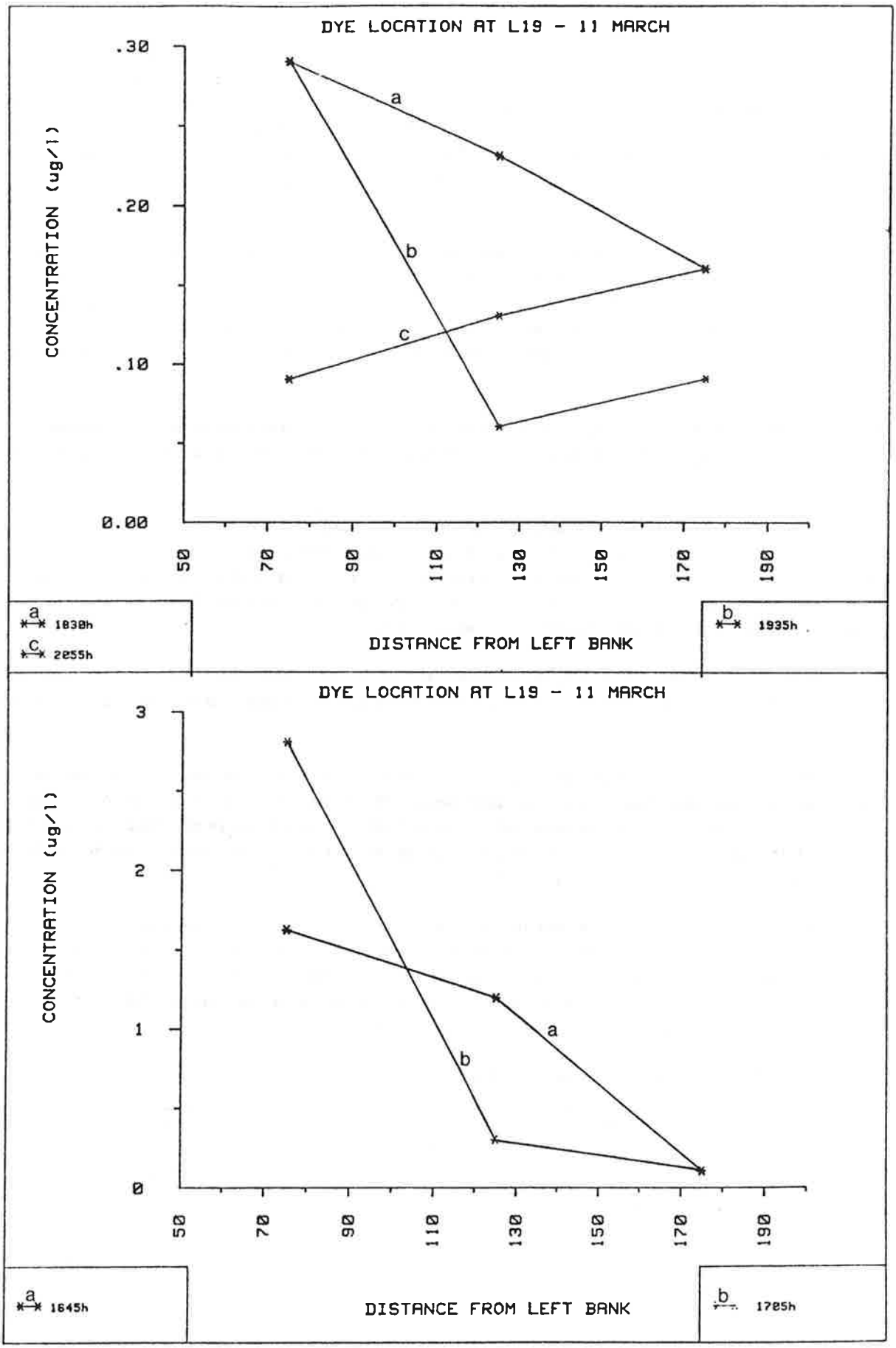



Figure 13 Dye Location at L19 - 11 March 1981

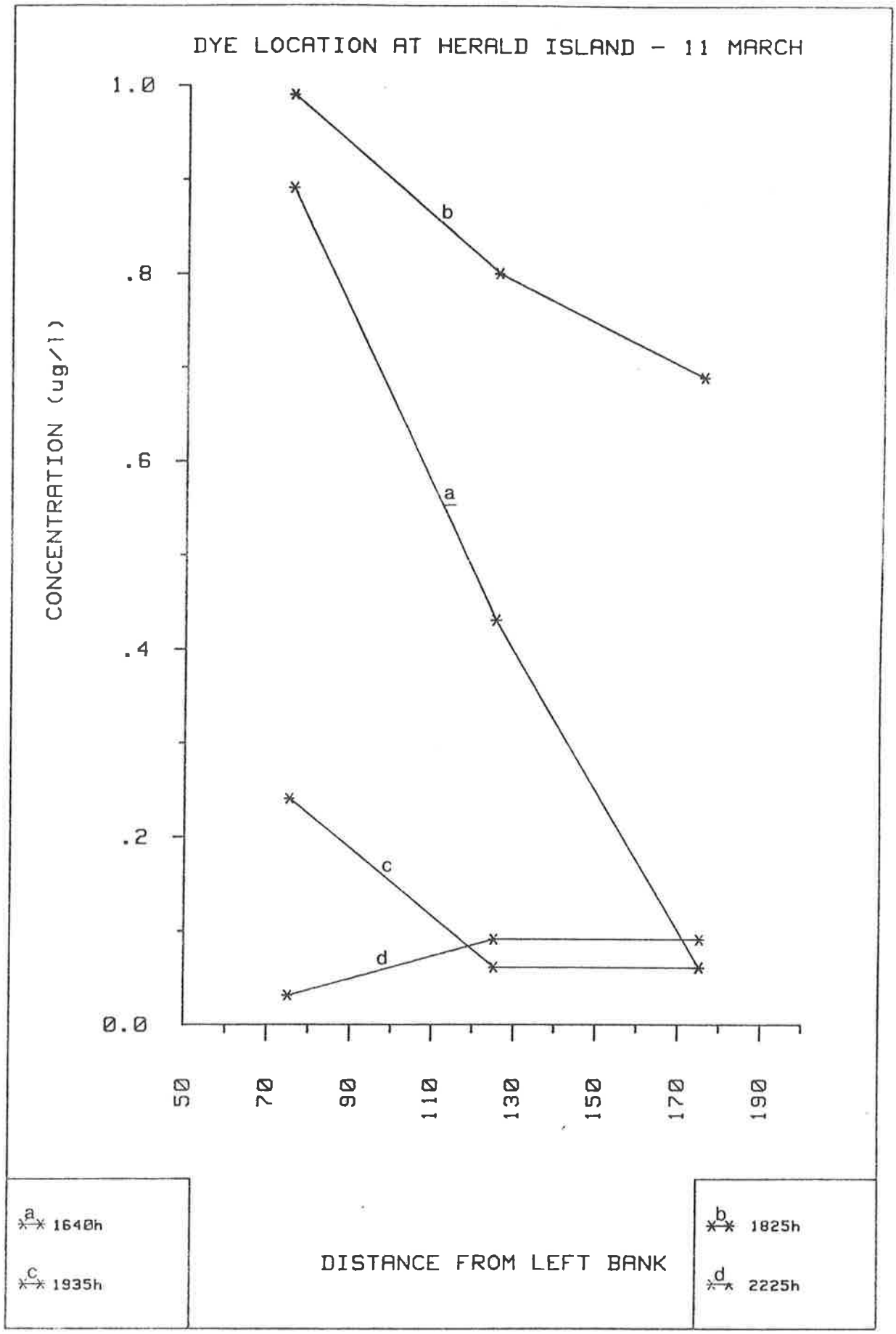



Figure 14 Dye Location at Herald Island - 11 March 1981

Therefore, because Lucas Creek contaminated water tends to remain near the left bank on the outgoing tide, much of this water returns to Lucas Creek.

The propensity for water to leave and return faster near the left bank is demonstrated as far upstream as L17, 614 m from the mouth of the estuary. Figure 15 shows dye distribution in the cross section during parts of the tidal cycle on 14 March. It is similar to that noted for L18 and L19, but the contrasts are less marked. It is also apparent that this is a daily pattern for which there are two probable explanations.

Firstly, at the mouth of Lucas Creek the water on the ebb tide comes into contact with the body of flowing water in the main harbour, causing eddies and a forcing of Lucas Creek water towards the left bank. Secondly, the presence of a subtidal bar on the right side of the channel near the mouth of the estuary (Fig. 16) causes a change in the velocity distribution as the tide nears its lowest level, slowing water near the right bank. It is thus permitted to move faster in both directions near the left bank. Because of this a significant proportion of the water leaving will return to Lucas Creek.

## CONCLUSIONS

With small freshwater inflow, the dominant cause of mixing and dispersion is tidal action. Tidal prism volumes are large, which, in the absence of other factors such as rock sills at L4-L5, would otherwise give a rapid potential mixing of contaminant and a consequent fast removal.

Differences in dye concentration occur in the vertical between ebb and flood tides, which reflect the velocity distribution. On the ebb tide higher concentrations occur near the surface, whereas on the flood tide the new water moving faster near the surface reverses this trend. Lateral differences in dye concentration are even greater. Concentrations near the bank tend to lag behind channel concentrations by up to 2 h as a direct result of differences in velocities.

A simple model of the processes of longitudinal dispersion can be proposed. At high tide most of the dye in the estuary is distributed in the upper section above the point where the volume upstream at high tide is equal to the entire volume of the estuary at low tide. Below this point are lower concentrations, principally derived from water that has returned from outside the estuary. Near the mouth, no dye is present. Dye travels down-estuary on the ebb tide at a rate equal to the tidal flow until a tide level is reached such that the section above L5 takes on the characteristics of a river with rapids. This traps some dye, but the remainder of the dye continues to move downstream at a rate equal to tidal flow. By low tide a considerable proportion (10%-25% depending on tide conditions) will have left the estuary, and a substantial quantity of dye will be in the estuary, near the mouth. Dye will have left the estuary close to the left bank and will be concentrated near the left bank in the main harbour channel. Residual high concentrations will occur in pools above L5 as the flushing rate in that section is determined by the stream inflow which is usually small.

On the incoming tide some dye will return to the Lucas estuary, again near the left bank, and some will be transported up the main Upper Harbour channel. Substantial quantities of 'new' water will enter Lucas Creek. Any water left in the estuary at low tide will be pushed back upstream, with some mixing beyond the point described previously. When the tide overtops the rock sills, the water ponded behind them will be pushed to the extreme upper end of the estuary, with water with less concentrated dye moving into the sections immediately upstream of the rock bars. The water entering the mouth of Lucas Creek will be substantially uncontaminated by the time the tide level reaches the level of the mudflats of the Te Wharau Stream arm of the estuary, and water downstream of this arm will have only low concentrations of dye. During successive tidal cycles the pattern of dye concentration remains substantially the same, but the total mass present gradually diminishes.

Under low flow conditions the pull-push of the tides and the resulting replacement effect is important, perhaps being more significant than the mixing that occurs due to differences in advective velocities.

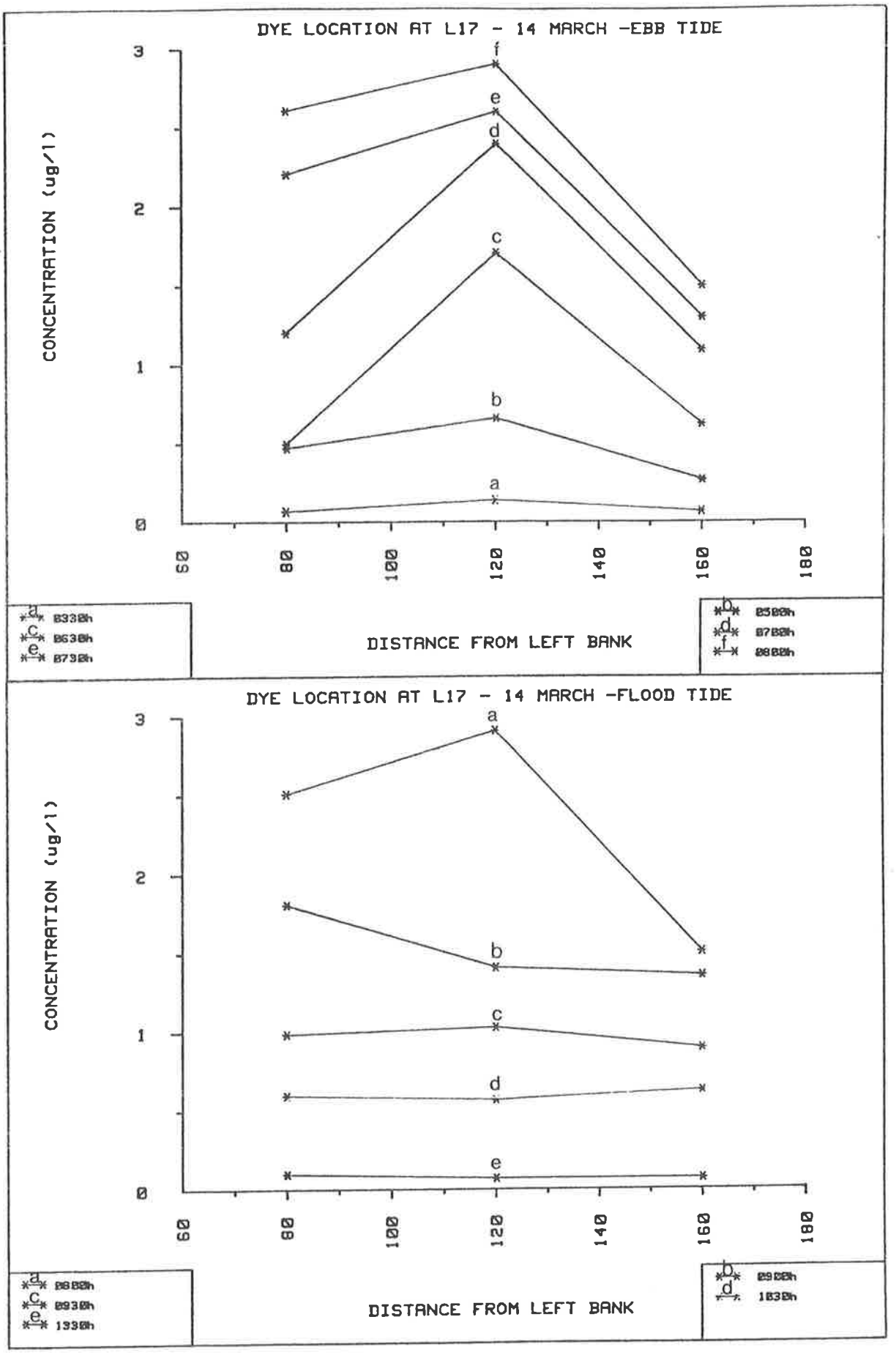



Figure 15 Dye Location at L17 - 14 March 1981

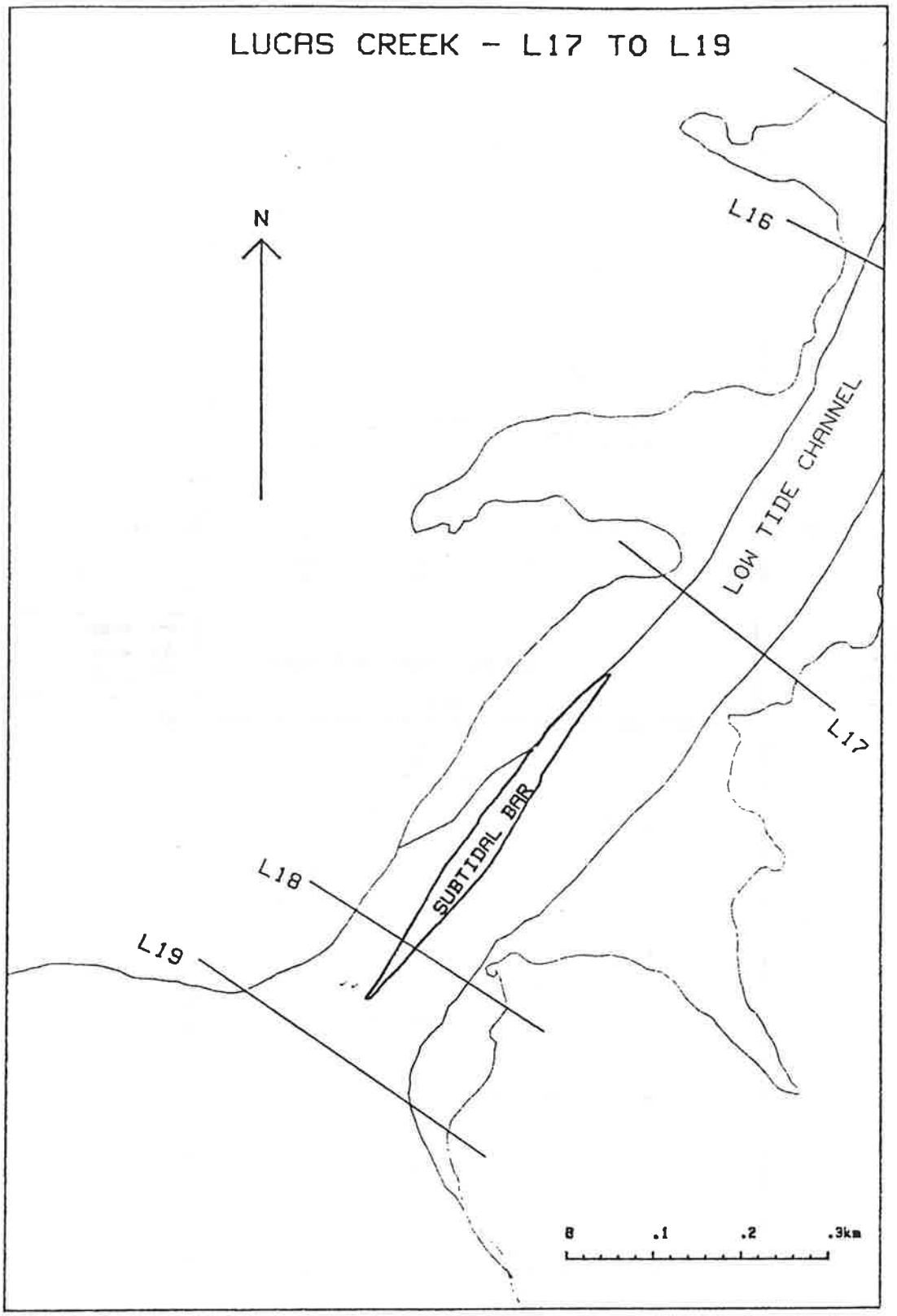



Figure 16 Lucas Creek - L17-L19

The constraining effect of the morphology of Lucas Creek on dispersion is also clearly demonstrated by the research. The rock sills above L5 cause considerable ponding during a significant proportion of the tidal cycle, and slows rates of dispersion considerably. Also, the subtidal bar near the mouth of the estuary appears to have a considerable effect on the distribution of dye leaving the estuary, and creating the potential for a greater proportion to return than may be expected.

