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MATHEMATICAL MODELS FOR POLLUTION

CONTROL IN THE USK ESTUARY .

Michael Werner Rogers, B.Sc.(Jt.Hons.)

Thesis submitted to the University of Aston 1977 .

Being the report of research supported by SRC, Usk River Authority
and the IHD Department of the University

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MATHEMATICAL MODELS FOR POLLUTION CONTROL IN THE USK ESTUARY

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Dept: IHD/Civil Engineering

Submitted for the degree of Ph.D. , Aston University December 1977

Brief Summary of Submitted Work

The now Usk River Division of the Welsh National Water Development Authority was required to make several major policy decisions on water quality within the Usk Estuary. These were highly relevant to a series of large capital projects and , after a public enquiry , the Secretary of State for Wales required a scientific investigation to be initiated that would rationalise the method of arriving at the decisions.

The process of biological degradation of pollutants would have to be understood. A number of models were reviewed , and also the attitude to the use of models by representatives of the public and management.

Three types of models were developed for differimg requirements of the overall management function : a Steady State Model for approximate trends that could be available quickly and can be adapted for use in economic/cost models, a time dependant multi-dimensional model for considering short term effects and dissipation of perturbations (pollution 'incidents') , and a semi-stochastic/deterministic model to utilise field data and generate realistic confidence limits on management projections.The models were so formulated as to be flexible and as interchangeable as possible.

A number of projected capital plans were simulated by management for load variants and flow reductions. Problems were encountered because of the rapidity of the worlds second largest tidal rise,the antiquity of Newport's drains system and the sometime overriding influence of the Severn Estuary. Telemetric monitoring proved inefficient because of field conditions.The awareness of management was developed in the appreciation of modelling and computing, to self sufficiency if required.Many routines were provided for the routine interpretation of data by the Pollution Control Department.The need for common modelling policy was recognised by WNWDA and these models were to be made available nationally through the Water Data Unit of the Department of the Environment for access at divisional level as an aid to management.

KEYWORDS MATHEMATICAL MODELLING, STEADY STATE, TIME DEPENDANT, STOCHASTIC, APPLIED MANAGEMENT

INTRODUCTORY NOTE

This work is submitted in two parts so that it can also be used as a reference manual by the model user.

Volume 1 consists of model descriptions and formulations and general topics.To maintain independant chapters, the chapter bibliography appears after each chapter in the body of the volume.

Volume 2 consists of formal descriptions of the model software and data layout for input, with flow logic. Other routines used are also described.

Dedication

To my family and that of man.

ACKNOWLEDGEMENTS

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The Science Research Council and the Usk River Authority for main financial support.

Dr. T R E Chidley , Senior Lecturer in Civil Engineering,Aston University as Principal Academic Supervisor.

Mr. H A Hawkes of the Life Sciences Dept.,Aston University as Associate Supervisor.

Mr. M D Hunter , Pollution Control Officer of the Usk River Authority as project instigator and Industrial Supervisor.

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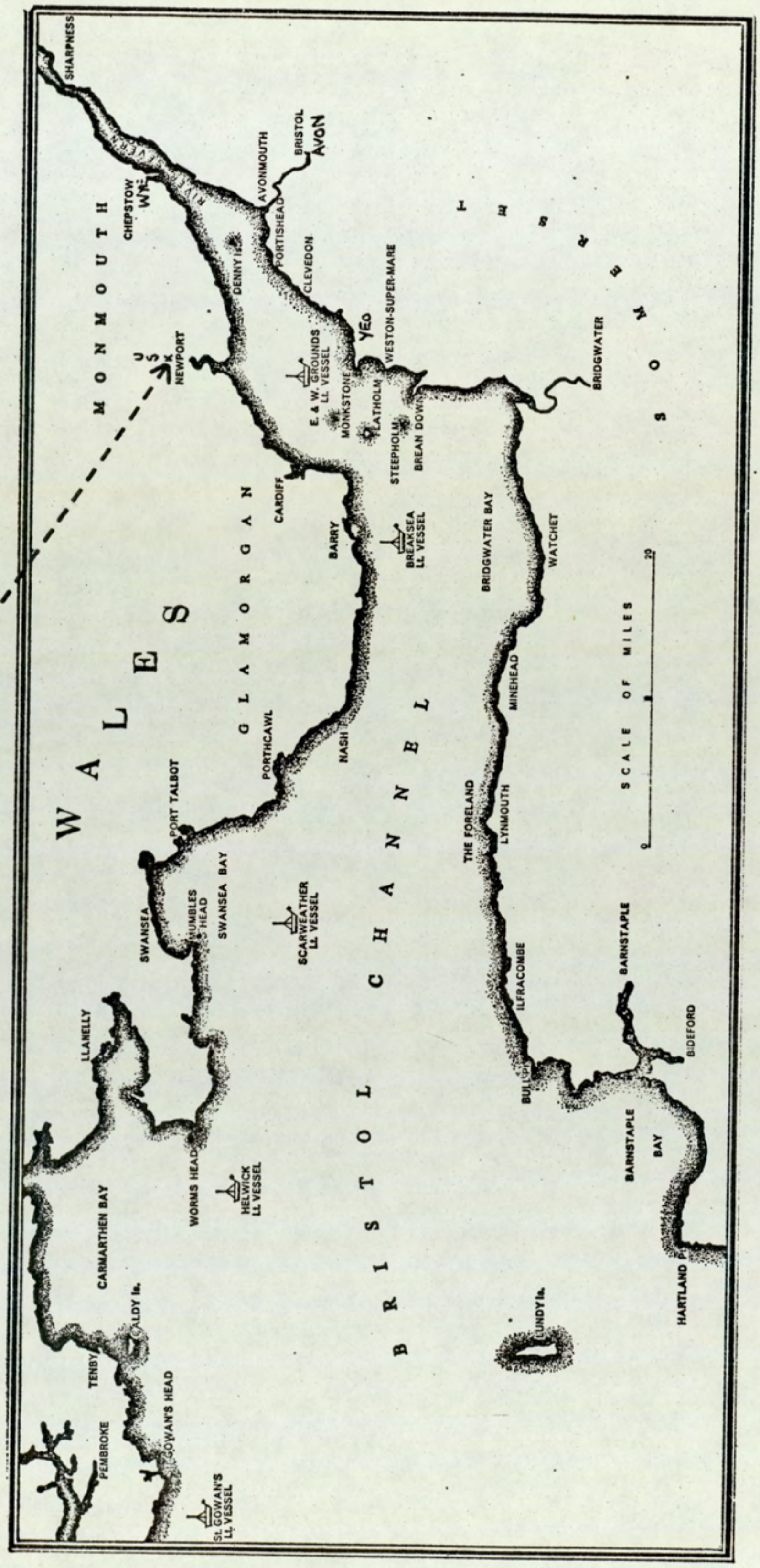
This is the final report of project B72-13003 of the Science Research Council.