Lucas Creek can be separated into two subsystems for the purposes of studying mixing and the dispersion of a contaminant. The separation is caused by the rock sills and reinforced by high tide and low tide volume considerations. Whereas the removal of contaminant in the section below L5 will be rapid (possibly as fast as predicted by conventional methods), removal from the section above L5, into which about two-thirds of the catchment drains, will be slow.

## REFERENCES


Auckland Harbour Board and Auckland Regional Authority 1975: "Waitemata Harbour Plan", Auckland.

Church, M. 1974: Electrochemical and Fluorometric Tracer Techniques for Streamflow Measurements. Brit. Geomorph. Res. Group Tech. Bull., 12.

Feuerstein, D.L.; Shelleck R.E. 1963: Fluorescent tracers for dispersion measurements. ASCE Vol. 89, No. SA4, 1-21.

Fischer, H.B.; List, E.J.; Koh, R.C.Y.; Imberger, J.; Brooks, B.H. 1979: "Mixing in Inland and Coastal Waters". Academic Press, New York.

Hubbard, E.F.; Stamper, W.G. 1972: Movement and Dispersion of Soluble Pollutants in the Northeast Cape Fear Estuary, North Carolina. U.S. Geol. Sur. Water Supply Paper, 1873 E.

Johnson, R.M.S. 1979: Interim Physical Data Report on the Upper Waitemata Harbour Estuary, Unpub. internal report, Water & Soil Science Centre, MWD, Hamilton.

Officer, C.B. 1976: "Physical Oceanography of Estuaries (and Associated Coastal Waters)." Wiley, New York.

Pritchard, D.W.; Carpenter, J.H. 1960: Measurement of turbulent diffusion in estuarine and inshore waters. Hydrol. Sci. Bull., NS2Q, 37-50.

Scott, C.H.; Norman, V.W.; Fields, R.K. 1969: Reduction of Fluorescence of Two Tracer Dyes by Contact with a Fine Sediment. U.S. Geol. Survey Prof. Paper No 650-B, pp.164-168.

Smart, P.L.; Laidlaw, I.M.S. 1977: An evaluation of some fluorescent dyes for water tracing. Water Res. Res. Vol. 13, No. 1, 15-33.

Turner and Associates 1974: "Turner Filter Fluorometer III Operating Instructions and Service Manual".

Warner, R.F.; Smith, D.I. 1980: The Use of Fluorometric Tracers to Study Effluent Movement in the Georges River NSW. Proc. of the 10th NZ Geog. Soc. Conf. and 49th ANZAAS Congress (Geog. Sci.), pp 48-53.

Wilson, J.F. 1968: Fluorometric procedures for dye tracing. In "Techniques of Water Res. Investig. of the US Geol. Survey", Vol. 2, US Geol. Survey.


## DISCUSSION

R K Penney Is there any evidence to suggest a pollutant entering Lucas Creek below L5 (as opposed to L1 which was the input site for the tracer) will enter the part of the Creek above L5 and thus have a much longer residence in the estuary?

Parnell The evidence suggests that dye entering Lucas Creek below L5 will be unlikely to travel upstream of L5. It appears there are two very different parts in the Lucas Creek.

J Kitto Have you any explanation for the differences in the effect of salinity on different batches of Rhodamine WT?

Parnell Like Smart and Laidlaw (1977), I cannot explain the difference. It is clear that each batch of dye needs to be tested and an appropriate correction factor used in any field test until some explanation can be found.

T M Hume When using dyes in estuarine research what is the biggest problem : adsorption of dye onto organic or inorganic (sediment) material?

Parnell Rhodamine B has poor resistance to adsorption, because of its cationic nature. Pyranine and Lissamine have high resistance to adsorption on both inorganic and organic material. Because of the relatively clean nature of estuary waters, if an anionic tracer is used, little adsorption will occur. Scott *et al.* (1969) conducted experiments on Rhodamine WT concluding that in the presence of fine material, any loss of fluorescence will take place in a short time, that loss of fluorescence is independent of dye concentration, and that loss of fluorescence may be appreciable for concentrations of sediment greater than 500 mg/l.

R J Wilcock (1) Why didn't you test the relationship between rhodamine fluorescence decay and salinity using seawater solutions rather than synthetic NaCl solutions?

(2) You experimented with lissamine FF, a green dye, in spite of the suggestion that high background fluorescence made use of green fluorescors impractical. Did you check this for the water of the Upper Harbour?

Parnell (1) I was unable to accomplish all the studies desirable in the time available. Also a number of salt concentrations were used in the experiments so some ability to control salinity was necessary.

(2) Samples for background were taken on many occasions. These were taken for each sample site and good explanations of background as a function of tide height and site were found. It was found that backgrounds could be easily removed.

R A Heath You mention that the salinity defect method for calculating the residence time in Lucas Creek may not be applicable because the flood tide water is already low in salinity. However, this is taken into account, for the Lucas Creek water will have a lower salinity than would be the case if the exchange was with more saline oceanic water.

Parnell The difficulties in using the $t = V_f/R$ where $V_f = V(S_o - S_n)/S_o$ method has problems for two reasons. Firstly, a satisfactory measurement of $S_o$ ('oceanic' salinity) is impossible because the main channel salinity changes. Secondly, the amount of water which has already left Lucas Creek (possibly many times) and returns cannot be accounted for.

J C Rutherford Can you tell us the residence times of Lucas Creek as calculated using:

(1) tidal prism;

(2) salinity method;

(3) dye results?

Parnell (1) Salinity measurements give about 5 days. (2) The basic tidal prism method gave a 1 tidal cycle. The modified Ketchum method gave about 3 tidal cycles. (3) Under the freshwater inflows which operated at the time, the dye half-life was 3½ tidal cycles, but some dye was still present about 90 tidal cycles after injection.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre November 17-18, 1981*

# UPPER WAITEMATA MODELLING STUDIES

*M J O'Sullivan & R K Penney, Theoretical & Applied Mechanics Department, University of Auckland*

## ABSTRACT

One-dimensional models are commonly used for analysing the hydraulics and mixing of rivers and because of the relatively narrow tidal creeks in the Upper Waitemata Harbour it is appropriate to model it as a branched network of one-dimensional rivers. In the present work the numerical implementation of one-dimensional hydraulic and pollutant dispersal models is discussed and model results for Lucas Creek (in the Upper Waitemata) are compared with results from field observations.

## INTRODUCTION

In recent years numerous studies of computer modelling of the hydraulics of estuaries and rivers have been carried out. Early work is summarized by Dronkers (1964) and the more recent publications by Hinwood and Wallis (1975a, b), Abbott(1979), Ramming and Kowalik (1980) and Nihoul (1978). For the special case of transient river hydraulics a good summary is provided by Henderson (1966).

The computer modelling of mixing processes in estuaries or rivers is well summarised by Fischer *et al.* (1979) and Fischer (1976a, b).

The main difference in the approaches adopted by various authors is in the level of averaging applied. Even using modern large computers it is not possible to produce a model which evaluates velocities and pollutant concentrations, for example, at every point in the body of water under investigation. Some averaging is always required. In the most detailed models the flow region is split up into a large number of discrete elements or blocks and the variables are assumed to be constant (or some simple polynominal form) within each block. A small amount of work has been carried out with two or three layers of blocks thus allowing for modelling of stratified flows. But most studies have assumed that the flow is vertically well mixed and have considered depth-averaged quantities. For rivers most studies have assumed that the whole river is well mixed and have used cross-sectional-averaged quantities.

## BASIC EQUATIONS

The basic equations used express conservation of mass, momentum and pollutant (or some chemical such as salt or dye). The variables used are A the area of the flow, U the average velocity, Q the discharge (Q=UA), y the depth of water, C the concentration of pollutant and h the elevation of the river bottom above datum. Then conservation of mass can be expressed by the partial differential equation

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad (1)$$

Here t is time, x is distance along the river and q is the lateral inflow (from small streams) of water. Similarly conservation of momentum is expressed by

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(UQ) + gA\frac{\partial}{\partial x}(y+h) + \frac{gQ|Q|}{C^2AR} = 0 \qquad (2)$$

Here g is the acceleration due to gravity, R is the hydraulic radius, and C is the Chézy friction coefficient in an empirical frictional "drag" proportional to average velocity squared. Conservation of chemical species is expressed by

$$\frac{\partial(AC)}{\partial t} + \frac{\partial(QC)}{\partial x} = \frac{\partial}{\partial x}\left(DA\frac{\partial C}{\partial x}\right) + qC_o \tag{3}$$

Here D is a diffusion coefficient and $C_o$ is the concentration of the chemical in the lateral inflow.

The basic governing equations often appear in different forms. Using the fact that, b, the breadth of the flow (see Fig. 1) is given by

$$b = \frac{\partial A}{\partial y} \tag{4}$$

then equation (1) can be rewritten as

$$b\frac{\partial y}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{5}$$

The usual form for the Chézy friction coefficient is

$$C = R^{1/6}/n \tag{6}$$

where n is the well known Manning's n. Then (2) can be rewritten as

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(UQ) + gA\frac{\partial}{\partial x}(y+h) + \frac{gn^2|U|Q}{R^{4/3}} = 0 \tag{7}$$

Equation (1) and (3) are often combined to give

$$\frac{\partial C}{\partial t} + U\frac{\partial C}{\partial x} = \frac{1}{A}\frac{\partial}{\partial x}\left(DA\frac{\partial C}{\partial x}\right) + \frac{q}{A}(C_o - C) \tag{8}$$

No analytical solutions can be found for the coupled, non-linear equations (5), (7) and (8) (or (1), (2) and (3)) and therefore numerical methods must be used.

Some variations of these equations are often used. An allowance for the velocity variation across the cross-section is often made by introducing an extra coefficient $\alpha$ in the second term of (7), that is $\alpha\frac{\partial}{\partial x}(UQ)$, where $\alpha$ is defined by

$$\alpha = \frac{A}{Q^2}\int_A u^2 dA$$

Here u is the velocity value at each point in the cross-section.

Also (7) is often rearranged, using the chain rule for partial differentiation, to give

$$\frac{\partial Q}{\partial t} + 2U\frac{\partial Q}{\partial x} + (gA - bU^2)\frac{\partial y}{\partial x} - U^2A_x^y + gA\frac{\partial h}{\partial x} + \frac{gn^2|U|Q}{R^{4/3}} = 0 \tag{9}$$

where $A_x^y = \left[\frac{\partial A}{\partial x}\right]$ y constant.

## NUMERICAL TECHNIQUES

The basic approach adopted here is to replace the governing partial differential equations outlined above by finite difference approximations. These discrete versions of the equations of conservation of mass, momentum and pollutant can also be derived without reference to any partial differential equations by considering conservation principles for a discrete representation of the river. In Fig. 2 the subscript i refers to quantities evaluated at the centre of the ith section of the river, each of length $\Delta x$, and the subscript $i + \frac{1}{2}$ to quantities evaluated at the interface between the ith section and the i+1 section. The superscript n refers to quantities evaluated after n time increments each of length $\Delta t$.

Conservation of mass (or volume if density is cancelled) then requires that the change of volume in a block must be balanced by the difference between the flow entering and leaving the block, together with any

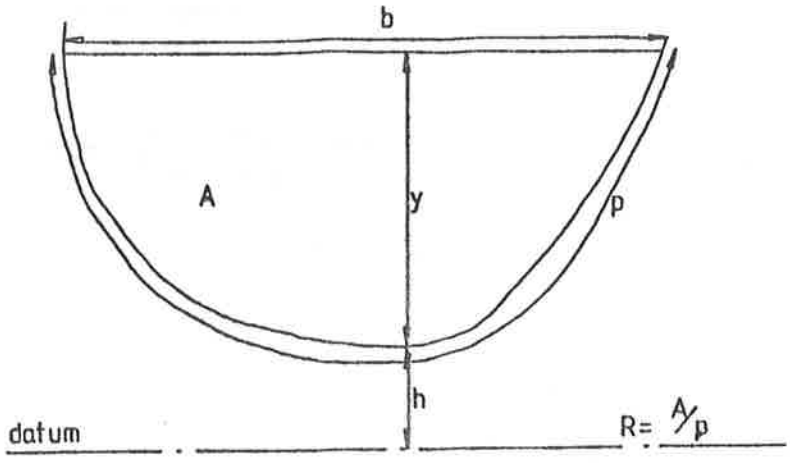



FIGURE 1: GEOMETRY OF FLOW.

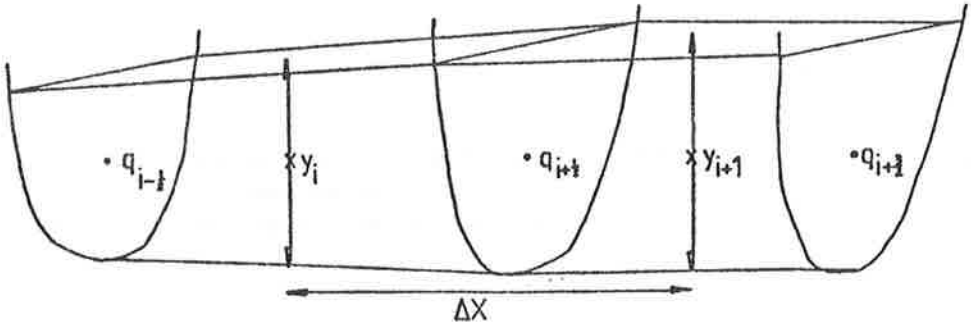



FIGURE 2: FINITE DIFFERENCE SCHEMATISATION

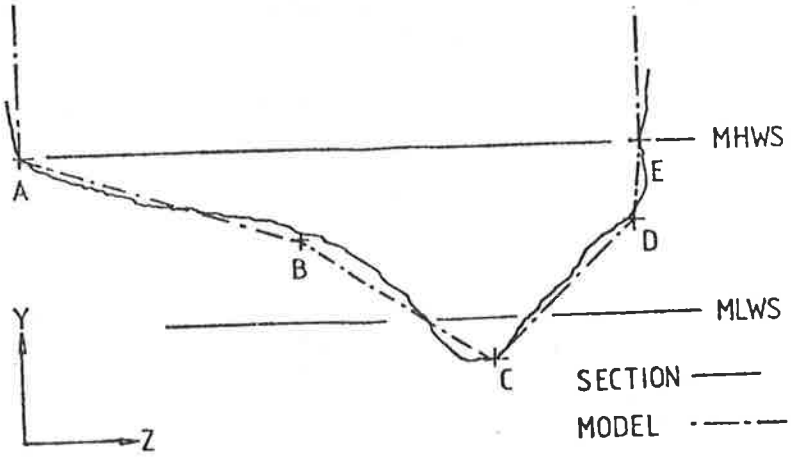



FIGURE 3: BATHYMETRY.

lateral inflow. Thus

$$\Delta x\, b^n_{i+\frac{1}{2}}(y^{n+1}_i - y^n_i) + \Delta t(\bar{Q}^{n+1}_{i+\frac{1}{2}} - \bar{Q}^{n+1}_{i-\frac{1}{2}}) = \Delta t \Delta x\, \bar{q}^{n+1}_i \tag{10}$$

Here the flows at the interfaces and the lateral inflow have been averaged, for example

$$\bar{Q}^{n+1}_{i+\frac{1}{2}} = \frac{1}{2}(Q^{n+1}_{i+\frac{1}{2}} + Q^n_{i+\frac{1}{2}}) \tag{11}$$

In order to obtain equations which are linear in the unknowns, $y^{n+1}_i$ and $Q^{n+1}_{i+\frac{1}{2}}$ the breadth $b^n_{i+\frac{1}{2}}$ is evaluated at the beginning of the time step. The conservation of momentum is expressed as

$$\begin{aligned}
&\Delta x(Q^{n+1}_{i+} - Q^n_{i+\frac{1}{2}}) + \Delta t\omega^n_{i+\frac{1}{2}}(U^n_{i+3/2}\bar{Q}^{n+1}_{i+3/2} - U^n_{i+\frac{1}{2}}\bar{Q}^{n+1}_{i+\frac{1}{2}}) \\
&+ \Delta t(1 - \omega^n_{i+\frac{1}{2}})(U^n_{i+\frac{1}{2}}\bar{Q}^{n+1}_{i+\frac{1}{2}} - U^n_{i-\frac{1}{2}}\bar{Q}^{n+1}_{i-\frac{1}{2}}) \\
&+ \Delta t g A^n_{i+\frac{1}{2}}(\bar{y}^{n+1}_{i+1} + h_{i+1} - \bar{y}^{n+1}_i - h_i) \\
&+ \Delta t \Delta x\, g\left(\frac{n^2|U|}{R^{4/3}}\right)^n_{i+\frac{1}{2}} \bar{Q}^{n+1}_{i+\frac{1}{2}} = 0
\end{aligned} \tag{12}$$

Here all barred quantities are averaged over a time step as in equation (11).

As before, all coefficients are evaluated at the beginning of the time step to preserve the linearity of the difference equations. The most important special feature of (12) is the "upstream" weighting parameter $\omega^n_{i+\frac{1}{2}}$ which is defined by

$$\omega^n_{i+\frac{1}{2}} = \begin{cases} 1 & U^n_{i+\frac{1}{2}} < 0, \\ 0 & U^n_{i+\frac{1}{2}} \geqslant 0. \end{cases} \tag{13}$$

This type of weighting ensures that the momentum flux which is transported into the block comes from the upstream direction (that is, from where the flow is coming). The feature gives good stability of the difference approximations, especially for fast moving flows. A similar scheme can be derived from the alternative form of the momentum equation (9) but is slightly more cumbersome because of the extra terms involved.

If only hydraulic modelling is of interest then (10) and (12) must be solved for M blocks in a river, say, together with initial data, $y^0_i$, i=1, 2,...,M, and $Q^0_{i+\frac{1}{2}}$, i=0,1,2,...,M and boundary conditions. The usual boundary conditions of interest in a tidal river are the specifications of a tidal height at the bottom of the river, thus specifying $y^{n+1}_1$, and the specification of a discharge at the top of the river, thus giving $Q^{n+1}_{M+\frac{1}{2}}$. Equations (10) and (12) can be combined, with some algebraic effort, to give a tri-diagonal system of equations which can then be solved very efficiently.

If the dispersion of a pollutant in the river is also of interest then the velocity distribution $U^n_{i+\frac{1}{2}}$ and flow areas $A^n_{i+\frac{1}{2}}$ can be used with the following difference approximation of (8):

$$\begin{aligned}
&\Delta x(C^{n+1}_i - C^n_i) + \tfrac{1}{2}\Delta t(U^n_i + |U^n_i|)\,(\bar{C}^{n+1}_i - \bar{C}^{n+1}_{i-1}) \\
&+ \tfrac{1}{2}\Delta t(U^n_i - |U^n_i|)\,(\bar{C}^{n+1}_{i+1} - \bar{C}^{n+1}_i) \\
&= \frac{\Delta t}{A^n_i \Delta x}\left[D_{i+\frac{1}{2}}A^n_{i+\frac{1}{2}}(\bar{C}^{n+1}_{i+1} - \bar{C}^{n+1}_i) - D_{i-\frac{1}{2}}A^n_{i-\frac{1}{2}}(\bar{C}^{n+1}_i - \bar{C}^{n+1}_{i-1})\right] \\
&\quad + \frac{\Delta t \Delta x}{A^n_i}\bar{q}^{n+1}_i(\bar{C}^{n+1}_{oi} - \bar{C}^{n+1}_i)
\end{aligned} \tag{14}$$

Again barred quantities are averaged as in (11). Upstream weighting of the advective or transport term, $U \frac{\partial C}{\partial x}$, has been used in (14). This type of difference approximation gives good stability properties but leads to the addition of "numerical" dispersion. Other more sophisticated schemes can be used to improve the dispersion characteristics (see Stone and Brian (1963) or Crowley (1968), for example).

For a river consisting of M blocks the initial concentrations $C_i^o$, i = 1, 2,...M must be specified and boundary conditions at each end given. For tidal creeks such as Lucas Creek it is appropriate to specify the concentration at the upstream end, $C_M^{n+1}$, while at the bottom the concentration should be specified during the incoming tide and a zero concentration gradient specified during the outgoing tide. For i=1, 2,...M (14) gives a tri-diagonal system.

## BATHYMETRY

In (10), (12) and (14) $A_{i+\frac{1}{2}}^n$, $R_{i+\frac{1}{2}}^n$, $b_i^n$ must be evaluated in terms of $y_i^n$, $y_{i+1}^n$. In the present study the bathymetry at a number of sections was obtained from field data and converted to an approximate piece-wise linear form as shown in Fig. 3. The data at cross-sections between these major sections is obtained by linear interpolation.

## RESULTS

The basic programs were tested on several idealised problems and then applied to data from Lucas Creek. During a field experiment conducted by MWD and University of Auckland personnel very detailed data was obtained for a period of 16 hours. The experiment was aimed firstly at monitoring dye concentrations in detail during this period (24 hours after a controlled release at the head of Lucas Creek near the Albany bridge) and in lesser detail over a longer period. Secondly, the experiment determined detailed hydraulic data in the form of velocity profiles at up to 8 verticals at 2 cross-sections and spot velocity measurements at various stations up the river. Tidal heights at each of these sections were also recorded.

From this data average velocities were calculated to allow comparison with the computer model results.

For the computer model the measured tide height at the downstream end (Section L17) was used as one boundary condition and the measured discharge at the top section (L5)was used as the upstream boundary condition. The main calibration required was the choice of Manning's n. After some experimentation a value of n=0.015 was found to give a good fit to the experimental data. Water level profiles at various times are shown in Figs. 4 and 5 and the velocity at the mouth (L17) is plotted in Fig. 6.

It was not found necessary to vary n with depth probably because the water slope is quite small and the flow is governed mainly by volume conservation rather than frictional resistance.

The results of the dye test are compared with the computer model results in Fig. 7. The measured concentrations at the top (L5) and bottom (L17) sections were specified during the outgoing and incoming tides respectively. Otherwise a zero concentration gradient was enforced at each end. A diffusion coefficient of 20 $m^2/s$ was used. This was taken as a typical value from Johnston (1980).

The results were not particularly sensitive to the choice of diffusion coefficient. This is partly because the data are for a period 24 hours after the initial release of dye and the sharp concentration fronts have already been somewhat smoothed. But the main reason is that the large numerical dispersion dominates the physical dispersion coefficient. Reasonable agreement between model results and field data is obtained but this result cannot be used to calculate the physical dispersion coefficient reliably. Improved methods are being developed.

Detailed dye measurements at each section confirmed that Lucas Creek is quite well mixed laterally and with depth and therefore the 1-D assumption is valid.

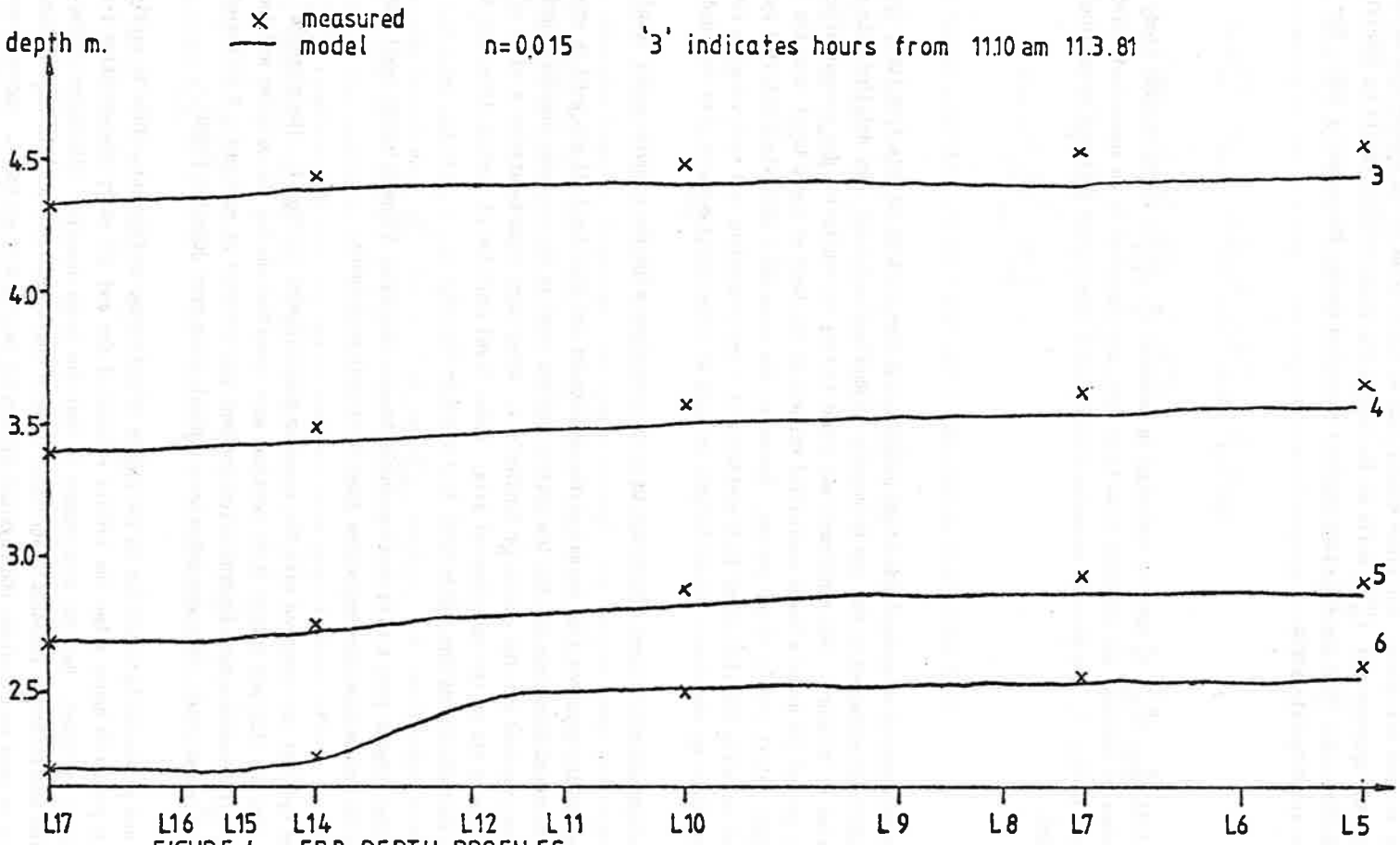



FIGURE 4 : EBB DEPTH PROFILES

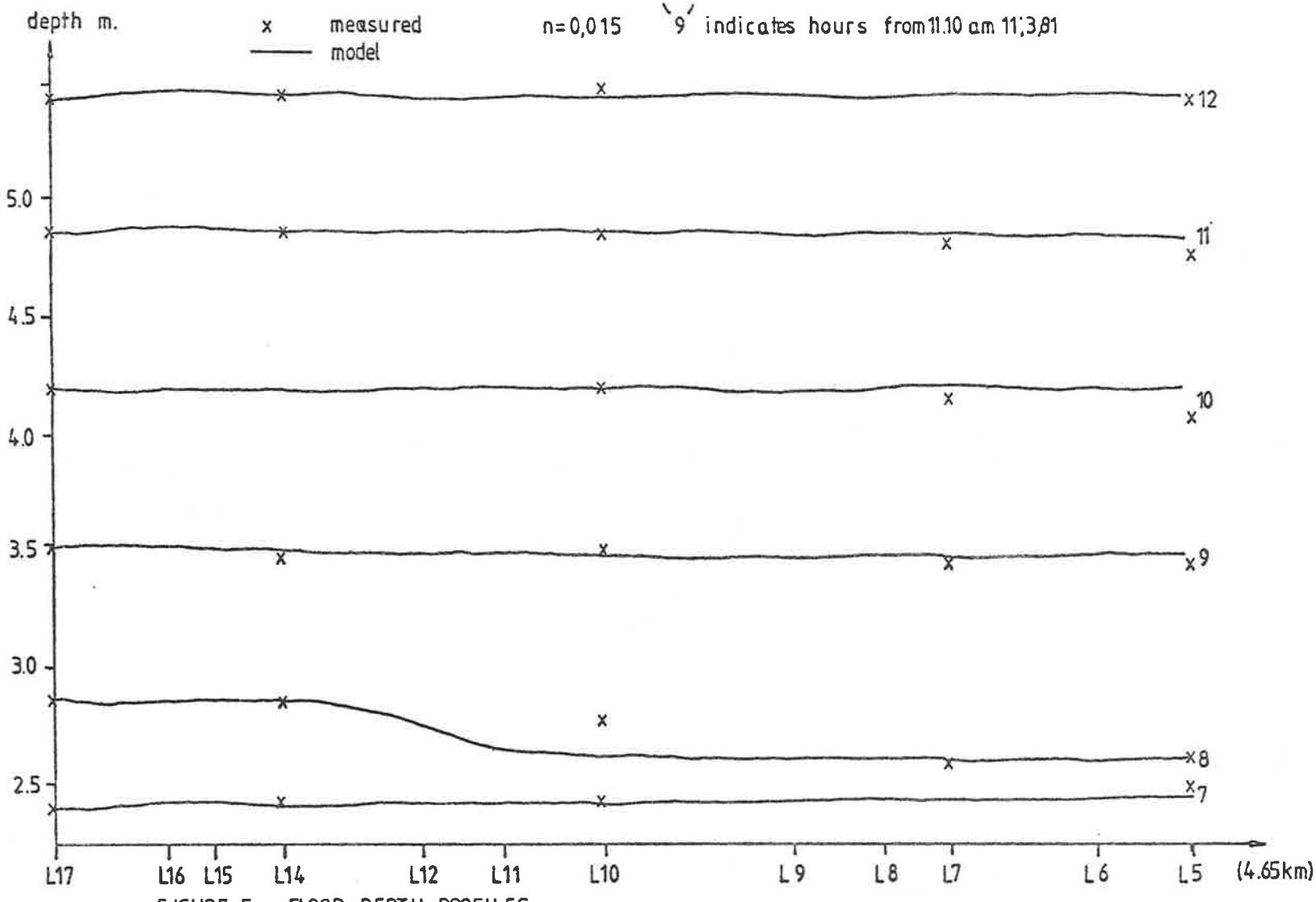



FIGURE 5 : FLOOD DEPTH PROFILES.

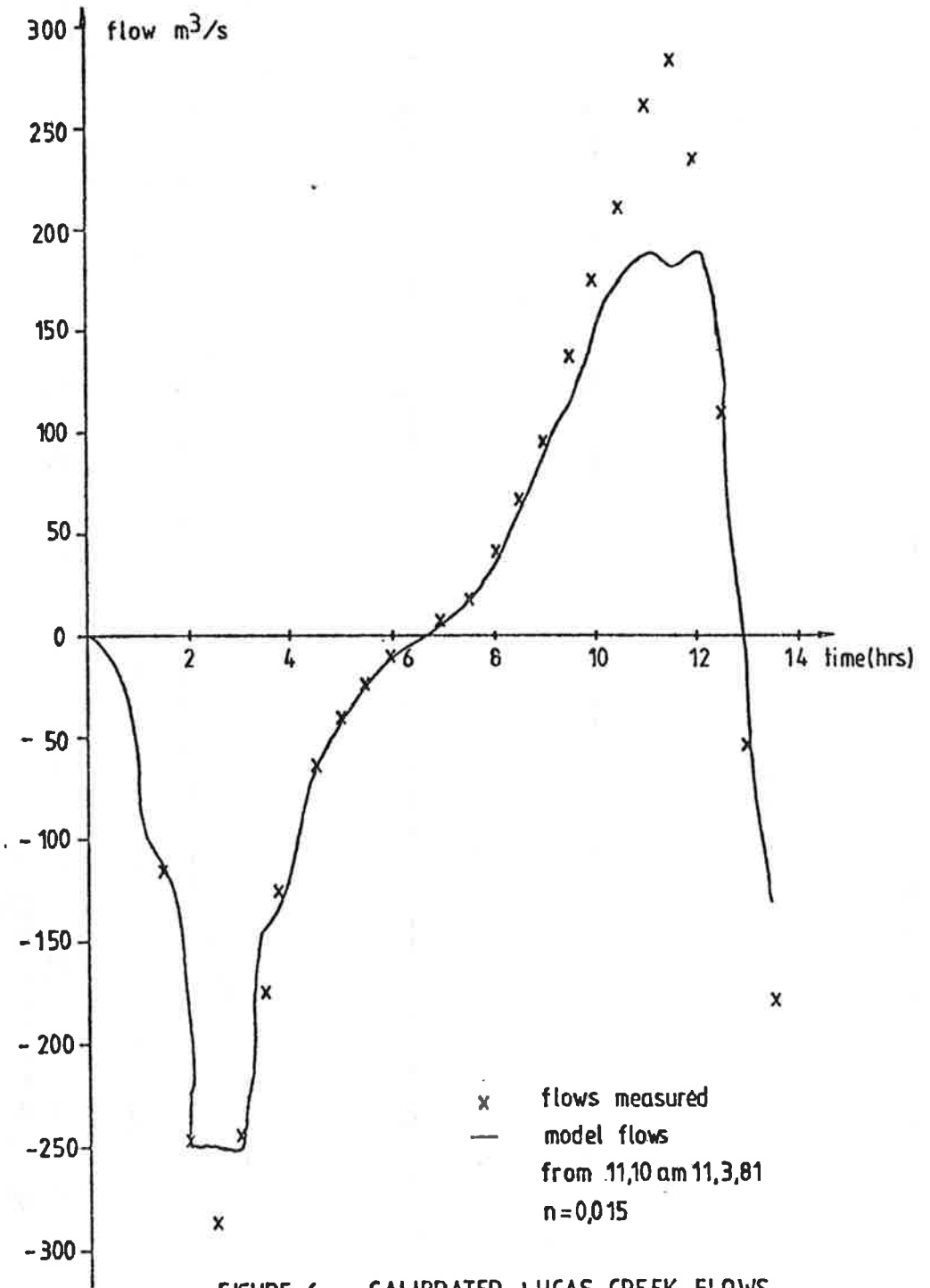



FIGURE 6: CALIBRATED LUCAS CREEK FLOWS

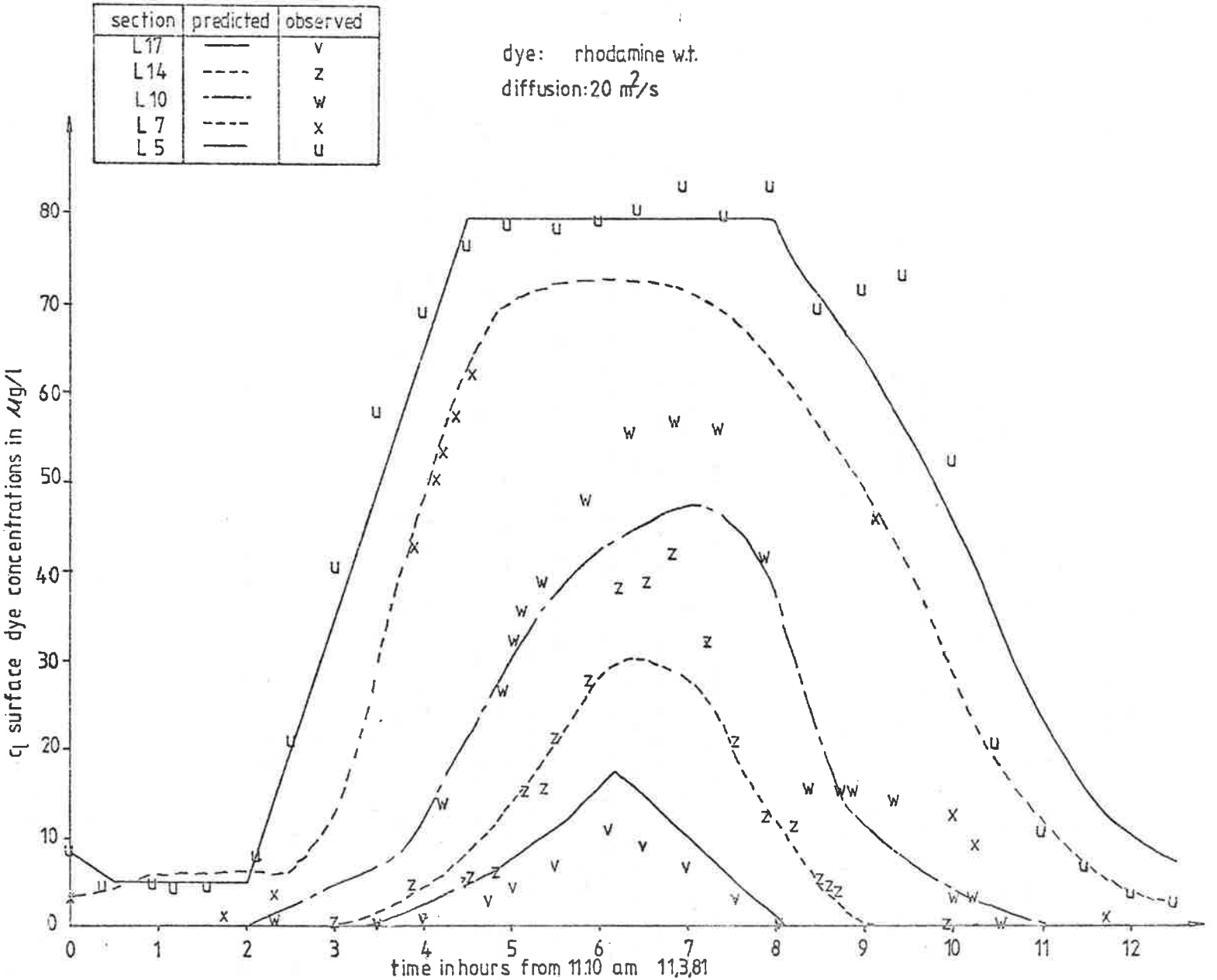



FIGURE 7: DYE CONCENTRATIONS IN LUCAS CREEK

## CONCLUSIONS

The 1-D modelling of Lucas Creek has produced good agreement between computed results and field data. The ease of achieving such a good match is rather deceptive and a number of points should be stressed as a warning.