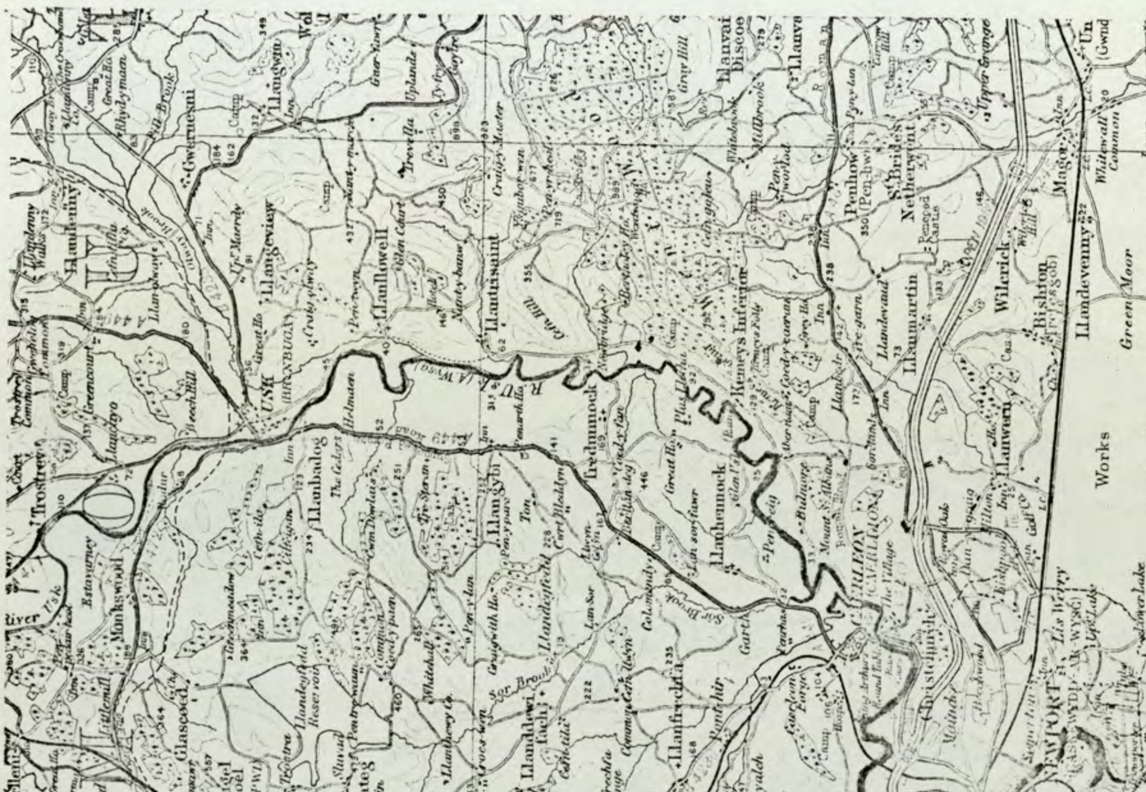
THE POSITION OF THE USK AND SEVERN ESTUARY

Area of Study



The Geographical Features of the Usk Estuary

Scale 1 : 126,720



CONCLUSIONS and

SUMMARY

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The project has suceeded in the broad aim to supply a management tool in the form of a mathematical model to the now Usk River Division of the Welsh National Water Development Authority. Work to make this tool available nationally has been initiated.

Three models were developed in parallel for differing management aspects :

1. A Steady State Model developed in the short term to provide a tool within the duration of the project and provide a model that offers trend predictions over long term averages,to so generate input to economic models of water resouce management.
2. A mixed dimension time dependant model with a hydrodynamic and Pollutant Transport phase . This allows simulation of the suspected effects of the Severn Estuary as a reservoir for the Usk Estuary. Predictions are extremes of quality within tidal cycles.
3. A deterministic-stochastic mixed model to provide management with a tool for estimating variations and thus confidence limits on predictions .Also allows the simulations of perturbations on the whole system.

Some subsidary aspects of the management function of the division were also developed :

1. Provision of software for analysing data generated by a network of Automatic Monitoring Water Quality Stations.
2. Increasing Management Awareness of the potential of Computer use and familiarisation with the methodology involved .
3. To build up a software library to assist the Pollution Control Department in its analytical data "interpretive" role. This has been largly superceded by a national system from the Department

of the Environment on very similar lines to pilot proposals put forward.

There has been a lack of cohesive data to validate the models in a wholly satisfactory manner. Such as it is , validation has been only with piecewise continuous data , mainly historic and so lacking any overall statistic strategy.

The project has established the need for a modelling strategy on a common national front. The Welsh Water Authority have accepted this responsibility by appointing such a co-ordinator. The necessary detailed validation surveys requested by the investigators in August 1973 will be carried out next summer. On hindsight it is clear that a smaller division of the water industry in its current role is not able to provide all the necessary facilities for such an investigation. This is because of the breadth of expertise and resources required. What is available is best utilized by supporting a centralised research facility in the applications/verification fields of a modelling project.

The project has also regressed through variation in mamagement policy as it existed as a subsidary of a departmental function in an atmosphere of flux at a time of majoe re-organisation. This would not have arisen if managed as a research phase where resource allocation is necessarily committed in a medium and long term plan capital and expenditure programme

Notwithstanding these difficulties, the various models have been used in their function of assessing the water quality implications on projects of proposed industrial development,sewage treatment plant design and location and a major water supply scheme. It is conservatively estimated that the total capital cost of schemes so investigated is £ 90,000,000 . In an area of high unemployment major projects are politically very sensitive. Measured against such capital costs, the production of a management tool involved relatively nominal expenditure. This cost ratio justifies the instigation of the project and its support. And indeed,even measured against the costs of providing this array of modelling software through consultants or software houses, the costs incurred are fractional.

This project was carried out in an industrial environment. Tools which would enable logical evaluations of water quality planning proposals were required. Because of this , simple technology which had been found useful elsewhere could not be overlooked. Outside of the water industry research bodies, little modelling expertise had been acquired. It was therefore considered useful to initially devolve a model already established, with simple input requirements that could be easily used by management in a stand-alone environment. It also provided a short-term solution to the planning applications already with the ind strial sponsor.

The project has suceeded in its broad aim of providing river management tools in the form of a selection of models for the Usk River Divison. Recognising the wide applicability of them , the Welsh National Water Development Authority has initiated work to make these models available nationally through a computer network.

Other regional authorities have expressed interest , with a definite committment from the largest of them to adopt these approaches. By 1978 upwards of 120 million equivalent heads of population will have their water quality projects modelled.

CHAPTER 1

INTRODUCTION

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1.1. The Project Scenario

1.1.1. Previous to the Discharge of Tidal Waters Act (1960) no legal authority could control the discharge to a stretch of tidal water. The Usk rises at Carmarthen Fau in Central Wales, flows in a wide loop easterly to a confluence with the Severn Estuary/Bristol Channel some miles below Newport (Gwent). In its 80 mile length, there are no major discharges until about 25 miles upstream from Newport, at Abergavenny, when it receives treated sewage from a population catchment of some 15,000. The tidal stretch commences below Usk, some 17 miles from its confluence with the Severn. In the last 12 miles it receives some 20 major discharges, mainly from the town of Newport.