1 Lucas Creek is quite well mixed laterally and with depth.
2 The experiment conducted did not adequately test the dispersion performance of the computer model.
3 An improved algorithm is required to reduce spurious numerical dispersion and allow a determination of the actual dispersion coefficient.
4 The ponding effect above L5 was excluded from the model because the hydraulic model was not designed to handle this situation and was made external to the computer model. Work is proceeding on this point with the aim of using the specified dye release as the only input at the upstream end of the computer model.
5 The interchange at the bottom of Lucas Creek was also made external to the computer model with the concentration at L17 being specified during the incoming tide. Work is proceeding on a network model of the whole of the Upper Waitemata Harbour which will allow a proper test of the ability of the model to represent mixing at the mouth of each creek.

## REFERENCES


Abbott, M.B. 1979: "Computational hydraulics". Pitman, London.

Crowley, W.P. 1968: Numerical advection experiments. Monthly Weather Review, Vol. 96, 1-11.

Dronkers, J.J. 1964: "Tidal Computations in Rivers and Coastal Waters". North Holland, Amsterdam.

Fisher, H.B. 1976a: Mixing and dispersion in estuaries. Ann. Rev. Fluid Mech. Vol. 8, 107-133.

Fisher, H.B. 1976b: Some remarks on computer modelling of coastal flows. ASCE Vol. 102, No. XXB, 395-406.

Fisher, H.B.; List, E.J.; Koh, R.C.Y.; Imberger, J.; Brooks, N.H. 1979: "Mixing in Inland and Coastal Waters". Academic Press, New York.

Henderson, F.M. 1966: "Open Channel Flow". Macmillan, New York.

Hinwood, J.B.; Wallis, J.G. 1975a: Classification of models of tidal waters. ASCE Vol. 101, No. HY4, 1315-1331.

Hinwood, J.B.; Wallis, J.G. 1975b: Review of models of tidal waters. ASCE Vol. 101, No. HY4, 1405-1421.

Johnston, R.M.S. 1980: Interim physical data report on the Upper Waitemata Harbour estuary. Unpub. report, Water & Soil Science Centre, MWD, Hamilton.

Nihoul, J.C.J. 1978: "Hydrodynamics of Estuaries and Fjords". Elsevier Oceanography Series No. 23, Elsevier, Amsterdam.

Ramming, H.G.; Kowalik, Z. 1980: "Numerical Modelling of Marine Hydrodynamics". Elsevier Oceanography Series No. 26, Elsevier, Amsterdam.

Stone, H.L.; Brian, P.T. 1963: Numerical solution of convective transport problems. AICE Vol. 9, 681-688.


## DISCUSSION

T M Hume (1) What are the minimum field data requirements to make the model applicable in other situations? (2) How does Manning's n vary with tidal range?

O'Sullivan (1) You need some bathymetry to begin with. In order to calculate Manning's n, water levels at two positions (and preferably three or four) during a tidal cycle are required. As an additional check enough velocity measurements to allow a reasonable estimate of discharge at the mouth of the creek would be useful. (2) It appears that a smaller Manning's n should be used in shallow water. However, to avoid complex "tuning" of the model a single value was used. Therefore, the water surface slope is too flat at high tide.

G J Macdonald Was the diffusion coefficient changed so as to achieve the best fit of your computed concentration profile to measured concentration values?
O'Sullivan Three values D = 10, 20, 30 $m^2/s$ were tried and D = 10 $m^2/s$ gave the best fit of the three. Further testing might provide an even better fit.

R A Heath Have you looked at how the model fits at different tidal conditions (i.e. springs versus neaps)?
O'Sullivan Unfortunately we do not have sufficient field data to make this comparison.

A J Sutherland Could you please elaborate on the concentration boundary condition applied at each end of the reach?
O'Sullivan Basically the boundary condition (at either end of the reach) is that the concentration is specified if the flow is into the reach and a zero concentration gradient is specified if the flow is out of the reach. At the bottom of the reach for Lucas Creek (L17) the concentration specified during the incoming tide was taken as a linear decrease from that calculated at the turn of the tide to that observed at high tide. Thus the form of the measured concentration vs time curve was preserved but not the exact values. This boundary condition corresponds to the physical situation where the water which leaves the mouth of Lucas Creek is partially mixed before returning.

G B McBride On Fig. 6 why is it that the observed and predicted concentrations at site L17 between hours 6 and 8 do not agree? I have understood that concentration, rather than concentration gradient, was specified at this site for an incoming tide.
O'Sullivan The concentration is specified at L17 during the incoming tide, but not the measured concentration. To be consistent with the calculated concentration at the turn of the tide a concentration vs time profile is specified with the same form as the measured one but starting at the calculated value at the turn.

P W Williams Many estuaries have pools at their upper ends. Is your model really adequate seeing it cannot cope with the upper ends of these tidal creeks?
O'Sullivan I don't see any problem chopping these little creeks off at some arbitrary location (say at a rock sill) provided the effluent sources you are interested in are still within the system. If they are not, then you have to specify boundary conditions carefully.

D G Goring Your Manning's n of 0.015 is low. From experience n this low occurs only in concrete canals. I put it to you that there is another degree of freedom in fitting your model to field data and that is the initial volume of water in the model.
O'Sullivan The Manning's n of 0.015 is low but this probably arises from the inaccuracy of the friction terms for slow moving tidal flows. Differences in the initial state for trial stimulations were "damped out" after about one tidal cycle.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre* *November 17-18, 1981*

# ASPECTS OF CIRCULATION IN THE WAIMEA INLET : DYE TRACER STUDIES

*K G Westcott, Nelson Catchment Board, Nelson*

## ABSTRACT

Field work conducted in an attempt to optimise location and operation of the outfall from the Nelson Regional Sewage Scheme into the Waimea Inlet is described. Likely effluent pathways were established at neap tide (worst case conditions) using dye injections (periodically replenished) photographed from the air. These confirmed that the Waimea Inlet is poorly flushed. Calculations suggest, however, that bacterial contamination of bathing beaches in the inlet will not be severe. Recommendations concerning outfall location, design and operation were made.

## INTRODUCTION

The testing described in this paper has been undertaken to help optimise the location and operation of the outfall of a major local sewage treatment plant near Nelson which will discharge into the eastern "wing" of the Waimea Inlet.

The Waimea Inlet is situated at the mouth of the Waimea River. It has two entrances (see Fig. 1) and the eastern and western "wings" drain largely separately with minimal crossflow. At the Waimea River mouth a small natural delta currently deflects all but flood flows into the western "wing". The fresh water continues to flow westward at all tidal stages. Under most conditions salinity measurements made in the eastern "wing" show only slightly differing values within and outside the inlet (28-32 ppt) with no obvious dilution patterns discernible. During flood events significant volumes of typically silt-laden floodwater pass eastward, especially at higher tidal stages.

At the turn of the century the river flow was directed mainly eastward towards Tahunanui Beach. The present westerly discharge is a natural change and has not required engineering for channel maintenance.

The exchange volume of the eastern inlet varies from $10 \times 10^6$ to $32.5 \times 10^6$ $m^3$ over a total eastern area of some 2150 ha. During the lower neap tides only a portion of this area is covered at high tide. Tidal ranges vary from 4.4 to 1.5 m.

An echo-sounder cross section for the eastern entrance shows a channel width varying from 1800 to 200 m at spring tide. At the same time cross-sectional area varies from around 6500 $m^2$ to only 1200 $m^2$. At higher tidal stages some water flows over the 1 km wide Rabbit Island sandflats. As the tidal level lowers, flow shifts into the narrow main channel and velocities increase. Peak velocities in the main channel occur at this shift stage.

At present there are several large industrial discharges to the eastern inlet. Two local authorities discharge comminuted and raw sewage, and in addition there are several emergency overload outfalls. The consulting engineers for the Nelson Regional Sewage Scheme estimate of present nutrient inputs are given in Table 1.

Discharges 1, 2 and 3 all currently carry high facecal coliform loadings. The receiving channels generally display levels from 1,000-10,000 coliform/100 ml at mid-tide, and levels typical of raw sewage at low-water. Severe algal blooms of *Ulva* and *Enteromorpha* occur periodically adjacent to the meatworks. Intense gold-green *Euglena* blooms mark the immediate vicinity of discharges 1, 2, and 3. High acid content and sugar

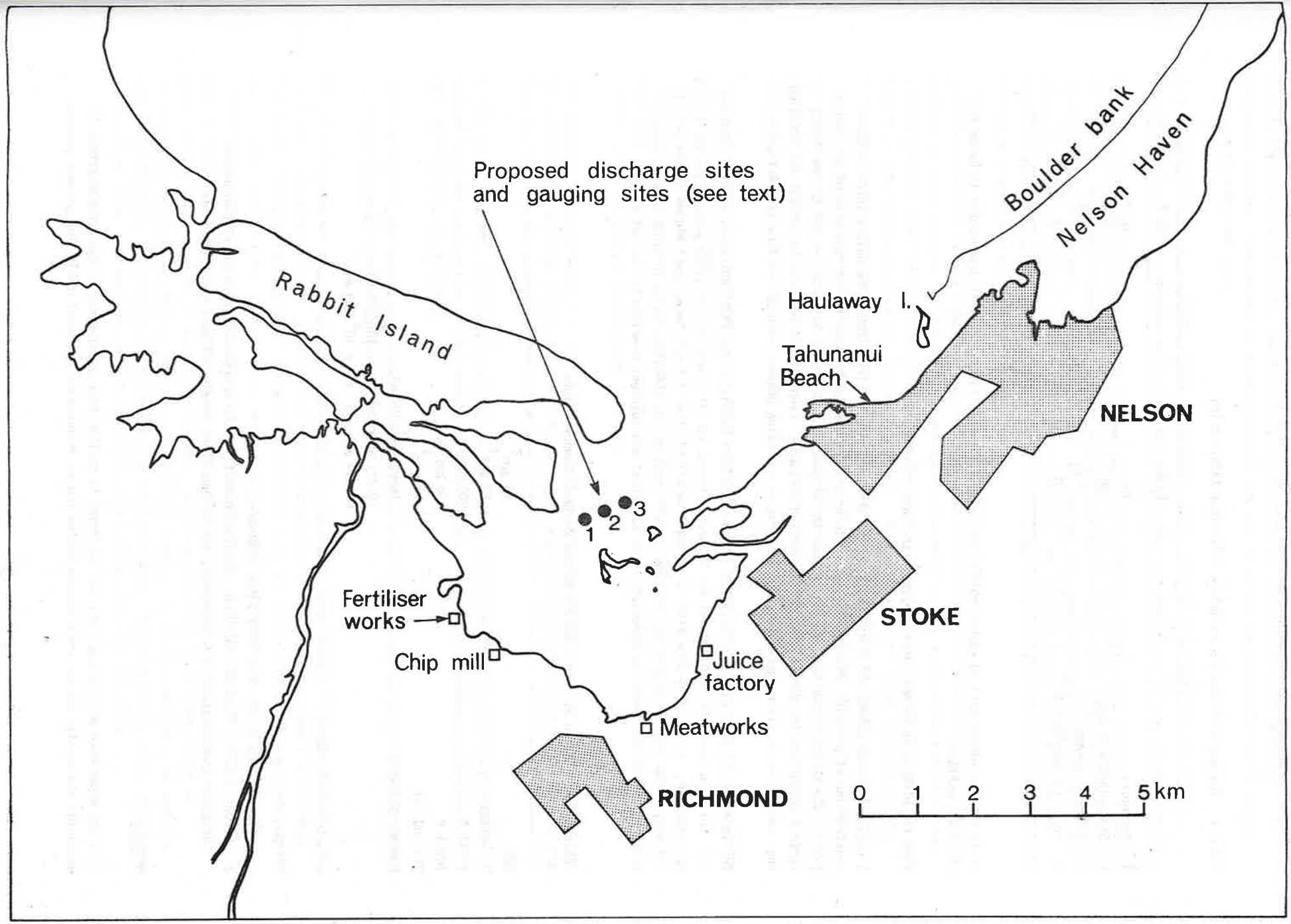



Figure 1 Location Map

combine to cause yellow fungoid growth at the juice factory point of discharge. Bark suspension has increased the organic content within the muds near the chipmill.

TABLE 1 The major discharges to Waimea Inlet (per tidal cycle)

| | kg N | kg P |
|---|---|---|
| 1. Meatworks | 181 | 16.25 |
| 2. Stoke/Tahuna Sewage | 49 | 8.4 |
| 3. Richmond Sewage | 24.5 | 4.2 |
| 4. Juice Factory/Chip Mill | 10 | 2 |

It is not considered safe to gather edible shellfish within the inlet and it is inadvisable to bathe near the Stoke outfall.

Thus the effects on Waimea Inlet presently are barely tolerable.

A regional sewage scheme which will collect and treat waste water from around the Waimea Inlet is under construction at present. After treatment effluent will be discharged into the eastern wing of the Waimea Inlet. The sewage scheme will greatly reduce the discharge of bacteria, nitrogen and BOD to the inlet. Table 2 summarises the composition of the treated effluent. Even without recourse to outfall optimization, the scheme clearly offers great benefits in terms of relieving present intense and localised pollution.

The inlet waters have an S.C. classification and Tahunanui Beach, at the tidal entrance, an S.B. Meeting S.B. coliform standards (200/100 ml) for Tahunanui Beach, at all times, is the primary objective for outfall optimization. A secondary objective is to avoid localised algal (*Ulva/Enteromorpha*) blooms in the outfall vicinity. The problem thus is to optimise outfall location and operation so as to avoid locally concentrated effluent slugs such as presently cause bacterial and nutrient overload in several areas.

TABLE 2 Specified effluent limits, Nelson Regional Sewage Scheme

| | |
|---|---|
| $BOD_5$ | 50 $g/m^3$ |
| S. Solids | 80 $g/m^3$ |
| Total N | 500 kg/day |
| Total P | 150 kg/day |
| Oil and Fat | 14 $g/m^3$ |
| Faecal Coliforms | always $<10^5$/100 ml |
| | 30-day average* $< 5 \times 10^4$/100 ml |
| | 60-day average $< 2 \times 10^4$/100 ml |

Notes:

1 *five samples at not less than 5 day intervals,

2 Boron, arsenic, fluoride, sulphide, cyanide, phenol, P.C.B.'s, organochlorides/organophosphorus, arsenicals/carbamates/2.4.D compounds, and various heavy metals all have specified limits.

## METHODS

Previous experience with tracers has led the Board to realise that measurement of dye concentrations in estuarine and coastal waters and accurate estimation of mixing rates is impracticable. The primary purpose

of these tracer studies has, therefore, been to indicate probable effluent "slug" pathways. Effluent (as visualised by dye) should be kept to areas of maximum turbulent mixing and minimum environmental impact, for as long as possible. Outfall siting, construction and operation would be based partly on the tracer studies. Some measure of dispersion rates can also be gained from close scrutiny of changes in dye patch colour and size.

Drogues were found to be unsuitable within these shallow waters because they inevitably ran aground. Outside, in deeper waters, drogues worked well but proved rather too expensive with loss of some drogues.

Both fluorescein and rhodamine have been used but rhodamine B or BN was quickly found to be most suitable. It is cheaper to purchase powdered dye and mix with methanol and/or dilute acetic acid, but this operation is fraught with difficulties; the powder form is carcinogenic, very persistent and badly staining, and extensive protective gear and other precautions are required. The best dye source found is pre-mixed agricultural spray-marker dye. Dye in this form is not cheap but is concentrated and long lasting so that economy of release will offset the purchase price.

In tracer studies it is most important that tracer movements accurately reflect real situations involving effluent or water movement. If dye is simply poured onto the water surface a proportion remains at the surface and will often not follow deeper water movements. Thus rhodamine was discharged at a depth of from 1 to 2 m in the shallow channels (3 - 5 m maximum depth). Thus ample vertical mixing occurred in both directions. Pumped sampling was earlier used to confirm adequate vertical dye distribution. A simple arrangement comprising a hand plunger pump (engine sump type) mounted through the lid of the proprietory dye container was used. Discharge was through a clear PVC tube of known length and the tube end necessarily was weighted. Several strokes on the pump gave a brilliant but localised dye patch. This release method was used in the most recent multiple dye-patch test.

Several dye-tracer tests have been carried out at differing tidal stages and at both neap and spring tides. Two separate tests are described here.

The first test was performed at an average tide on 23.10.75 and involved release of one dye patch at high tide (0827) followed by the release of another patch three hours later (1130). Dye was tracked until dark (1750); about 2½ hours before the next high tide.

The second test was performed at a neap tide on 2.12.80 and involved the release of seven separate patches at approximately half-hour intervals starting ½ hour before high tide with the last 3 hours after high tide (see Fig. 2). Dye was tracked until low tide.

Fading dye patches were replenished by guiding the dye-boat to the desired position with instructions radioed from the tracking plane. Each radio crew carried a standby unit, and headphones proved useful when ambient noise was a problem.

Concurrently with the second test, velocity measurements were made at three depths (0.2, 0.6 and 0.8 depth), at three possible effluent release points.

Aerial photography was used to track dye movements. The topography around Waimea Inlet was unsuitable for long term, land-based tracking. When dye was confined to recognizable channels or crossed identifiable "landmarks", vertical (through floor hatch) photography was best, due to the direct plotting capability using colour positive film. In many cases, however, oblique photos were preferred so that sufficient landmarks could be framed to permit triangulation of the dye patch. A shutter speed of 1/250th second was the lowest which guaranteed blurr-free prints when using hand-held 35 mm reflex cameras through open plane windows in the presence of some air turbulence. Colour positive film in 36 frame format was preferred, for plotting flexibility and to reduce in-flight loading. In cloudy conditions, 400 ASA film was found necessary. Photography was through an open window or hatch. An ultra-violet filter, behind a polarizing filter, was routinely used to aid clarity and to reduce haze and reflection.

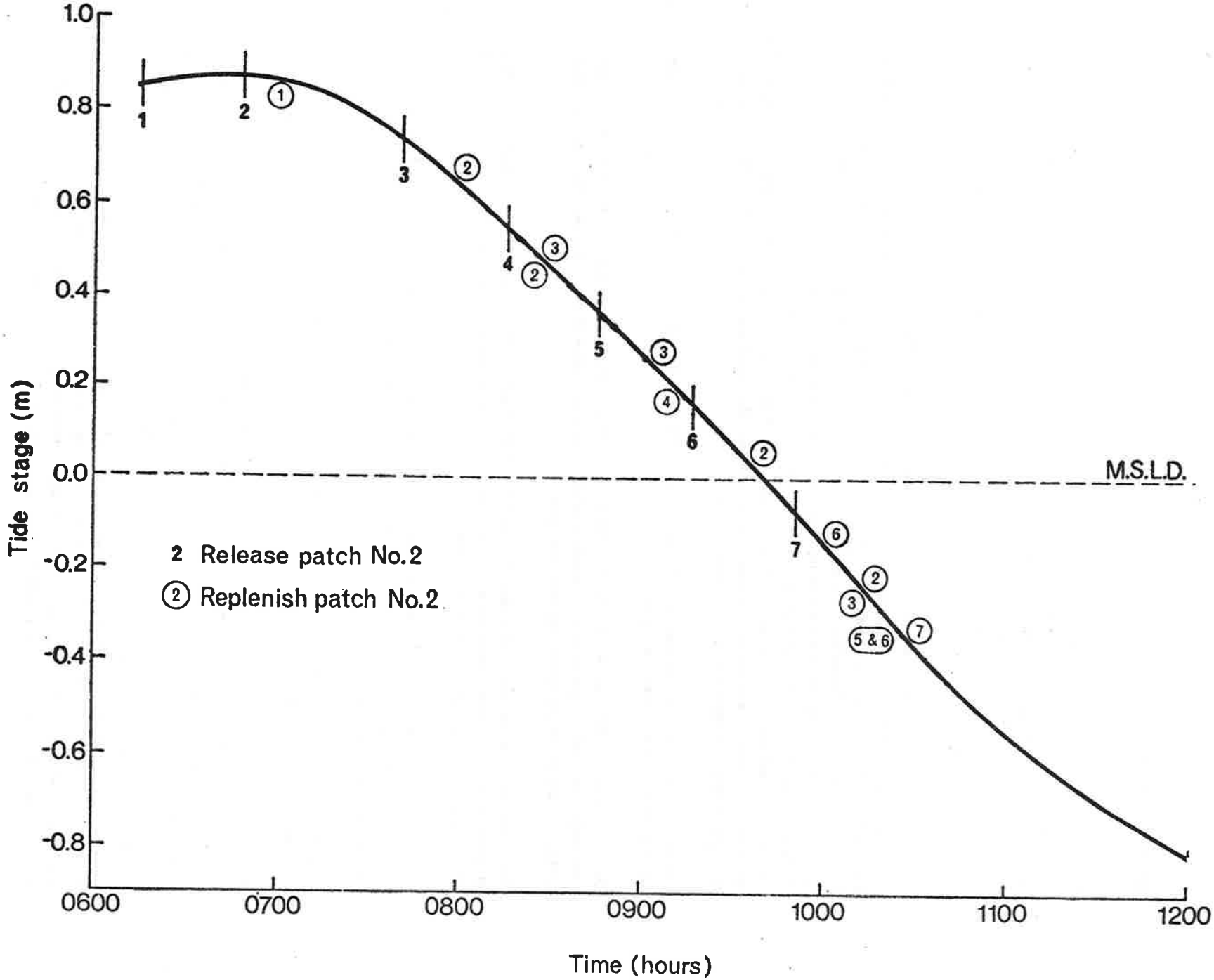



Figure 2 Times of release and replenishment, Test 2, 2.12.80

Dye photographs were mounted and patches sketched onto existing contour plans of the inlet in contrasting colours. The timing of dye patches was accurate to within ± 5 minutes.

Figure 3 shows the observed dye pathways for the first test. The striking feature of these results is that tracer swept out to sea on the falling tide returned on the rising tide. The net displacement after one complete tidal cycle would probably have been found to be very small had sampling been feasible until high tide (2030). It is important to remember that the dye patch was replentished several times during the course of the experiment. Thus some dispersion took place.

Figure 4 shows the observed dye pathways for the second test. Dye release (1) was made at 0630 (½ hour before high tide) in order to gauge "up-inlet" travel of effluent (the water-right permits discharge between ½ hour before and 3 hours after high tide). Dye was found to move up inlet and thereafter behaved in much the same manner as dye released 1½ hours after high tide. A portion of dye release (2) (made at 0705, high tide) escaped over the Rabbit Island sand banks and release (3) fragmented in the Tahunanui Beach area, with partial main channel escape, west of Haulashore Island. No other releases escaped from the eastern end of the main channel. Release (7) made at 1010 at the other end of the discharge period specified in the water right did not get much past Oyster Island, in the Upper Airport channel area.

Figure 5 shows measured water levels and velocities at the three gauging stations (Fig. 1). Ebb tide velocities did not reach an arbitrary 0.2 m/s minimum at all stations until ½ hour after high tide. Peak velocities were reached at 1.25 - 1.75 hours after high tide and similar velocity duration integrals appear at all three sites for the ½ hour to 1½ hour period.

Velocities were not derived from dye patch travel rates for this particular test although in previous studies similar estimates were obtained from dye and tidal gaugings.

## DISCUSSION

Previous dye tests had strongly suggested a lengthy residence period for the Waimea Inlet. Were Waimea Inlet "flushed" to say 60% per tidal cycle several dye patches in these tests should have completely escaped the entrance channel. The trend towards dye patch confinement within the Inlet is clearly illustrated, by comparing relative pathways of successive dye patches. These results confirm that pollutants have a relatively long residence time in the Waimea Inlet.

Often dye persisted for long periods without requiring replenishment. The patches were seen to lengthen and alter shape with little significant visual dispersion. This suggests that over much of the inlet rates of dispersion are low. When such a patch encountered a major tidal channel branch, considerable turbulent mixing was inferred by rapid colour fade, and "ragged" edges. Thus the majority of dispersion appears to occur at such channel branches. Because flow patterns indicated by dye patch changes were so variable, it was felt little could be gained from mathematically-based coefficients of channel dispersion, as are often used in river systems. Dilution rates of real effluents passing down various channel systems were seen as complex and unique variables, intimately related to topography and tidal stage.

## ESTIMATION OF COLIFORM DILUTION

Data gathered during these tests were used to investigate potential contamination of the bathing beach at Tahunanui Beach (Fig.1).

First knowledge of the likely diffuser performance and measured velocities at the discharge site were used to estimate initial dilution. Expected initial dilution for the diffuser operating in 5 m of still water is 33 times. Operational initial dilution rates could be as high as 90 times (at spring tides for a velocity of 1 m/s) and should be at least 50 times.

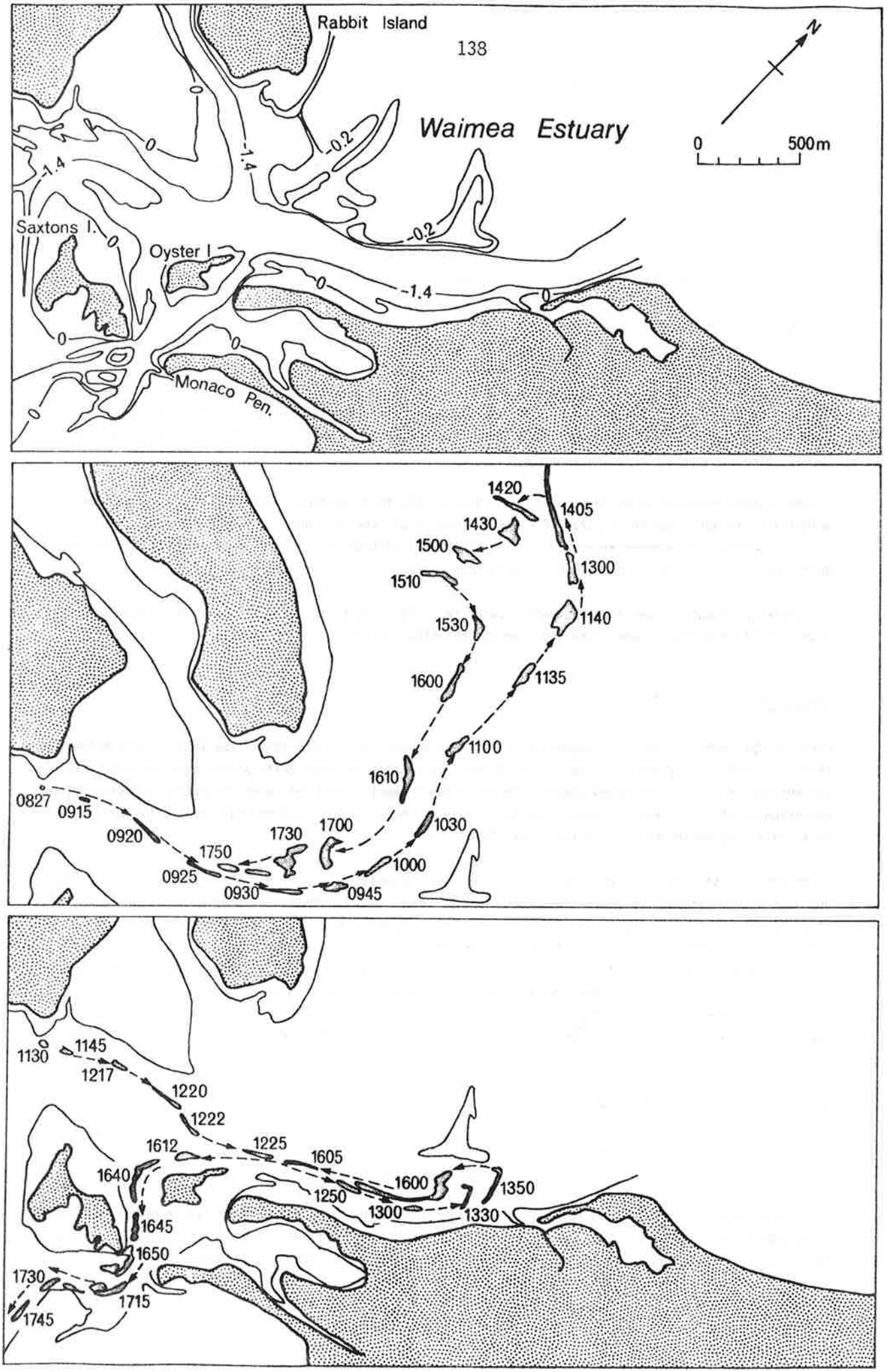
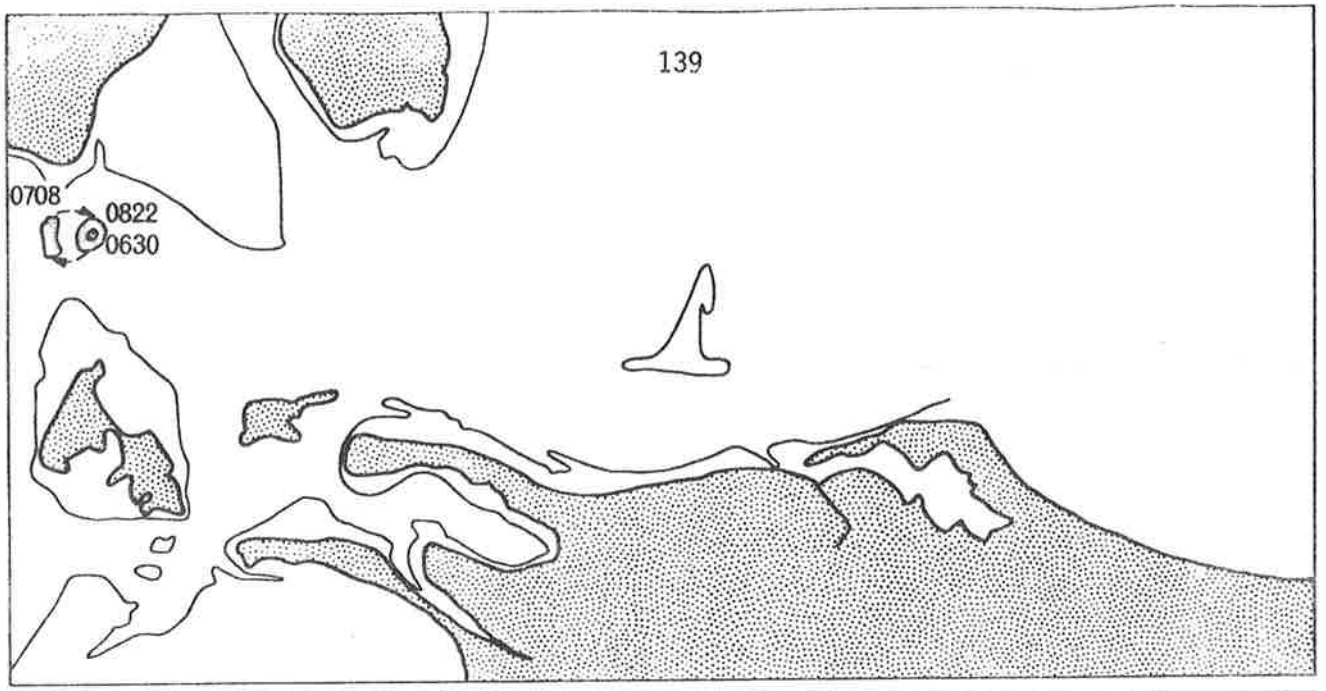
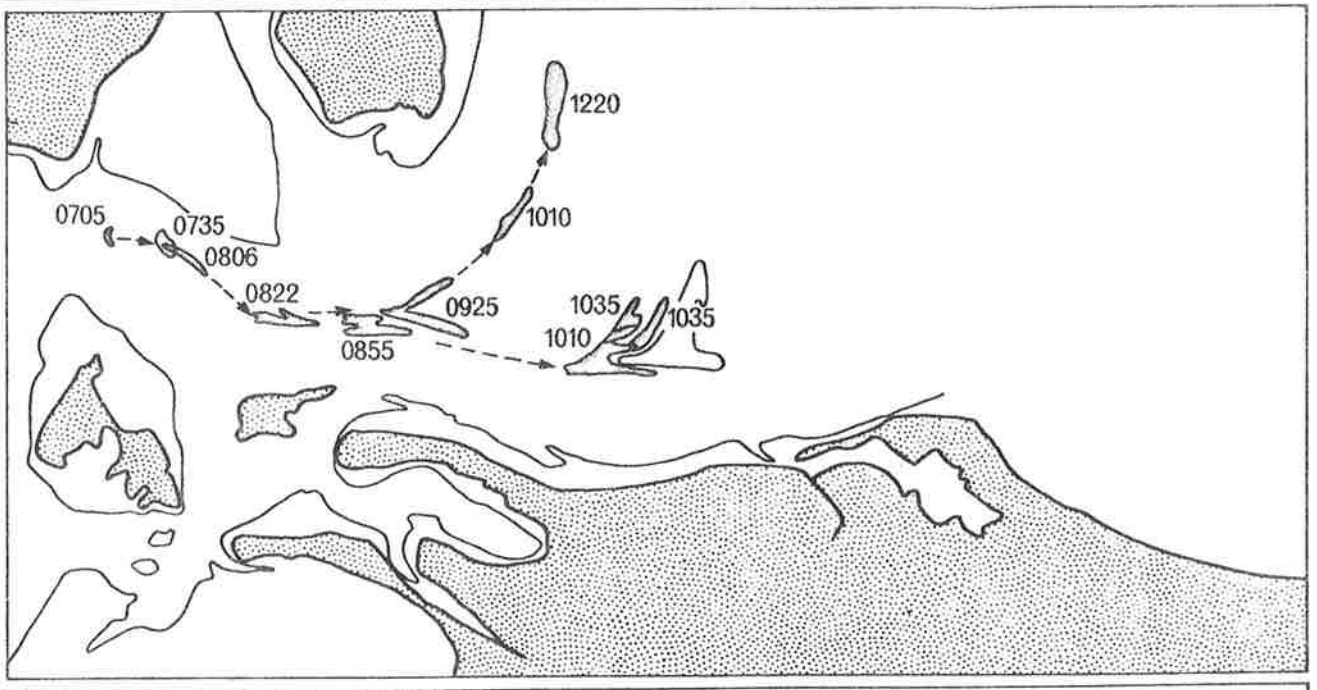



Figure 3 Test on 23.10.75. High tide 0827. Average tide.