1.1.2. Of these, the majority are crude discharges of pure domestic and only occasionally industrial affluents are part constituents. Additionally a CEGB Power Station abstracts cooling water which are reintroduced upstream of the intake, so forming a loop. Many of the discharges are tidelocked, so the majority of polutants are discharged to minimal dilution levels. The peculiar shape of the Severn/Usk estuaries gives the World's second highest tidal rise at a maximum of 50 feet (15.4m). With a dry weather flow of 80-100 m.g.d.low water dilution can be as little as 5 - 6 times. In favourable summer conditions these factors can combine to produce areas of low dissolved oxygen and environmentally undesirable bank effects.

1.1.3. With the introduction of legislation, the Usk River Authority were now required to make management decisions on discharge applications to Tidal Waters, an area where little expertise was available.[21] Drinking water, industrial water and recreation water all had to be available.[18]

1.2. The Eastern Valley Sewage Board

1.2.1. On the 12th May, 1955 effluent from the works was the subject of consent issued by the Usk River Board for 1.8 m.g.d. under dry weather conditions and limits of B.O.D. 20 p.p.m. and suspended solids 30 p.p.m. in relation to a discharge of the Afon Lwyd.

The Minister determined that a temporary consent for a period not exceeding 5 years from the 4th June, 1964 for an altered discharge maximum rate of flow be fixed at 3.4 m.g.d. in dry weather and 10.2 m.g.d. in wet weather and that the storm tank discharge should not exceed 10.2 m.g.d. and that no further additional discharge of trade wastes should be accepted in the sewerage system until suitable extensions had been built.

Proposals to take the treated sewage effluent to the Usk Estuary resulted in an application being made for a 42 inch diameter outlet on the 14th April, 1965, and a consent dated 25th May, 1965 for 5.14 m.g.d. during dry weather and 15.42 m.g.d. maximum rate of discharge was issued, granting among its consent conditions B.O.D. 20 p.p.m., suspended solids 30 p.p.m. standards. On the

30th June, 1965 Eastern Valleys (Mon.) Joint Sewerage Board appealed against the conditions of this consent, although Angling Clubs supported the conditions. Their main point was that, with the increased dilution in the Usk Estuary compared with the Afon Lwyd, they wished a more relaxed standard. It was pointed out at the Inquiry that dilution factors at the new point of discharge for certain parts of the tidal cycle during dry weather were still of the order of 20 to 1 and that for some 8 hours in a tidal cycle of 13 hours fresh water was opposite the point of discharge. The Secretary of State's decision dated 20th October, 1965 granted a consent B.O.D. 60 p.p.m., comprised of a proportion of the effluent derived from the existing Ponthir Treatment Plant not taking up 20 p.p.m. of B.O.D., i.e., some part of the effluent additional to 3 m.g.d. could be mixed with a B.O.D. of 20 p.p.m. fully treated effluent and a new consent on the tidal discharge meant B.O.D. 60 p.p.m. and suspended solids 65 p.p.m. would have to be complied with. The consent took effect from the 20th October, 1965 for volumes of 3.86 m.g.d. and 11.58 m.g.d. during dry and wet weather respectively.

By April, 1966 the Eastern Valleys (Mon.) Joint Sewerage Board was concerned as the effluent from Messrs. Girling's was being considered in relation to acceptance into the trunk sewer, and in March, 1968 duplication of the trunk sewer between New Road, New Inn, Pontypool and Rose Cottage, Llanfrechfa was the subject of notice to land owners in respect of laying a trunk sewer which varied in diameter from 39", 36", 30" and partly 27" for 7,750 yards.

In 1967 an Usk Estuary Investigation scheme was compiled by myself and was taken to the Welsh Office on the 7th August, 1968, when senior engineering inspectorate, together with the Chief Chemist for the Ministry of Housing and Local Government vetted the scheme. Reference was made to a Secretary of State's decision that investigation of the estuary be carried out in no less than 5 years and not more than 10 years, in order that sufficient information on which to base accurate consent conditions could be made available.

On the 4th December, 1968 consent on treated effluent to the Usk Estuary stipulated maximum B.O.D. 92 p.p.m., suspended solids 100 p.p.m., and maximum rate of discharge 15.06 m.g.d., being 3 x dry weather flow. Settled storm sewage was granted consent at 15.06 m.g.d.

A survey of the estuary on the 10th and 11th March, 1969 indicated that the quality of the dilution water suffered very slightly, and on the 3rd September, 1969 acceptable increases in B.O.D. above and below the effluent were accompanied by permanganate values which rose from 1.8 p.p.m. above to 9.0 p.p.m. below.

It is worth noting that during the last few years the effluents from I.C.I. Fibres, Parke-Davis and Recham International Limited, Pontypool, are all included in the trunk sewer. The level of capital expenditure to achieve this was considered excessive by the Sewage Board and consequently a public enquiry was convened by the Minister of State, Welsh Office.

1.2.2. To prepare for the enquiry the River Authority conducted several long surveys to measure BOD/OD variations in the Estuary. The extrapolation of these data was questioned and a compromise of partial treatment was reached. The Minister of State, Welsh Office, did then charge the Authority to produce a sound scientific system from which predictions could be produced. The Authority were allowed time to complete this work, this period expired on 31 December 1975.

1.2.3. The Authority saw themselves being involved in many future situations along similar lines and took up the condition imposed on them by the Minister.

1.3. Statistical Models

1.3.1. It seemed as if the problem was insufficient data. The Authority embarked on a monitoring project (Ref. 1.8) with the thought that sufficient volume of data would allow regression of the most relevant parameters against the pollution index - per cent dissolved oxygen. This had been successfully attempted in the Clyde Estuary[22]. This approach soon became less favourable with the problems of mass data collection, collation and analysis.

1.4. The Theoretical Approach

1.4.1. A considerable amount of resource had been invested in monitoring, and it was with some reluctance that the Authority asked the author to produce a theoretical approach. Management were much impressed by the progress in the theoretical approach when attending the Symposium on Modelling at Water Pollution Research Laboratory, Stevenage, in April 1972.

1.4.2. Any model produced would have to satisfy the following criteria:

A) Forecast worst effects of a variation in input variables.

B) Forecast broad trends resulting from input variations.

C) Take some account of accuracy attainable.

D) Be capable of being understood and controlled by Management in absence of any permanent technical back-up.

E) Be economically viable as computer time would have to be purchased externally, at commercial rates.

F) Be flexible and as machine independent as possible.

1.5. Integration of the Project

1.5.1. The Interdisciplinary Higher Degree scheme was approached at Aston University to consider the outlines above for a project. Because of the Multidisciplinary approach required and the practical requirements the project was considered to be suitable and was accepted to commence in October 1972. The author was based at the offices of the River Authority in Caerleon with use of the University College Cardiff Computing facility. Aston University and Water Pollution Research Laboratories provided an academic and theoretical basis. The Science Research Council sponsored the project for 3 years, and for the last quarter, the River Authority were sponsors with the Department of Civil Engineering, Aston University.

1.5.2. The resultant thesis was to fulfil the following requirements:

A. Satisfy an examining body as to the work carried out to award the author a higher degree under the IHD regulations.

B. To act as a reference for the scientific foundations of the models used.

C. To act as a reference manual for management to use the models without expert assistance.

D. To catalogue the software that had accumulated during the course of the project.

1.5.3. Hence this is not a thesis in the conventional sense, it must meet a wide variety of criteria and cover a broad field of interest.

1.6. Attitudes to Environmental Pollution

1.6.1. Public attitudes are recognised as the most fickle of parameters as far as trends and predictibility are concerned. It is therefore more surprising that for the past decade, attitudes towards pollution have been substantially constant. Only in the extreme financial crisis of the past year were opinions modified to temporary relaxation of standards.

1.6.2. The author conducted private correspondence with many prominent people in the field of pollution control. Opinion is remarkably consistent amongst representatives of the public[(1)-(5)], the only differences being in the rate and priority of improving the environment. One prominent councillor[(3)] interestingly suggests that the breakdown of organised religion led to a consequent lack of faith in the future and an increasing self awareness that personal intervention was more likely to succeed than of the divine variety. All agreed that the apathy of the public is only aroused by personal experience of a pollution and too little of the technical aspects are understood. The representatives all recognised their own lack of knowledge and dependence on technical advice and guidance. This is the kind of advice the models would hopefully supply.

One prominent civil servant expects a net increase of pollution[(6)]. It is reasonable therefore to wish to minimize the effect of this by optimizing current treatment facilities. Again, the models would be of great assistance in this respect.

1.6.3. Some correspondants recognised that in the short term, pollution control may have to be relaxed to allow economic reflation. Only a few suggest a compromise cost benefit analysis of pollution vs employment[(7),(10)]. Lack of effective controls were mentioned[(7),(11)], as well as the difficulty of defining exactly what pollution constitutes.[(4),(7),(11)].

1.7. Attitudes to Modelling

1.7.1. Correspondence on the matter of modelling also flowed during the project. Here the expected dichotomy between the expert and the layperson dominated views. Some public representatives had little concept of what a model is[(1), (4),(5),] and some limited experience [(2),(7)]. They were all aware, however, of the potential use and misuses so lack of in-depth knowledge is less relevant.

1.7.2. Experts in the field all had experience of modelling of varying forms. The limitations seem to be well established[(6),(9),(10),(16),] and so borne in mind when predictions are employed in the real world. No reference was made to the relative cost of modelling/cost of works to which models are applied. In this project, the ratio is about 1:15ØØ. So even limited modelling success quickly becomes economically beneficial.

1.7.3. There was a notable lack of differentiation between physical models and theoretical mathematical model. Physical Models may be expected to be morally more acceptable to laypersons. The lack of discrimination is to be welcome as a sign of increasing acceptance of the attempts to understand the modelled processes rather than just duplicate them.

1.8. The Usk Estuary

1.8.1. The Usk Estuary has the world's second highest tidal rise[17], Spring tides recently reaching 51 feet. This is largely due to its geographical position as an offshoot of the Severn Estuary (see figure 1.8.1.) The lower 17 miles of the Usk are tidal, nearly to the town of Usk. It has a narrow but low flood plain and above Newport meanders violently to make erosion and flooding a frequent event in previous years[10].

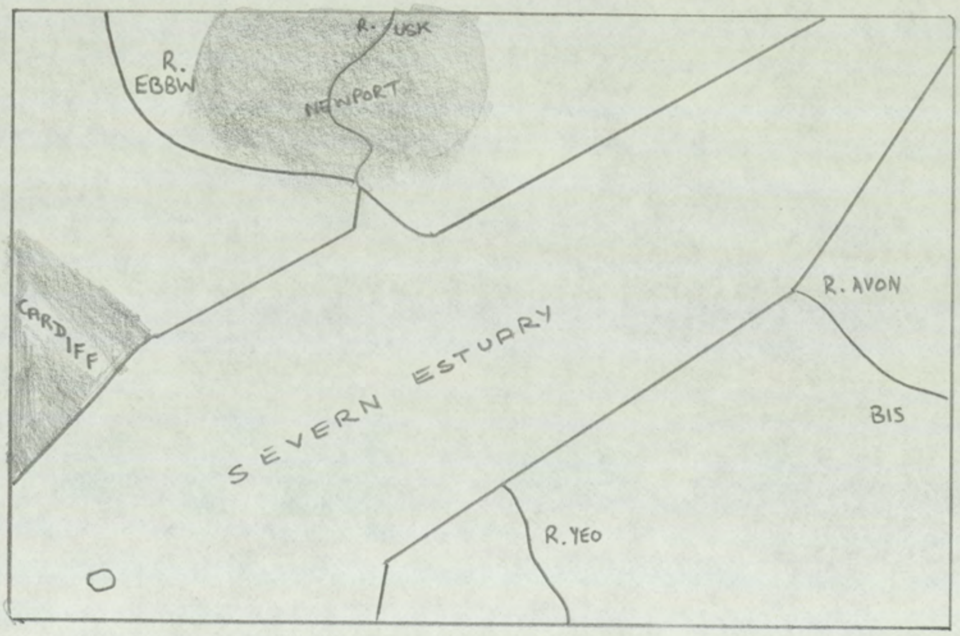


Figure 1.8.1. Schematic Diagram of the position of the Usk Estuary.

1.8.2. At low water it is no more than a stream above the Newport Road Bridge sill. There are numerous small tributaries, the main ones being:

Sor Brook	-	only 1 m.g.d. DWF just above Afon Llwyd.
Afon Llwyd	-	just above Eastern Valley outfall, a useful source of dilution for that outfall.
Ebbw	-	about 30 m.g.d. DWF, below Pierhead so only a small overall factor.

1.8.3. The Ebbw once held the undistinguished title of most polluted river in Europe. As its primary volume source is the British Steel Corporation works at Ebbw Vale, this is perhaps not unexpected. Part of this pollution found its way into the main estuary system and compounded the situation. In recent years the evident improvement in this river has been a major achievement for the Pollution Control Department at the Usk River Division. The Ebbw can now be considered primarily as a dilution source.

1.8.4. The estuary receives some 20 major discharges, of which the Eastern Valley outfall is the major source of volume and leading - about 5 m.g.d. at 90 p.p.m. BOD. (ref 1.2.1). The town of Caerleon has a partially treated discharge from its own sewage works some 10 miles from Pierhead. The remainder are crude discharges from the town of Newport. These are usually tidelocked for some part of the cycle so their effective position is displaced. Newport Borough Council Main Drainage Scheme will eventually treat most of the discharges and the estuary will receive an increasingly major discharge about 2 miles upstream of Pierhead. Corresponding loading further upstream will also be removed.

The stretch of the Usk between the Pierhead and the end of the dredged channel is of interest. At low water it is a narrow channel, maintained and dredged by the British Docks Board, until it meets the water body of the Severn Estuary. As water level rises, there is a dramatic change in shape as the river widens to over a mile, narrowing to a mere 100 yards at the Pierhead. Much of this water is slow moving and 'liquid mud' with very high suspended solids. Parameter range was wide enough to allocate two channels for suspended solids on the British Aluminium Corporation Site Monitor Station.

1.9. Monitoring The Estuary

1.9.1. In 1970 the Usk River Authority set out on an integrated monitoring programme for the tidal water. There were to be 5 complete monitoring stations and a similar number of depth probes. Management then hoped to establish a pure statistical regression model.

The data loggers were of an experimental variety, the physical conditions were extreme and many problems were encountered. Specifications in the design gave an accuracy of 1 scan in a million. No clock track was therefore included. Final analysis of the first few data tapes showed an accuracy of less than one per hundred scans.