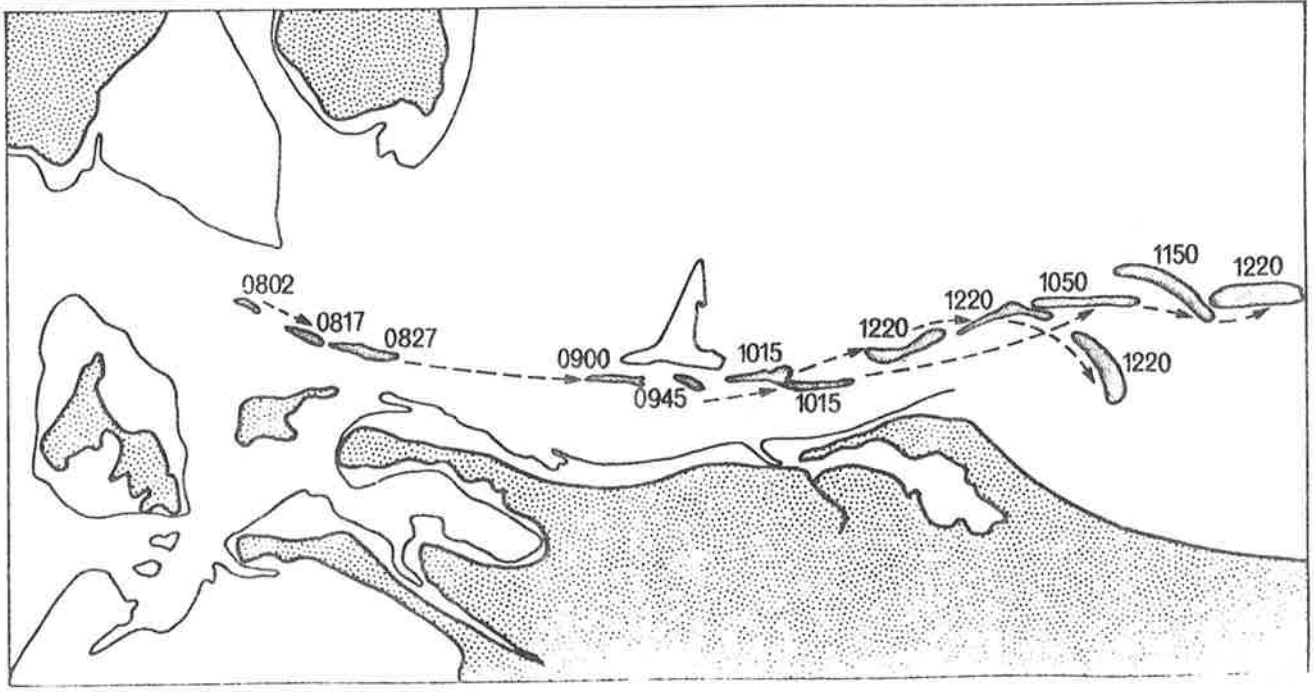
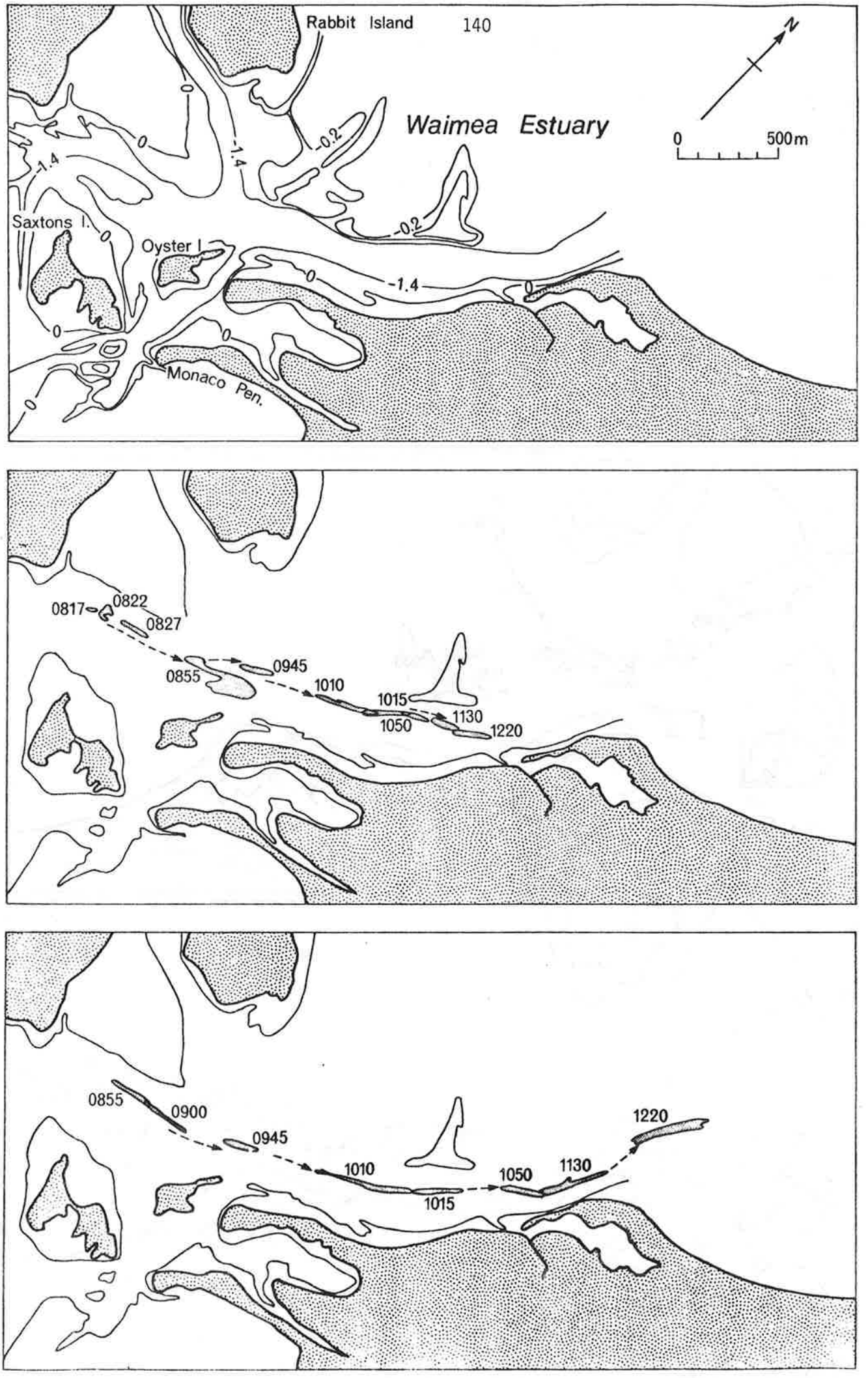
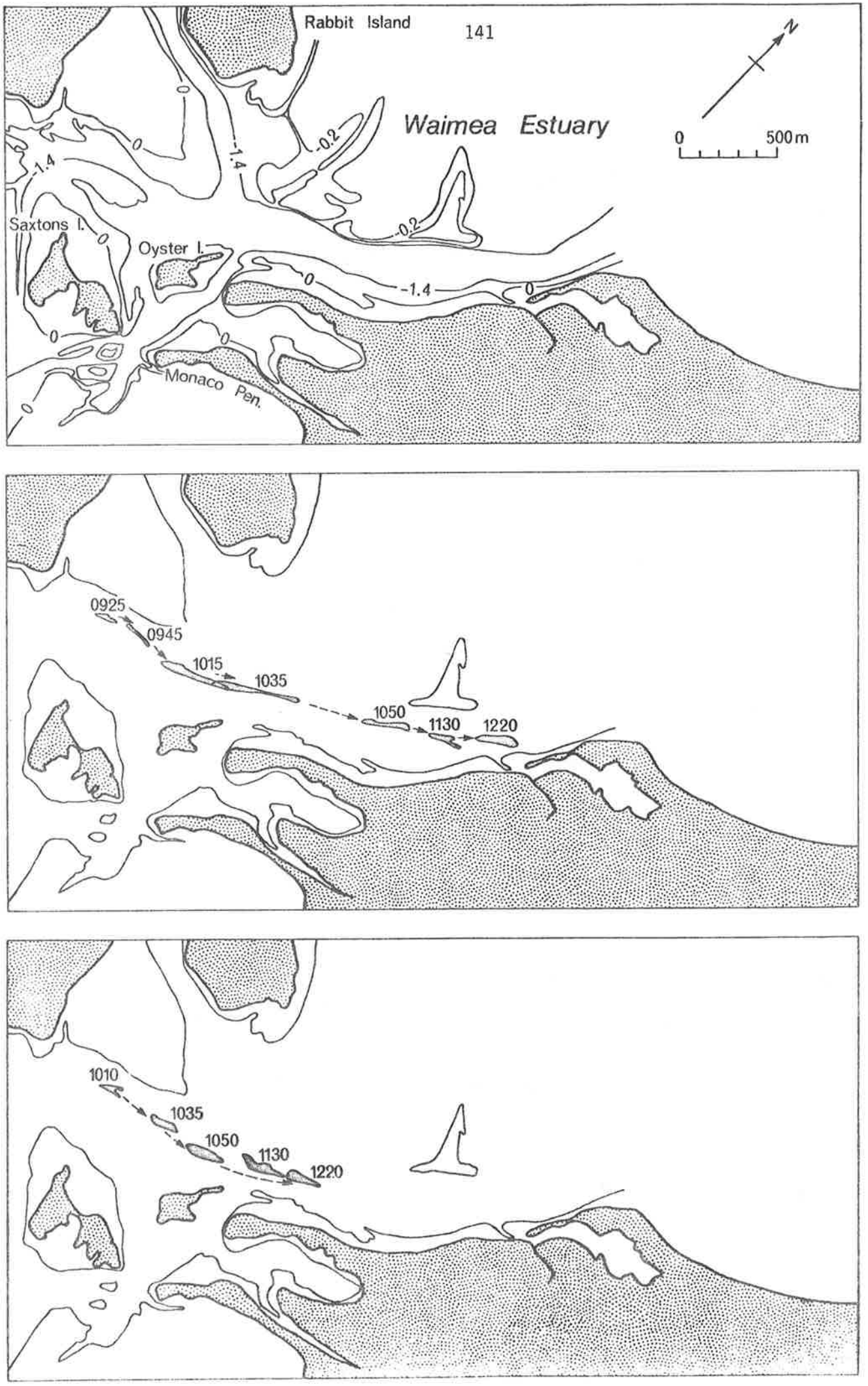




Figure 4 (3 pages) Test on 2.12.80. High tide 0705. Neap tide.


Rabbit Island
Waimea Estuary
N
0
500m
-1.4
0
-1.4
-0.2
-0.2
Saxtons I.
Oyster I
0
-1.4
0
0
0
Monaco Pen.
0
0817
0822
0827
0855
0945
1010
1015
1130
1050
1220
0855
0900
0945
1010
1015
1050
1130
1220


Rabbit Island
Waimea Estuary
N
0
500 m
-1.4
0
-0.2
Saxtons I.
Oyster I.
Monaco Pen.
0925
0945
1015
1035
1050
1130
1220
1010
1035
1050
1130
1220

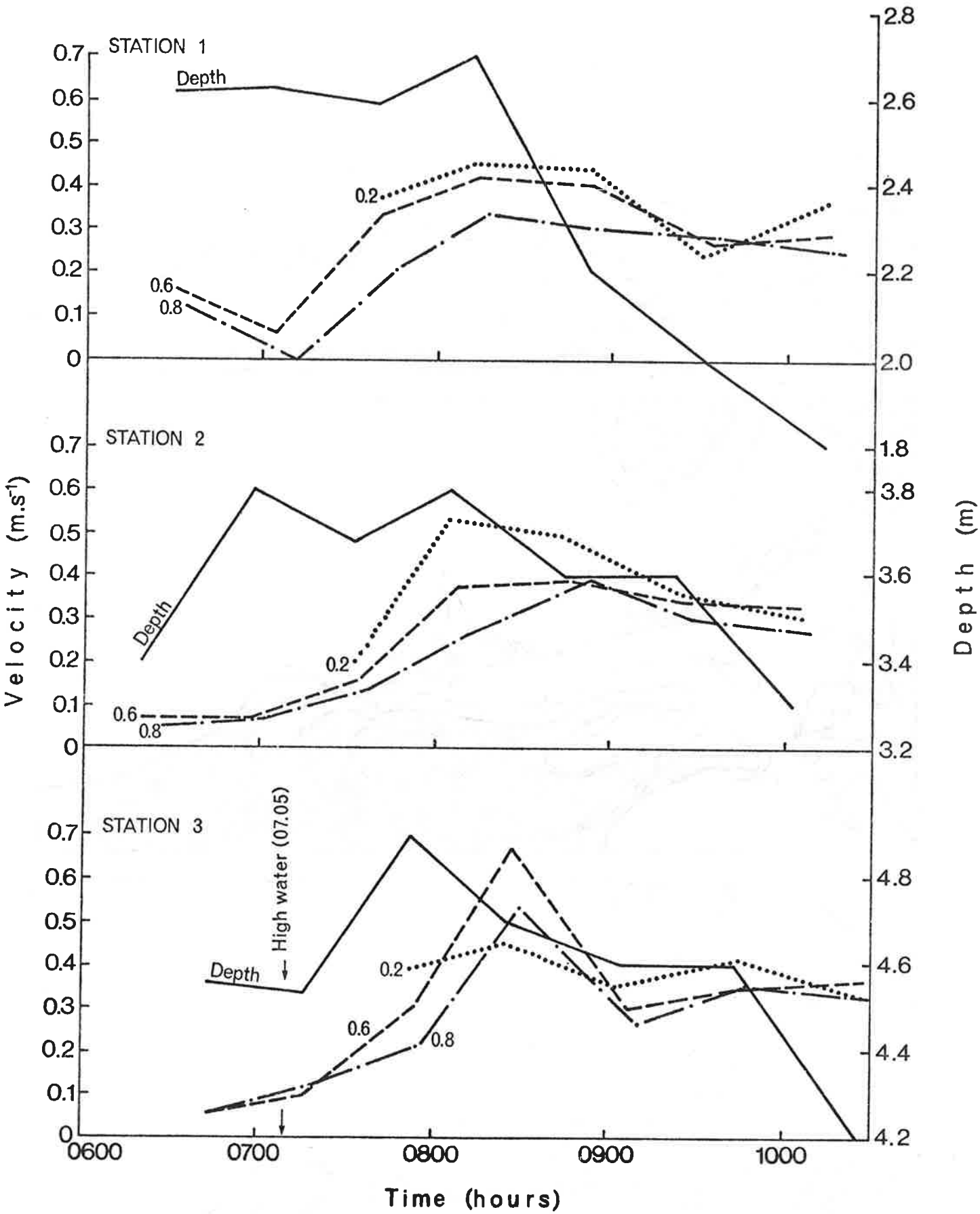



Figure 5 Velocity measurements at three proposed outfall sites.

In-channel dilution rates are still unknown due to lack of direct measurements. Measurement of coliform concentrations originating from meatworks effluent indicated a maximum in-channel dilution rate of about 10 times per 400 m, but a more conservative estimate of 100 times was assumed for the 4½ km length of channel.

Up to 35% die-off of coliforms can be anticipated in the channel.

Thus a total dilution of at least 7500 times can be expected (50 from initial dilution, 100 from in-channel dilution and 3/2 from die-off). For effluent with faecal coliform levels of $2 \times 10^4$/100 ml (the normal level), concentrations well below the specified limit (200/100 ml) can be anticipated at Tahunanui Beach, and for effluent concentrations of $5 \times 10^5$/100 ml (the level permissible for short periods) no breach is likely. Short of discharging at 3 times the maximum permitted bacterial levels, there seems little chance of beach pollution.

## SUGGESTED OPERATIONAL PROCEDURES

Operational conclusions derived from this test were:

1 Release of effluent ½ hour prior to high tide is disadvantageous due to:
(i) lowered diffuser efficiency due to low current velocities;
(ii) counter productive up-inlet effluent movement.

2 Not until ½ hour after high tide is there any arbitrary minimum current velocity of 0.2 m/s which would increase diffuser dilution rate from 33 times (design minimum) to between 50 and 90 times.

3 Timed effluent release cannot prevent effluent reaching the Tahunanui Beach.

4 Effluent released from 1 hour after high tide onwards increasingly affects the Tahunanui Beach.

Recommendations for effluent release are:

1 The discharge should occur between ½-1½ hours after the high tide. The optimum currents for diffuser operation occur at this time. Cut-off 1½ hour after high tide is recommended based on the observation that dye released later than this "stalls" in the Tahunanui Beach area.

2 The diffuser should be designed bearing in mind that worst case tidal velocities are from 0.1 to 0.4 m/s.

3 The diffuser will gain slight advantage only from siting at the most downstream station (3). Perhaps the cost of additional piping would be better spent on more rapid, pumped, discharge at the nearer station (1) (see Fig.1).

## SUGGESTIONS FOR FURTHER WORK

The tracer study details above and several earlier dye tests have proved useful to wider local coastal water management plans involving the Nelson Catchment Board.

The most obvious research directions required from the Board's point of view are:

1 Circulation patterns of southern regions of Tasman Bay. This would involve a more detailed look at the gross current patterns.

2 Dilution rate studies in the Waimea Inlet and the areas where tidal inlet waters mix with the above currents.

3 Extensive "baseline" data for areas likely to be most affected by Regional Sewage Scheme effluent as indicated by dye tracer movements.

Research in these three areas could involve:

- current recorder work;
- further dye tracer studies and/or drogues;
- instantaneous tidal velocity recordings;
- widespread bacteriological testing with a view to determining dilution rates;
- studies of nutrient levels and their changes during flood events when tremendous river-originated silt loads appear;
- other chemical analysis to discern estuarine, marine, and land-originated proportions of various water parcels;
- biological monitoring (e.g., algal biomass, shellfish analyses) of affected ecosystems.