1.9.2. Physical problems encountered were:

A. Coordinating a stabilised power supply. Especially the British Aluminium Company logger showed marked deviation from linearity with a fluctuating mains supply.

B. Physical pumping of water up to 500 feet from the submerged pump to the probes.

C. The reliability of a pump exposed to up to 50 foot pressure heads and a river with frequent large items of flotsom.

D. Technical support and back up from the Company[20].

E. Vandalism at stations.

1.9.3. The resultant data was in two forms, a chart with coloured plots taken by a multiple recording head and a plotting frequency of about 72 seconds, and a locked cassette magnetic tape which the logger firm translated (see system component chart[21]). The problems with data encountered were:

A. Absence of clock track on analogue/digital interface leading to temporal displacement of data.

B. Corruption of Data through

i. Absence of scan marker,

ii. Extraneous channel or one missing channel

iii. 'Dropping' a bit.

C. Frequent redesign of sampling frequency and channel interpretation by the technical staff.

D. Data format was in continuous binary, no parity track on paper tape, the tape being of poor quality and often fan folded. This was unsuitable for the high speed reader (1.5k c.p.s. against 10 c.p.s.)

1.9.4. Clearly the task was formidable. The software to edit the above as much as possible was written and tested in the first year of the project. When hardware errors also caused long delays it was recommended that the main modelling should receive its due priority for the remainder of the project.

This phase of the whole project cost some £30,000 of which nil has been recovered by providing useful data. However, software to the value of £12,000 (supplying company quote, 1970 prices) is available and applicable to most similar monitoring schemes. In view of the investment in this phase, it is suggested that the computing facilities at the Water Data Unit, Reading are employed for cleaning the tapes. This makes editing feasible because of excellent paper tape facilities and low costs for users mill-time. To be noted is the financial loss through over stretching available expertise. The supplier must bear the major moral burden here. Details of the software appear in the attached volume of programs.

1.10. Software Support For Programs in This Thesis

The Project hinges on the numerous programs developed for the modelling. The modelling software will be used by the Welsh National Water Development Authority, through the Water Data Unit at Reading, to provide a national service for the individual river divisions to interrogate.

All software will also be supported in its current form by M.R.S.S., currently of 11 Highfield Road, Caerleon, Gwent, NP6 1DU (Registered under the Business Names Act, 1916) until at least 31.12.1979.

Note: Private Correspondence represents the views of the individuals only, and not those of employers or public bodies.

(1) Private Correspondence, Councillor Rosemary Butler, Newport Borough District Council.

(2) Private Correspondence, The Mayor, Newport Borough District Council.

(3) Private Correspondence, Councillor Paul Flynn, Newport Borough District Council and Gwent County Councillor.

(4) Private Correspondence, Councillor James T.Kirkwood, Gwent County Council.

(5) Private Correspondence, Rt. Hon. John Stradling Thomas, M.P.

(6) Private Correspondence, Head of Economics and Statistics, Dept. of Energy.

(7) Private Correspondence, Chief Environmental Health Officer, Torfaen.

(8) Private Correspondence, Director of Civil Trust for Wales, Cardiff.

(9) Private Correspondence, Director of Planning and Scientific Services, Northumbrian Water Authority.

(10) Private Correspondence, Director of Resource Planning, Severn Trent Water Authority.

(11) Private Correspondence, Research Fellow, Dept. of Botany, U.C.Cardiff.

(12) Private Correspondence: Divisional Water Quality/ Fisheries Officer, Glamorgan River Division.

(13) Private Correspondence, Water Quality Planner, Thames Water Authority.

(14) Private Correspondence, Director of Resource Planning, Anglian Water Authority.

(15) Private Correspondence, Divisional Manager, Usk River Division.

(16) Private Correspondence, Divisional Scientist, N.W. Water Authority.

(17) Newport Tide Tables and Shipping Directory, Annually published by R.M.Johns Ltd, Daniel Str, Newport.

(18) "We live in Thirsty Times", published by W.N.W.D.A.

(19) The Usk River Authority Annual Reports, 1965-1974 (1st to 9th). Land Drainage, Section "Flooding".

(20) M.W.Rogers, Progress Report on the I.H.D. Project to SRC, Project B72/13003, Oct 1973.

(21) M.W.Rogers, Initial Report on the I.H.D. Project, SRC Ref B72/13003, Dec 1972.

(22) D.W.MacKay, "A Model for levels of dissolved oxygen in the Clyde Estuary", Proceedings, WPRL Technical Paper No.13 on "Mathematical and Hydraulic Modelling of Estuarine Pollution", 18th April 1972, p.85-p.94.

CHAPTER 2

Water Management and the General Philosophy and Review of Models

Chapter 2

CONTENTS

2.1. Mans Use of Water

2.1.1. Water is the single most important commodity to life as we know it, as a sustainer of life, as a means of transport, as a raw chemical, as a production aid and as a means of pleasure. The supply of water is the most generally accepted necessity to exist in any degree of comfort. Past abundance has fashioned public attitudes and perceptual variables were stronger in impact and opinion moulding than any demographic data [1,2]. Unfortunately by the time water quality changes are perceptible by the public they are usually gross variations and may have caused irreversible ecological damage.

2.1.2. The main uses of water are:

A Domestic e.g. supply to the home,

B Industrial e.g. use as a chemical,

C Cooling Waters e.g. Power Station cooling. Often classed as industrial use,

D Recreational e.g. Non contact use e.g. boating and contact use e.g. swimming,

E Commercial Fisheries,

F Irrigation for Agriculture.

Each geographic area places varying emphasis on the subjects above. In the South Wales Area all except F) are important, with A) and C) predominating due to population concentration and cooling waters in Steel and Power complexes. There is no irrigation in the area other than some deliberate flooding of fields to allow silting.

2.1.3. Domestic Supply and Use

The main criteria applied here is twofold: the supply must be potable and palatable at the point of consumption. This criteria provided the inertia for the initial models of stream pollution by Streeter and Phelps[5]; the maximising of the natural stream purification and thus minimizing artificial treatment supplied. Water use is on the increase. The AM population of Wales require four million gallons per day for all uses, 40 galls per capita per day domestic consumption.

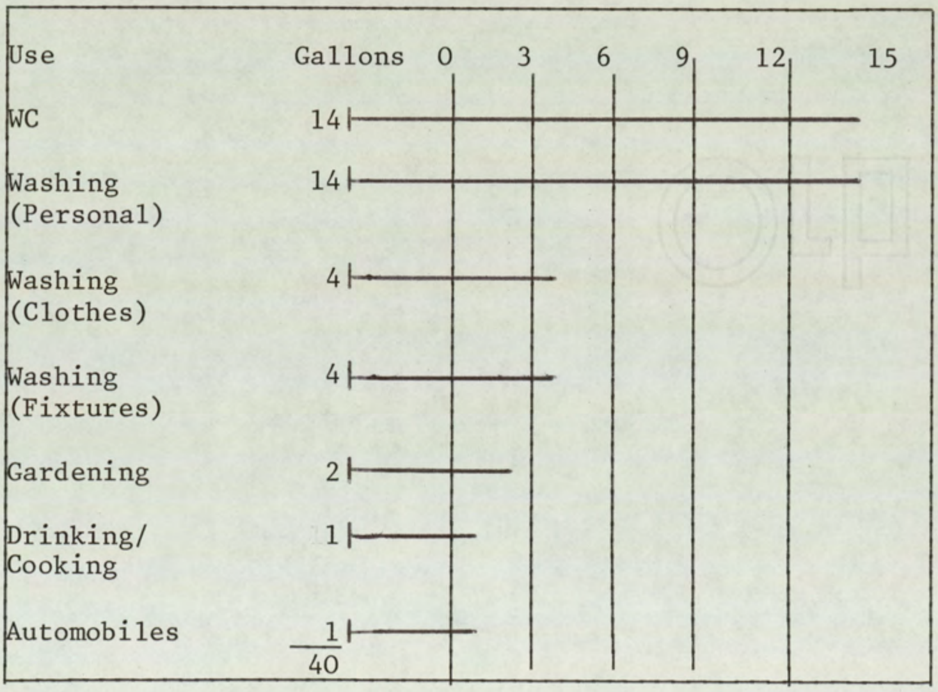


Fig.2.1.3.A - Breakdown of Water Use by Domestic Consumers in Wales[6]

The breakdown shows that an increased living standard will increase water demand. So in a situation where population expands and living standards rise the demand situation becomes more acute. This is the situation in the long term for Gwent. The low level of metered supply (virtually nil domestic users) and the system of general rate levy does little to encourage any degree of conservation. Only the absence of supply is of interest to the domestic user. The problem is not localised. In the USA, consumption rose by 50% in the period 1965, finally at a rate of 125 gallons per capita per day [7].

The desired tolerances for this water use are strict and digression outside limits can have wide reaching consequences. Some of the limits for raw domestic water are: Coliform bacteria 5,000 per 100 ml, Streptococci/Faecal Coliforms 100 per 100 ml, Cyanides 0.1 p.p.m., Pesticides <0.1 ppm, Temperature <95°F (36°C), Phenols 0.02, Oil-Turbidity-Colour not to be visible etc. [3,4]. Of these, possibly the Faecal Coliform count is the single most important criteria.

2.1.4. Industrial Supply and Use: Cooling Water

Due to the variety of processes employed, generalisations of use in this area are very different. Some figures generally accepted for Steel manufacture are:

To produce 1 km of Steel [9] requires

Blast Furnace Smelting	25,000
Milling etc	21,000
Rolling and Drawing gallons of water	3,500

Fig.2.1.4.A.

Other heavy consumers are the pulp/paper industry, one ton of paper requires 50,000-60,000 gallons, a manufactured vehicle requires about 20,000 gallons per ton of vehicle. For each water consuming industrial user, a certain quality must be attained. If a supply is networked to several users, the most stringent conditions have to be applied. This is uneconomic and a burden on all users. Generally a moderate standard is networked and then each user can upgrade to their required standards. Cooling waters are generally not required to be of a high standard unless high temperatures are attained. The table below gives some measure of the variations allowed (8)

Type of Consuming Process	Dissolved Solids	Chlorides	pH	Iron	Manganese
Fresh Cooling Water	1000	600	5-8.3	NE	NE
brackish Cooling Water	35000	1900	6-8.3	NE	NE
Textile Manufacture	1000	-	4-9.0	1.3	.01-.05
Organic Chemical Manuf.	-	NE	6.5-8.7	.1	.1
Petroleum Refinery	1000	300	6-9	1.0	NE
Vegetable/Fruit growing	500	250	6-8	0.2	0.2

Notes: A) NE - Any value acceptable within normally received waters
B) All concentrations in p.p.m.
C) Depending on process used

Fig.2.1.4.B.

The low quality requirement of cooling waters and the amount of steel production in the Gwent area is reflected by the level of non potable water supplied by the water undertaking.

2.1.5. Recreational Use of Water

Recreational use may be split into two groups; water contact and non-contact activities. Separate quality may be permitted.

Contact Recreation	Non Contact Recreation
Swimming	Boating
Diving	Fishing
Water Skiing	Waterside recreation

2.1.5.1. Contact Recreation demands more stringent standards than raw domestic supplies with a faecal coliform count of less than 200 per 100mg/l, with a maximum of 400 in the area of a discharge. These limits are applied to 5 samples over a period of not more than 30 days. A pH in the range 6.5-8.3 and a temperature under 85^{o}F is also required. A Secchi disc should be visible at least at 4 feet depth.

2.1.5.2. For Non Contact Recreation the public health standards can be relaxed. However, fishing and other aquatic life require separate standards which may in part be more stringent. Minima generally accepted for fisheries are

D.O.:	5mg/l for 70% of time, yet never <3.0mg/l	Resident Fisheries
	4.5mg/l for up to 25% of day, never <4.0mg/l	Migratory Fisheries

Artificial heat should not raise water temperatures by more than 1°C in summer, 2°C in winter months. If molluscs are reared, coliform levels should be below 70-100 per 100 ml. Water pH should not vary outside 6.7-8.5.

2.1.5.3. Water Side recreation require little explicitly in the way of standards as they are only affected by grossly polluted situations like eutrophication or anaerobic kinetic producing unsightly vegetation or pungent odours. As these are rarely pursued without a mix of other activities mentioned, it is likely that any desirable site will have sufficient water quality to meet these loose requirements.

2.1.6. Agricultural Use of Water

The proportion of water used for this purpose naturally varies widely. In the area of this project there is little use of water for irrigation, and the main abstractions are for drinking and cleaning water. Yet, within 200 miles these factors are a major source of use and with a high growth rate and only $^{1}/_{3}$ of the Usk's precipitation, will require major expenditure[11]. Other managements have a major obligation to provide irrigation waters, the U.S.A. water undertakers provided 135,000,000 million gallons for irrigation in 1965[10], a major factor in the self sufficiency of America.

2.2. Management of Water Resource

2.2.1. The Gwent Water Division (formerly Gwent Water Board) is responsible for supply, while a section of the Engineering Section of the Usk River Division is responsible for resource management. Both liaise closely with local authorities and industry and the pollution control function of the Usk River Division.

2.2.2. The Gwent Water Board

The Gwent Water Board is the major Water Board in the Usk/Newport Area. On March 31st, 1973 it serviced 433,000 people over 576 sq. miles[12], placing it 8th and 16th in order of population and area amongst all water undertakers in the United Kingdom. Out of a daily supply of 47,000,000 gallons[13], 50% is unpotable, so its 1.27p income for every 1,000 gallons supplied is not ungenerous when compared to 7p for East Shropshire Water Board. Against 0.01p for the Fylde Water Board it appears sufficient. 160,000 properties were connected to the mains supply of which 6,000 were metered. This is a very low ratio compared to most other areas where a ratio of 1:10 to 1:20 is more usual. Now reorganised into the Gwent Water Division, it still carries a responsibility of supply maintenance. Currently the supply is 50,000,000 gallons a day, of which 30% are supplied to Spencer Steelworks at Llanwern. The recent unusual climatic conditions have caused a crisis situation in water supply. The main reservoir (Talybont) of 50 days supply has a level at 20.02.76 of only 28 days. This highlights the possible need to abstract further from the Usk whilst maintaining quality for the estuary section. The media are in no doubt as to the gravity of the situation(14) and the responsibility of the Board and Consumer.