The tests covered here have been largely a direct charge upon the Regional Sewage Scheme. As always an extension of testing programmes beyond this stage involves the funding problem and is unlikely to proceed as rapidly as desirable.

ACKNOWLEDGEMENTS

B L Williams and J C Rutherford for suggested modifications to early drafts and K Stewart for draughting the figures.

DISCUSSION

B L Williams Did you give consideration to the effect of wind in your circulation studies?
Westcott Wind recorder charts were made available to us by the Meterological Service. Our observations on these plots were that gross tidal exchange is not affected even by high (30 km) winds but that littoral dispersion is greatly affected. The common northerly wind would increase the likelihood of effluent affecting the bathing beach if this effluent is not properly dispersed throughout the water column.

T R Healy Did you consider dropping dye directly from the aircraft rather than from the jet boat and did you consider using drogues rather than dye?
Westcott Photography was carried out from a Cessna 172 flying at an altitude of around 900 m and at an airspeed just above stall in many cases. Even so, relative ground speed generally varied from 110-280 km/h depending on flight altitudes and wind velocity. 900 m is obviously too high to drop dye but to come down to say 300 m would have greatly interfered with photographic coverage. Also we were operating within airport approach corridors and the control tower was not happy with routine flight at lower altitudes. I also think 110-170 km/h at 300 m would not permit accuracy greater than several hundred meters in dye placement by "bombing", whereas jet-boat replenishment was generally within 5 m and often within 2 m of optimum. Helicopters were precluded by expense. Drogues were tried and found ineffective in depths less than 4-5 m as wave action rapidly ran them aground. Outside the inlet entrance, however, a drogue was observed to remain central to a dye patch for 2 hours whilst moving several hundred metres against the prevailing wind and wave action.

A J Sutherland The effluent from the sewage treatment plant will presumably be buoyant and therefore confined to surface layers. If this is the case, would not surface movement be more informative than current movements at 1 or 2 m below the surface?
Westcott There are two main reasons for assuming the effluent to be relatively non-buoyant.

1 The design consultant states a multiport diffuser with minimum dilution rate of 33 times is planned. Thus planned in-channel effluent concentrations are initially diluted to relatively non-buoyant level by the diffuser (in practice this dilution rate may not be achieved).

2 It would be difficult to envisage how dye could be used to follow circulation patterns in the surface water only, where replenishment is carried out. Dye poured onto the water surface will still eventually sink to deeper layers. Until our dye clouds passed through deeper waters vertical distribution normally extended throughout the water column at any rate.

In other words, I feel other techniques would be more appropriate should the studies have been directed at more precise effluent circulatory definitions rather than simply as an illustration of existing trends.

T M Hume Would it have been preferable to use (natural) salinity rather than dye tracing techniques?
Westcott These tests were carried out to illustrate existing inlet circulation patterns so that probable effluent slug pathways could be inferred. As such, the tests were not strictly speaking "dye tracer tests" at all. The absence of fluorometry apparatus prevented us from carrying out quantitative dye testing.

We were also unable to carry out quantitative saline/freshwater sampling because there is no significant freshwater input except during high river stages. Inlet salinity has been investigated and found not to vary significantly either horizontally or vertically.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre* *November 17-18, 1981*

# THE USE OF RADIOACTIVE ISOTOPES AS TRACERS IN RIVERS

*B J Barry and W J McCabe, Institute of Nuclear Sciences, DSIR, Lower Hutt*

## ABSTRACT

The use of radioisotopes for studying mixing in rivers is discussed from a practical point of view. The unique features of the technique, its advantages and disadvantages, its sensitivity and its cost are all considered so that those interested in tracing work should be able to make at least a preliminary assessment of its potential value to them.

## INTRODUCTION

Radioactive tracers may be used in river studies with the same aims in view as for any tracing study - namely to measure the mixing, dilution and residence time of some component in the river - but the practicalities of using them are unique.

Accordingly, in this paper we will try to do two things. Firstly, we will briefly describe the methods used for radioactive tracing in rivers with emphasis on the unique features. Secondly, we will discuss those experimental details and costs which need to be considered at the outset. After this we would hope that the reader interested in having some tracing work done will have enough information to make at least a preliminary decision on whether or not radioactive tracing could be of benefit.

## DISCUSSION

### The tracer

The tracer we commonly use is $^{131}I$ as iodide ion. Its half life is 8 days which is more than sufficient for river tracing tests. The energy of the radiation emitted by it is low enough that comparatively large quantities can be imported by air without needing excessively heavy containers for shielding, and high enough that it can be detected with high efficiency in the field. It is comparatively cheap and is available at 1-2 weeks notice.

Finally, iodide ion is recognised as being a good water tracer.

Other tracers such as $^{82}Br$ as bromide and $^{3}H$ as water are possible. Bromide is generally accepted to be a better tracer than iodide but the isotope may not be procured in such large quantities due to its high energy radiation. Its half life of 36 hr is probably ideal for river tracing experiments. Tritiated water ($^{3}H_2O$) would be the ideal tracer but for the fact that it cannot be detected in the field and that its use could conceivably affect measurements of environmental $^{3}H$.

Although the use of either of these tracers should not be discounted the remainder of this paper assumes that $^{131}I$ is the isotope that will be used.

### Health hazards

Account must first of all be taken of the health hazards associated with the use of radioisotopes. This is dealt with in the first instance by the design of the experiment. The greatest hazard occurs at the injection site since exposure to radiation is greatest at that point; however, only trained staff will be handling the

tracer at this time so exposure will be minimised and, in fact, it would be rare if the exposure to operators exceeded 1% of limit which can be tolerated before the health authorities are to be notified. In order to deal with emergencies such as a spill of tracer, contingency plans must be made. These might include covering the contaminated area with soil and/or isolating it with suitable barriers and warning notices. Once the injection of tracer has been made the problem becomes one of exposure to the public. This is dealt with by designing the experiment so that tracer concentrations are as low as reasonably possible; in general this can be done so that well before the end of the experiment the tracer concentrations are well below those accepted as the maximum permissible for drinking water.

All these plans to minimise health hazards must be outlined in a safety assessment which is submitted to the National Radiation Laboratory of the Department of Health who must approve it before it is possible to proceed with the work.

Measurement of tracer concentration

The tracer concentration is measured with the aid of submersible detectors which measure a radiation count rate above background proportional to the tracer concentration. To obtain quantitative results it is necessary to determine a calibration factor to convert count rates to concentrations; this is done by taking a sample from the injection solution and counting it later with the same detectors used in the experiment.

With the equipment available at Institute of Nuclear Sciences it is in principle possible to use up to seventeen different detectors. However, the logistics of doing this are very difficult so in practice fewer systems would be used.

It is possible to use either fixed or mobile detectors. A fixed detector might be attached to a stake driven into the riverbed, or it might be suspended from a float or a bridge. A mobile detector is attached, preferably with a rigid fitting to maintain it at a constant depth, to a boat which can be used to make traverses across or along the river. In either case more than one detector can be used to monitor different depths at the same point.

Because measurement is carried out *in situ* a continuous record of the passage of tracer is possible. This is important for such things as determining the exact tracer velocities, calculating the total return, establishing the validity of low concentration measurements, fitting of results to a model, and being able to detect transient events.

The measurements are independent of chemical and physical interferences, such as dissolved contaminants and turbidity, which may well affect chemical tracing methods. Radioactive tracing is thus particularly suited to polluted rivers.

The tracer has an extremely small mass - in its transport container it will occupy a few millilitres in aqueous solution - so there is no problem associated with injecting a tracer whose density may be much greater than that of the receiving waters.

Radioactive tracing is a less useful technique if it is necessary to take samples for later counting - this might be the case if the river is very shallow. Samples of at least ten litres are needed to even approach the sensitivity possible using submerged detectors and each may have to be counted for several minutes; under these conditions processing of a large number of samples would be difficult.

For reasons connected with carrying out *in situ* measurements and discussed further below, it is not possible to assign a single background count rate (equivalent to the blank value for a chemical tracer technique) for all measurements. Accordingly a considerable amount of time and care must be devoted to determining just what the background will be at a specific point.

## The sensitivity of radioactive tracing

With a chemical tracer it is possible to give a detection limit which represents the lowest concentration necessary to ensure that the tracer is detected. This limit might be, say, 1 ppb (1 $mg.m^{-3}$). This figure in itself means nothing until it is related to the amount of tracer that can be injected. Say, for arguments sake, that 100 kg of tracer is a reasonable maximum amount. Then the amount of water that could be uniformly labelled so that the tracer could just be detected is

$$V_d = \frac{100 \times 10^3 \text{ g}}{10^{-3} \text{g.m}^{-3}}$$

$$= 10^8 \text{ m}^3$$

This is a very large body of water (say 100 m wide x 1 m deep x 1000 km long) but it should be emphasised that there is no suggestion that it represents the volume that could be labelled in a real experiment since it is necessary to be able to get quantitative results over widely varying concentrations. Nevertheless, calculation of $V_d$ for different tracers will serve as a valid point of comparison.

The results of such calculations for radioactive tracers are given in Fig. 1. The calculations assume that the amount of tracer used is 200 GBq, the maximum amount that can be transported by air in a reasonable size package, and that the detector is suspended at the optimum level in water of the depth shown. If a lesser amount of tracer is to be used $V_d$ should be reduced in proportion.

The salient feature of Fig. 1 is that $V_d$ increases with increasing depth, so that for radioactive tracing it is desirable that the water of interest be as deep as possible. The variation with depth arises because of the variation with depth of three factors contributing to $V_d$.

Firstly, the background count rate, which must be subtracted from the count rate measured in the experiment, decreases with depth because of the shielding effect of the water around the detector. Secondly, because the minimum detectable count rate above background decreases along with the background, lower concentrations are detectable at greater depths. Thirdly, the calibration factor, the ratio of the measured net count rate to the tracer concentration, increases with increasing depth because the detector is counting radiation from a greater volume of water.

It can be seen from Fig. 1 that in the most favourable conditions the radioactive tracer is at least as good as the hypothetical chemical tracer discussed earlier.

## Costs

The cost of tracer is very approximately shown in Fig. 2. To this must be added manpower costs. If equipment maintenance and replacement costs are included the current INS charge out rate for time spent on the test itself is around $35 per man hour. Transport, accommodation and meals are extra. (There is provision for lower charges if the study is one of wider scientific or technical interest.) The most likely number of people involved would be three, although more might be needed if the test is expected to last a long time or if many measurement systems, especially mobile ones, are set up. The time on the job includes setting up of equipment, background runs and so on. Charges would also be made for extra materials or equipment specific to the job.

## SUMMARY

We would hope that the foregoing will enable potential users of radioactive tracing to make a preliminary assessment of whether or not it is likely to be valuable to them. Local knowledge of the site of the experiment should enable them to estimate the likely sensitivity of the technique by reading a value of $V_d$ from Fig. 1. If this compares favourably with $V_d$ for alternative tracing techniques, or if features such as the ability to monitor continuously are considered to be important, and if the costs are considered reasonable, then radioactive tracing may be judged the most suitable technique.

Fig. 1. Detectable volume with maximum tracer amount.

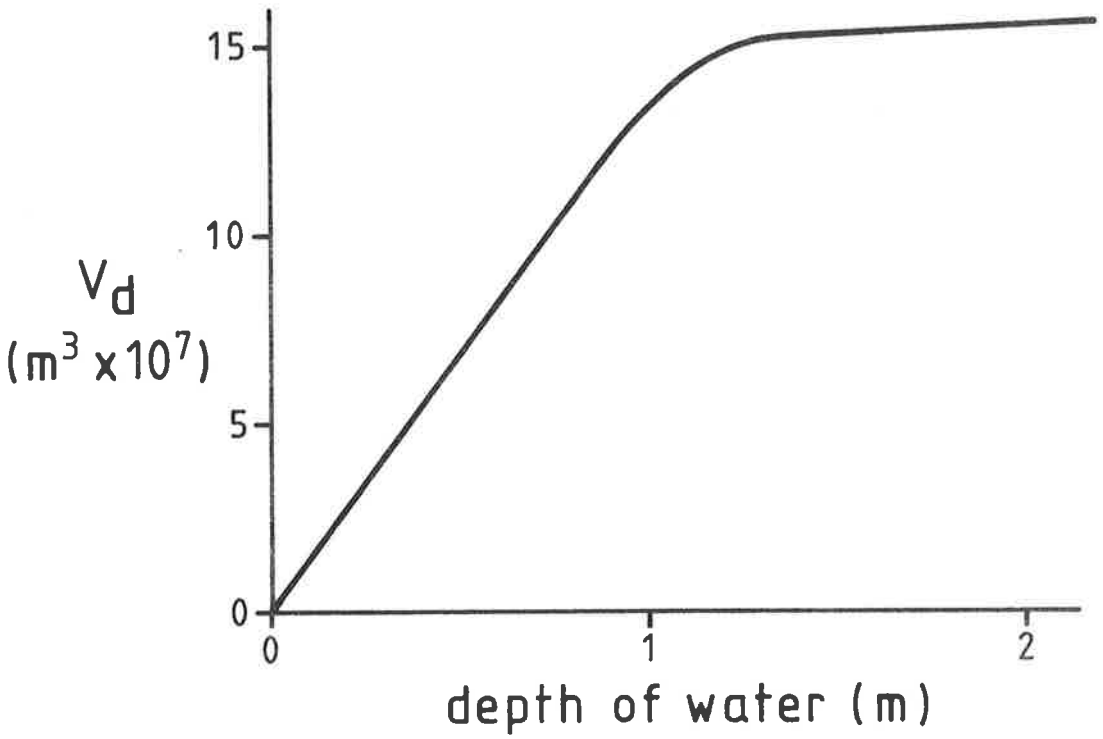
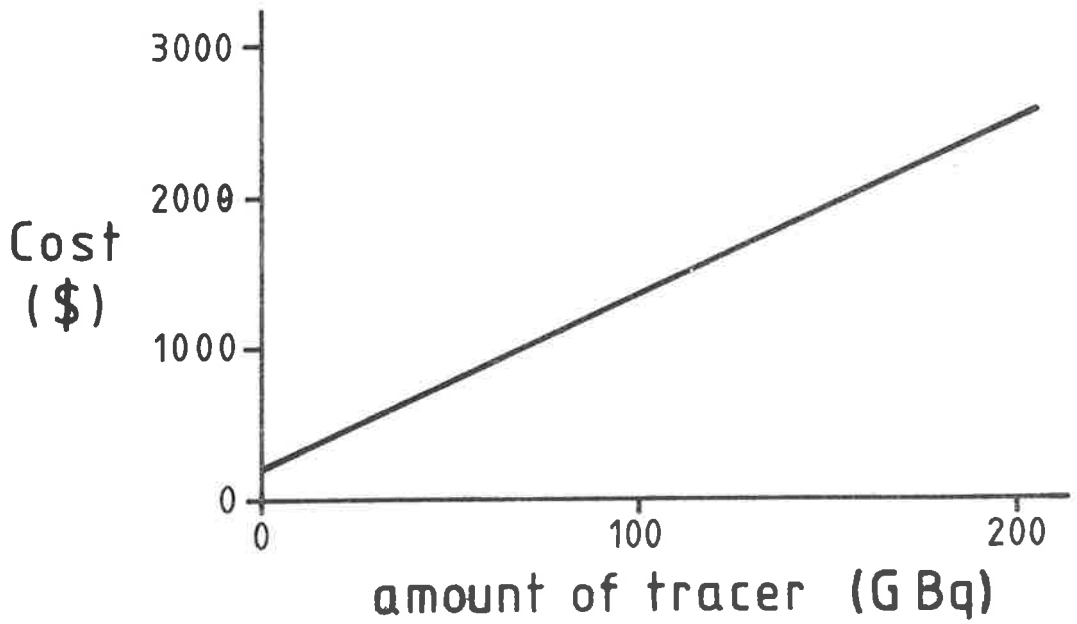



Fig. 2. Cost of tracer.



*River and Estuary Mixing Workshop*
*Hamilton Science Centre November 17-18, 1981*

# TRANSCRIPT OF DISCUSSION SESSION

*Edited by J C Rutherford*

Dr A G Barnett chaired the session and suggested as an agenda

1 Queries about "Handbook on Mixing in Rivers"*
2 Computer, physical model and field studies
3 Funding
4 Communication of results
5 Co-ordination of new projects
6 Other topics

## HANDBOOK

T R Healy There are a lot of mixing problems in estuaries. How about an estuary mixing handbook?

A G Barnett This is the first of our handbooks and we tackled the easier problem first! At the time this handbook was first mooted the Science Centre found itself being asked to advise on a number of river mixing problems. We felt those seeking advice could adequately have tackled the problems themselves if they had had such a handbook. We are well aware that there are estuary problems which are deserving of attention.

G B McBride The "Handbook on Mixing in Rivers" is the first of a series planned by the Water and Soil Science Centres: a handbook on prediction of dissolved oxygen in rivers is being written at the moment. We would like feedback from people who read and use the "Handbook on Mixing in Rivers" on whether it covers the right material and is pitched at the right level.

D A Carter The handbook appears to be pitched at the right level. What we have to do is go back to our own areas and find out whether the numbers we get from the handbook fit our particular situation.

A G Barnett I think I should make it clear that originally we intended the handbook to be published six months earlier than the workshop but publication date stretched on and on and we eventually got it out a month beforehand. We are disappointed that you have not had a better chance to work through it more extensively.

D A Carter There are one or two industries which have just installed diffusers or some other structure to mix effluent and it will be a big test to see whether or not these are really working efficiently. If we did come back in six months time with results as per the handbook and the results we have found, that would be valuable both to the Science Centre and to us.

A G Barnett Certainly for the North Island catchment authorities there will be an opportunity to comment in six months time at a planned meeting in Palmerston North.

J C Rutherford We should perhaps contemplate putting out a circular in say six months time, asking for comments on people's experience using the handbook.

---

* Rutherford, J.C. 1981: Handbook on Mixing in Rivers. Water & Soil Misc. Pub. No. 26, MWD, Wellington.

M E U Taylor Is there some merit in having a central agency which, as people around the country come up with values of say D, collects them and comes up with a compendium of appropriate values for various parts of the country? Table 4.2 p. 36 of the "Handbook on Mixing in Rivers" would be much more useful if it was a list of NZ rivers rather than overseas rivers of which we do not have a physical picture.