2.2.3. Water Resources

The Usk has a protected flow level of 100 m.g.d., which must be maintained. The Steel industry are major users and due to the nature of pollutants from Newport, enough solvent must be present to ensure adequate dilution factors at low water. In practice, flow levels of 55-60 m.g.d. have been attained during summer 1975. Some preventive schemes are available, the three principles being:

a)	Wye-Usk Transfer Scheme.	Capable of maintaining guaranteed flows.
b)	Graig Goch Reservoir Scheme	A major project to guarantee storage capacity.
c)	Circulation Scheme.	Reservoir resource is pumped to head of section, released & recaptured further downstream.

South Wales is a highly populated area with a high water demand. High rainfall in mountainous regions provide sufficient resource, which must nevertheless be efficiently managed as a large proportion is lost to areas outside the region.

2.2.4. Management Objectives

Water industries are usually financed through public funds and thus extensions of government in some form. This leads to frequent overriding political motivation when alternative proposals are analysed. It is a simpler task to study various economic alternatives than to attempt to predict political/social factors. Incorporating shadow cost benefits into any model provides considerable area for dispute.

The simply water management objective is:

"To provide sufficient water resources of sufficient quality at minimum cost and environmental impact".

It is only recently that indirect costs/benefits are being considered in long term impact analysis. The feasibility of such considerations depend on equally long term forecasts of social developments and economic trends in the region under study.

2.2.5. Benefit Models

There are three main benefit models, where the Annual Net Benefit (i.e. Social and Capital) B is related to the time from the design year of the project Y:

A: Constant Benefit $B(Y) = K_A$

B: Linearly Increasing Benefit $B(Y) = K_B Y$

C: Compound Increasing Benefit $B(Y) = K_C \Theta^Y$

D: Linear Combinations of above $B(Y) = \alpha K_A + \beta K_B Y + \gamma K_C \Theta^Y$

A represents a situation where a project used at constant capacity for its design life, as near to the optimum design level as possible.

B represents a situation where a resource is not fully used at once but steadily increasing while on initial resource decrease to expiration at a similar rate.

C Represents a demand resource situation where controlled supply increases resource potential due to rising demand.

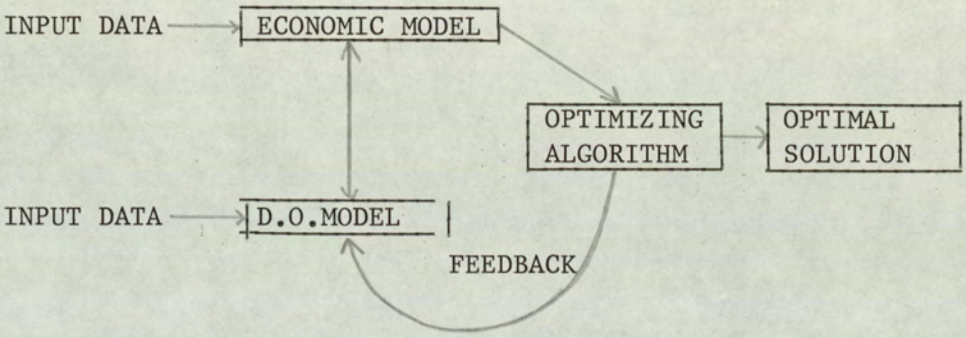
In a long Design Life (50 years or more), the eventual model is a case of a D type model, a linear combination of the previous models to take account of variants in social/economic patterns. If any K are negative, either the benefit is decreasing, for example in a resource exhaustion situation, or benefit is negative. A highly undesirable scheme from water quality aspects but more politically desirable would yield a permanently negative benefit model. Parker[15] and Davis/Hanke[16] cite some models and examples of the dilemma encountered. Parker concludes that

A) Benefit is primarily determined by regional growth.

B) Benefits have found to decrease rapidly
with advances in technology,

so the main factors are outside the sphere of influence of the design.management.

Successful integrated models have been applied successfully[20] using simple sub models but effectively integrated:



and the work showed the problem is likely to be in the implementing of the least cost solution.

2.2.6. The Ruhr

The Ruhr management system has been extremely successful in coping with limited supply, heavy industry and extensive recycling[22] since 1913. It operates two strong bodies, the resource: Ruhrtalsperrenverein and the user: Ruhrverband, controlled by constant legal statutes laid down by state government. Instream aeration, transfer schemes backup reservoir and automatic monitoring stations together provide an effective management system. The Thames estuary has also been a prime example of a good concerted Management programme in pollution control[23].

2.2.7. Water Quality Indices

Growth in scientific method and wider acceptance of its output by other disciplines has led to a greater degree of standardisation and reproducibility within. This has signalled the demise of adjectives in data as they are considered to be subjective and objectivity is the goal.

They are invaluable though in transmitting managements view of water quality to the general (and usually paying through taxed) population. A substitute has clearly to be found for this role. There are hundreds of potential pollutants to be found in a water body, many physical/ biological characteristics to be considered when classifying. The populace are not interested in, say, the level of 2-4-6 Tri-chloro-benzoic acid, but want to know whether they can swim or not. Yet to the tomato grower 'down the way', the level of TBA is most critical for his crop[19].

Attempts have been made to coagulate all the measured parameters into single index, for example[17]

$$WQI = \{\Pi\{f_i^{w_i}(P_i)\}^{1/S}$$

where $S = \sum_{i=1}^{N} w_i$, w_i = weight attached to ith variable, P_i = value of i^{th} variable, f_i is sensitivity function of i variable. The Water-Quality-Index (WQI) so devised is the geometric mean of the N sensitivity functions. This is more sensitive to extreme values and being numeric removes the need to itemise physical/chemical tests individually. No account can be taken of the use of the body of water to which a single index has been specified.

As the primary purpose is to pass information on to allow public decision making as to use, the water quality of each use must be identified individually.

A similar but more explicit index would seem more useful[(18)]. Levels of acceptability are graded in a base n into n categories 0 to n-1. The tests are then strung together to form a digit string which can be condensed by grouping digits.

Consider a water body with the following characteristics:

Parameter	Level	Category
B.O.D.	Unacceptably High	0
D.O.	Unacceptably Low	0
Coliform count	Acceptable	1
Odour	None	1
Suspended Solids	Average to moderate	1

A single index would show that its a fairly poor state, but would it be possible to say that it is good enough for contact recreation from a single figure? If written

BIN'11100' or OCT '34' or HEX'1C'

knowing the position of the coliform count (the most important single deciding factor for contact recreation) in the string, it is possible to identify whether the category is acceptable or not. The tomato grower would only be interested in the TBA digit when he considers irrigation waters. This sort of index minimizes information loss and allows the public to make decisions rather than the fait accompli of a single index which predetermines the YES/NO decision of the public. The disadvantage is that much public education would be necessary to enable a reasoned thought process to digest the implications of the number string.

The complex WQI retains the property of the simpler WQI that for two water body quality indices n_1, n_2, if $n_1 > n_2$ the quality of body 1 is 'better' than that of 2, thus allowing simpler comparison procedures as only classes of acceptability and order of priority in a string has to be considered.