A G Barnett This handbook is the first step which hopefully will help put us all on the same footing. When we have come to know and love our big dees, then we can progress to the next step.

D A Carter One or two erros have been mentioned. Are there others?

J C Rutherford We plan to produce an errata sheet in due course.

A J Sutherland Can I make a plea for the use of SI units. There was considerable confusion yesterday in the working session arising from the use of centimetres and decimetres.

A G Barnett What are the legal ramifications of a standard handbook such as this in water right hearings?

D A Carter I cannot see much of a problem. Someone giving evidence would presumably quote the handbook as a reference work in the same way they would do so for, say, a COD analysis.

M E U Taylor NWASCO has tried to give guidance to the courts from time to time by endorsing various codes of practice. This does not give them formal legal status but gives them a very strong push. You can really only quote the handbook as background supporting your evidence, just like any other text book.

FUNDING

A J Sutherland and R A Heath We would be interested in hearing people's experiences with funding, and learning how present investigations are being funded.

K G Westcott For the Waimea Inlet, the last tests were a direct charge on the consultants. I think that for some of the earlier tests in 1975-76 there may have been some funding from NWASCO.

J Kitto The Taranaki Ring Plain Survey is supported one third by the Taranaki Catchment Commission and two thirds by central government. It is a general data gathering operation, and for specific investigations it would be the people applying for the water right who would pay.

B W Gilliland Part of the money for the Manawatu River Study came through a GA38* grant from central government. There was additional assistance from the Water and Soil Science Centre, Hamilton, whose costs fell where they lay.

B Knowles Part of the money for the Manawatu River Study came from the Board's Water Resources funds. GA38 grants are not specifically for one-off studies such as those discussed over the last two days but are intended for studies which are part of, say, 3-5 year resource projects.

M E U Taylor Really where a lot of the problems lie is that the legislation does not make enough provision for funding of specific studies. This deficiency has been realised for some time.

R O Carr I feel there is plenty of scope for funding these types of projects provided you do your public relations right. In Marlborough we had an estuarine mixing problem involving many people : County Council, Harbour Board, Waitaki NZR, a dairy factory, Montana Wines and several others. The problem was seen as a general environmental problem. By widespread publicity and the fact that several of them had been in a modicum of trouble with the Board, we managed to get contribution from all of them. Central government also contributed to the study through the GA38 grants.

---

*GA38 grants are made from NWASCO to Regional Water Boards for resource surveys and special studies.

R A Heath Who in fact does the investigation work for these projects, say an outfall?

A G Barnett As I understand it, it is up to the applicant to satisfy the Regional Water Board that the discharge is acceptable and they must make whatever arrangements are necessary.

M E U Taylor Some water boards have been moving into the area of doing work for the applicant. This is possible but they have to be careful what legal stance they take in doing the work in respect of recovering costs. If they do the work as general background to the water right, then costs are laid down in the Act and are often very low. However, the board can legally require the applicant to provide certain information and the applicant may retain the board as a consultant to do these studies.

R O Carr and D A Carter Once you have granted a water right and are monitoring, then you can no longer charge the applicant.

D A Carter The board has to be careful not to set itself up as judge and jury with an application.

A G Barnett Another problem arises when the board realises there is a need for 2-3 years of investigation before anyone applies for a water right.

M E U Taylor If the board knows this is coming up it can warn the applicant that certain information is going to be required and point out that it is the applicant's responsibility to provide that information.

R O Carr The principle is the same here as for underground water movement. There is the need for general background information for water management.

## CO-ORDINATION AND COMMUNICATION

R A Heath I would like to draw people's attention to the newsletter "Coastal Processes" as one method of communication.

N R Hall I would find a list of names and addresses of participants at this workshop useful.

A G Barnett Particularly in the coastal and estuary field, historically there seems to have been a number of different organisations working on various problems in an *ad hoc* way. It would be difficult to co-ordinate their activities formally but we can communicate at a staff level at meetings such as this. The Technical Group on Water, set up by Institution of Professional Engineers, is another method of communication.

R A Heath Meetings such as these are the corner stone of communications and co-ordination and we must urge our employers to let us attend at least one of these each year.

A G Barnett Certainly some of the annual conferences of the learned societies are often very diffuse. Much though we might like to, our interests in the Science Centre are too broad to allow us to run another workshop like this next year; but certainly in two or three years time we could.

T R Healy I don't think people should overlook the importance of the annual conferences of societies such as Hydrological Society, Marine Sciences Society etc. Many of the papers presented here could equally well have been presented at one of them. It is interesting to note that many consultants are realising this and attending.

J C Rutherford We have been talking about our communication of results but are we well placed to co-ordinate our resources when a major new study like a Pelorus Sound or Whangarei Harbour Study comes along? Can we make sure that the most appropriate people are involved with the most efficient deployment of resources?

T R Healy Do you think it is desirable to have a "Big Brother" committee saying who will do this work and who that?

R A Heath I don't think we need be too worried about "Big Brother". In many cases studies are done in isolation, for example Pelorus Sound, and it would be desirable to have some co-ordination between disciplines. It would be a difficult task.

R O Carr The appropriate regional water board should also be consulted.

D A Carter On mining in the Coromandel we heard that various agencies were interested in the problem. We organised a co-ordinating committee attended by all interested groups and this should enable everyone to keep in touch.

M E U Taylor It is NWASCO policy that where a regional water board is looking at a problem, we support the establishment of a technical committee or liaison committee and are prepared to provide whatever technical backing it may request.

R O Carr It is possible that some people here are not aware of the interests and activities of catchment boards in areas like landuse capability.

M E U Taylor Sometimes it is easiest for various government agencies to come together under the aegis of a catchment board.

J C Rutherford Certainly it has been our experience that *ad hoc* catchment board technical committees, for example the Waikato River and Manawatu River Technical Committees, have provided a very useful forum for communication and co-ordination. Another more formal example is the Upper Waitemata Harbour Study.

P W Williams For some studies, for example, a freezing works outfall, you probably do not need a co-ordinating committee. If you are interested in the biology, sedimentation and hydrology then your study might benefit immeasurably from a co-ordinating committee. The next problem is where do you get the man-power? In many cases the appropriate people are committed and/or at the other end of the country.

A G Barnett There seems to be consensus that the water boards know the local scene and are the best agencies, in most cases, to co-ordinate new studies.

P W Williams Perhaps the major difficulty is to perceive the important problems and convince someone that it needs to be tackled. I suggest that NWASCO has the national overview, and that it should be their job.

M E U Taylor NWASCO prefers to see problems tackled by local interests, as in the Upper Waitemata Harbour Study or mining in Coromandel and Southland, by asking the local bodies to co-ordinate the studies.

R A Heath What is the connection between NWASCO, National Research Advisory Committee (NRAC) and Univeristy Grants Committee (UGC)?

M E U Taylor We ensure parity with UGC grants to students, and maintain considerable dialogue with NRAC. NRAC allocate some staff slots within Water and Soil Division, MWD. First priority for support, as stated by government policy, is for research likely to bring in overseas funds, and the second for areas likely to help unemployment.

The discussion session closed after an hour and Dr M E U Taylor gave a brief address to close the workshop.

*River and Estuary Mixing Workshop*
*Hamilton Science Centre* *November 17-18, 1981*

# APPENDIX A : BACKGROUND NOTES FOR DISCUSSION SESSION

*Prepared by Science Centre Staff, MWD, Hamilton*

## A RIVER MIXING

### A1 Preliminary desk studies

Simple (Fickian uniform flow) models are available (see for example, "Handbook on Mixing in Rivers"). Choosing coefficients for desk studies is difficult, but upper and lower bounds can usually be estimated.

### A2 Computer model studies

Research is well advanced for modelling dead-zones, and non-uniform flow. However, these models have not been applied widely and few coefficients are available for desk studies. It is desirable to apply these models to real problems as they arise, doing whatever field studies are needed, and to collate model coefficients and methodologies.

### A3 Physical model studies

These are often costly, but have proved useful for some problems (e.g., Huntly Power Station outfall, Hamilton sewage outfall). There can be scaling difficulties.

### A4 Field studies

There is considerable experience in New Zealand using dye (viz, Waikato, Manawatu and Taranaki Rivers) and radioactive tracers (viz, Institute of Nuclear Sciences work). The logistics in river studies are usually tolerable. Computer modelling is useful to extrapolate field results to conditions outside the range studied (e.g., flow, location, mass injected etc.).

## B ESTUARY MIXING

### B1 Preliminary desk studies

There are several ways of estimating the so called "flushing time" of estuaries. In some instances the concept has been misunderstood and the estimates misused. There is a need to produce guidelines for their application in New Zealand estuaries.

### B2 Computer model studies

A small number of computer model studies have been conducted in New Zealand, but generally applicable tools are not yet available. There are a number of people in New Zealand with the skills and a knowledge of the overseas literature who could contribute to the development of such tools and/or contribute to specific projects. It remains to develop and use these skills.

### B3 Physical model studies

No physical estuary mixing models have been described at this workshop. Several organisations in New Zealand have experience with physical models of some aspects of estuarine hydrodynamics (e.g., Universities of Auckland and Canterbury; Central Laboratories, MWD; Otago and Whangarei Harbour Boards).

Experience overseas suggests that the best approach to many estuary mixing problems is to develop physical and computer models simultaneously. At present in New Zealand those with physical and computer modelling skills are scattered and will need to be better co-ordinated if major estuary mixing problems are to be tackled and solved using locally available expertise.

### B4 Field studies

No physical or computer modelling is possible without reliable field data. Various agencies have relevant specialist skills (e.g., Institute of Nuclear Sciences, (radiotracers), Oceanographic Institute (currents, bathymetry), MWD (dye tracers, currents, bathymetry)). Given good co-ordination and co-operation these skills could be deployed on particular important estuary mixing problems.

## C GENERAL

### C1 How new problems can be approached

The following approach appears to be working quite well:

(a) conduct preliminary desk study using whatever data and models are available;
(b) decide on cost/benefits of further study;
(c) if necessary, do one or a combination of:

  (i) field tracer studies,
  (ii) physical model studies,
  (iii) further computer modelling studies.

It should be possible to refine estimates made in C1(a) and C1(c)(iii) by:

(i) accumulating data and coefficients as various case studies are done;
(ii) gaining experience with dead-zone and non-uniform flow models.

It is still likely that recourse will need to be taken to steps C1(c) for important problems.

### C2 Science Centre role

The Science Centre is in a position of being able to offer technical advice on mixing problems. At present our policy is:

(a) to encourage clients (notably RWB's) to undertake steps C1(a) and C1(b) themselves and then to discuss ways and means of undertaking step C1(c) if necessary;
(b) to undertake or contract out (e.g., to Universities) research where needed which will produce tools applicable nationally;
(c) to minimise direct involvement with planning application hearings and appeals.

## APPENDIX B : LIST OF PARTICIPANTS

| | |
|---|---|
| Mr A K Attwood | District Office, MWD, Hamilton |
| Mr R Aves | Hauraki Catchment Board, Te Aroha |
| Dr A G Barnett | Water & Soil Science Centre, MWD, Hamilton |
| Dr J W Barnett | NZ Dairy Research Institute, Palmerston North |
| Dr B J Barry | Institute of Nuclear Sciences, DSIR, Lower Hutt |
| Mr I Basire | TSE Group Consultants Ltd, Wellington |
| Dr R G Bell | Water & Soil Science Centre, MWD, Hamilton |
| Mr D Bewick | Head Office, MWD, Wellington |
| Dr J D Boyle | University of Exeter, UK |
| Mr D G A Brickell | Auckland Regional Authority, Auckland |
| Mr J M Campbell | Alliance Freezing Co, Invercargill |
| Mr D N Campin | NZ Forest Products Ltd, Tokoroa |
| Mr B Carlisle | Hamilton City Council, Hamilton |
| Mr R O Carr | Marlborough Catchment Board, Blenheim |
| Mr A G Carter | Hamilton City Council, Hamilton |
| Mr D A Carter | Hauraki Catchment Board, Te Aroha |
| Mr D M Carver | Christchurch Drainage Board, Christchurch |
| Dr D A Challis | Auckland Regional Authority, Auckland |
| Mr D J Clark | TAM Dept., University of Auckland, Auckland |
| Mr R Cornforth | Commission for the Environment, Wellington |
| Dr A B Drysdale | Lincoln College, Christchurch |
| Mr J Endert | Murray North and Partners Ltd, Hamilton |
| Mr M C Freeman | Massey University, Palmerston North |
| Mr R W Fullerton | Steven Fitzmaurice and Partners, Auckland |
| Mr B W Gilliland | Manawatu Regional Water Board, Palmerston North |
| Dr D G Goring | Water & Soil Science Centre, MWD, Christchurch |
| Mr M G G Gurr | Hamilton City Council, Hamilton |
| Mr N R Hall | Electricity Division, Ministry of Energy, Wellington |
| Mr C R Hannah | Waikato Valley Authority, Hamilton |
| Dr R A Heath | NZ Oceanographic Institute, DSIR, Wellington |
| Dr T R Healy | Dept. Earth Science, University of Waikato, Hamilton |
| Dr T M Hume | Water & Soil Division, MWD, Auckland |
| Mr L Jacobson | Whangarei City Council, Whangarei |
| Mr R M S Johnston | University of Waikato, Hamilton |
| Mr J Kitto | Taranaki Catchment Commission, Stratford |
| Mr B Knowles | Manawatu Catchment Board, Palmerston North |
| Mr I G G MacBean | District Office, MWD, Hamilton |
| Mr G B McBride | Water & Soil Science Centre, MWD, Hamilton |
| Mr G J MacDonald | Steven Fitzmaurice & Partners Ltd, Christchurch |

| | |
|---|---|
| Dr P N McFarlane | Massey University, Palmerston North |
| Mr J J McIntosh | Bay of Plenty Catchment Commission, Whakatane |
| Mr A J McNulty | Dept. of Civil Engineering, University of Canterbury, Christchurch |
| Dr M J O'Sullivan | TAM Dept., University of Auckland, Auckland |
| Mr L Parker | Northland Catchment Commission, Whangarei |
| Mr K E Parnell | Geography Dept., University of Auckland, Auckland |
| Mr R K Penney | TAM Dept., University of Auckland, Auckland |
| Mr J Quinn | Head Office, Water and Soil Division, MWD, Wellington |
| Dr A M D Rennes | Waikato Valley Authority, Hamilton |
| Dr J C Rutherford | Water & Soil Science Centre, MWD, Hamilton |
| Mr D Smith | NZ Co-Op Dairy Co., Hamilton |
| Dr A J Sutherland | Dept. Civil Engineering, University of Canterbury, Christchurch |
| Dr M E U Taylor | Head Office, Water & Soil Division, MWD, Wellington |
| Mr W A Taylor | Bay of Plenty Catchment Commission, Whakatane |
| Dr P Tortell | Commission for the Environment, Wellington |
| Dr E M Valentine | Central Laboratories, MWD, Lower Hutt |
| Ms M Van Roon | Auckland Regional Authority, Auckland |
| Mr K G Westcott | Nelson Catchment Board, Nelson |
| Dr R J Wilcock | Water & Soil Science Centre, MWD, Hamilton |
| Mr D R Wilkie | Steven Fitzmaurice & Partners, Christchurch |
| Mr B L Williams | Water & Soil Science Centre, MWD, Hamilton |
| Prof. P W Williams | Geography Dept., University of Auckland, Auckland. |

## WATER AND SOIL TECHNICAL PUBLICATIONS

1. Liquid and waterborne wastes research in New Zealand. Sally Davis ($1) 1977
2. Sampling of surface waters. M. Kingsford, J. S. Nielsen, A. D. Pritchard, C. D. Stevenson ($1) 1977
3. Water quality research in New Zealand 1976. Sally Davis ($1) 1977
4. Shotover River catchment. Report on sediment sources survey and feasibility of control. 1975 (out of stock) 1978
5. Late Quaternary sedimentary processes at Ohiwa Harbour, eastern Bay of Plenty with special reference to property loss on Ohiwa. J. G. Gibb ($1) 1978
6. Recorded channel changes of the Upper Waipawa River, Ruahine Range, New Zealand. P. J. Grant (out of stock) 1978
7. Effects of domestic wastewater disposal by land irrigation on groundwater quality of the central Canterbury Plains. G. N. Martin and M. J. Noonan ($1) 1978
8. Magnitude and frequency of floods in the Northland-Auckland region and their application to urban flood design. J. R. Waugh ($1) 1978
9. Research and Survey annual review 1977. (out of stock) 1978
10. The problem of coastal erosion along the "Gold Coast", western Wellington, New Zealand. J. G. Gibb (out of stock) 1978
11. The Waikato River: A water resources study. ($12) 1979
12. A review of the habitat requirements of fish in New Zealand rivers. Compiled by D. F. Church, S. F. Davis, and M. E. U. Taylor ($3) 1979
13. The Ruahine Range: A situation review and proposals for integrated management of the Ruahine Range and the rivers affected by it. A. Cunningham and P. W. Stribling ($5) 1978
14. A Survey of New Zealand peat resources. A. Davoren ($10) 1978
15. Effects of urban land use on water quantity and quality: an annotated bibliography. I. Simmers ($5) 1980
16. Research and Survey annual review 1978. (free) 1979
17. Investigations into the use of the bacterial species *Bacillus stearothermophilus* and *Escherichia coli* ($H_2S$ positive) as tracers of groundwater movement. L. W. Sinton ($1.50) 1980
18. A review of some biological methods for the assessment of water quality with special reference to New Zealand. Part 1. ($3) 1979
19. The frequency of high intensity rainfalls in New Zealand, Part I. A. I. Tomlinson (free) 1980
20. Regional flood estimation in New Zealand. M. E. Beable and A. I. McKerchar ($8) 1982
21. Coastal hazard mapping as a planning technique for Waiapu County, East Coast, North Island, New Zealand. J. G. Gibb ($5) 1981
22. A review of some biological methods for the assessment of water quality with special reference to New Zealand. Part 2. ($4) 1981
23. Hydrology of the catchments draining to the Pauatahanui Inlet. R. J. Curry ($3) 1981
24. Potential for contamination of the Heretaunga Plains aquifers ($10) 1982