The public must be given maximum information to allow self-determination. The indices should be widely publicised and would act as a public yardstick to monitor the effectiveness of the pollution control body of the region. The public do want to know[21].

2.3. Models as a Management Tool

2.3.0. Introduction

Management have two functions, planning and control. In the water quality context both roles can be supplemented by models. For example, a housing estate development generating 50 gal/day water usage can be planned for as its likely effect can be estimated, then planned in advance of the realisation of the physical reality. Should a milk tanker crash and discharge to a river, the area most critical downstream can be located and controlling action taken in advance of it. On a system with retention times in the order of a week this is feasible even in the absence of an on-line computer facility.

Scant regard is usually paid to the overall philosophy of the modelling process, resulting often in management using tools they have no background to, thus increasing the dangers of unsuitable extrapolations of usage. This can be prevented by the formulator retaining the control of the production phase of the model. This is unsuitable in the many instances where the building phase may be through temporary retention of expert staff. In any case, it is desirable to have management aware of the powers of their tools and thus lessen the black-box concept that emerges through rapid advances in theory or technology beyond the grasp of up-line management. Publications on models are now recognising this[28].

2.3.1. Advantages and Disadvantages of Models

As with all tools, there are gains and losses. Some of the advantages are

A Time scales of feedback quicker once established.

B Situations can be simulated and effected without disturbing the system.

C Alternatives can be simulated without additional field effort.

D Alternative solutions can be optimized from water quality criteria.

E Most viable economic alternative can be found

F Cost of simulation on a computer should be cheaper than field observation.

G Cost of simulation will be several orders of magnitude less than the possible economic savings through its use, so it need only be useful in some applications.

H Useful for substantiating broad opinion.

I Pollutants need not be introduced into the system for measurements (e.g. colour, radioactive or virus tracer techniques).

J Should provide some measure of errors/fluctuations in system which can easily be missed in field studies.

K Easily updated on advances in process knowledge.

L Can be written interactively for direct use by management.

Some of the disadvantages are:

A Initial capital/manpower investment before anything concrete is achieved.

B Initial time of response to management query, until established.

C Difficulty of 'translating' output for or by management.

D Extrapolation beyond bounds of statistical significance.

E Possibility of unconsidered processes emerging as dominant factors.

F Skewed data input would distort output.

G Interdependent on continuous computing expertise availability and a high percentage of up-time by the central processor unit.

Overall, the gain in a good model is high. The models in this project have been used on the following projects.

A	Impact of Graig Coch Reservoir Scheme	£80 M
B	Variations in consent for Eastern Valleys Discharge	£2 M
C	Estuary Fisheries	£1 M [31]
D	Newport Main Drainage Scheme	£12 M
E	Various industrial and other schemes	>£5 M
		PROJECTED CAPITAL COSTS

2.3.2. What is a Model?[27]

The Oxford Dictionary defines the word 'model' as "something which resembles something else". The implied resemblance being physical as these are the 'models' a layman associates with the word. The water industry uses both types of model - physical and conceptual. Any process can be modelled mathematically, the success of which determines the acceptance of a model [29].

2.3.3. Physical Models

These are well established in the water industry for certain applications and will continue to be of practical significance for some time. They are usual strict scaled versions of physical reality, with lateral scales more reduced than vertical scales. They are usually of estuaries, reservoirs, bays etc. and of great importance in studies of tidal action, sedimentation siting of works etc. They are also useful for calibrating mathematical models for any hydraulic predictions. Current models exist for the Thames, Humber and other estuaries.

Physical models are infrequently used to study water quality problems as the distortion of scale confuses diffusive mechanisms and the problems of monitoring concentrations are equal to that of field data collection. Considerable progress has been made in the adaption of hydraulic models to simulate pollution phenomena by adjusting the physical parameters of the model [30]. Tracer studies can be carried out economically in the laboratory.

2.3.4. Mathematical Models

2.3.4.1. Mathematical Models are conceptual models. A physical process is considered, dismembered, translated analytically and reassembled as a system of mathematical manipulations. The various models resulting for any given situation arise through differing methods of achieving the above reconstructions. A model does not exist in isolation, but is a part of a larger systems plan towards a desired objective.

2.3.4.2. Formulating a model is a stepwise procedure with many options at each stage.

A. Definition of Objectives. A management phase. They must have a clear idea of the questions they wish investigated, some concept of time scale of the problem and of the time delay in receiving useful feedback.

B. Selection of Scale. The objectives are analysed to consider what level of modelling is required, and the extent of the sub system to be included in the model

C. Selection of theoretical environment, i.e. which processes are to be modelled, to what depth of accuracy, alternatives available etc. An ideal model would include all processes, with a perfect understanding of their individual behaviour. So practical models are not ideal models.

D. Planning of verification/calibration and of translating step C to an algorithmic form.

2.3.4.3. Modelling success or failure hinges on reducing the following error sources to a consistent minimum.

A. Inherent data errors, depending on quality of analytical chemistry available.

B. Process equations are approximations to the real response, especially at extremes of response or time scales.

C. Arithmetic errors through hand and especially digital computers. Well designed algorithms reduce this source of error to a 6th significant place phenomena.

D. Exclusion of factors under C of 2.3.4.2. that do have a profound effect on the real system. This is usually a major source of error, but it is not unknown for one parameter in a model to be the 'x-factor', i.e. adjusted to allow for all other phenomena.

Random errors are not as serious as others, as volume of data can compensate for lack of definition in analytical chemistry. Consistent errors may cause reaction mechanisms to be switched in which should not be invoked. Lack of consistency in calibration of the model usually indicates an error under section D, in which case the theory has to be revised.

2.3.5. A Loosely Mathematical Theory of Models

2.3.5.1. Consider a subset E_s of the real world E, so that E_s is an isolated system. The subsystem E_s comprises n independent parameters, m dependent parameters and p processes.

The set of dependent variables is V_d, and has m items

The set of independent variables is V_i, and has n items

For each item in V_d, we can define a non-trivial subset of V_i called V_{id}.

The set V_{id} forms a sub-environment within E_s for the variable considered in V_d.

A set of processes is now defined, P_{id}, which defines the process kinetics for each respective element in V_{id}.

The sum of operations on elements of V_{id} by processes P_{id} form the model of dependent variables V_d.

2.3.5.2. Consider Newton's Law of Gravity $F = g. m_1 . m_2/r^2$

An unlikely choice of V_d could be {F, velocity of light C, density of the ether ρ}, V_i is chosen as {m_1,m_2,g,r, refractive index of ether μ_E}. To model the Newtonian Law, select F from Vd. This is modelled on a subset of V_i, in this case {m_1,m_2,g,r}. The 'process' set is P_{id} = {*, *, *,/( )^2}. Combined, these give the model:

$$F = m_1 * m_2 * g/(r)^2 = P_{id} \circ \{V_d(F) \circ V_i\}$$

2.3.5.3. In a practical model, the set V_d contains those variables to be investigated, and V_i should contain all those variables considered to be necessary to influence the set of variables in V_d. The processes P_{id} are more complicated, as these may involve various interactions and dependencies. This is where the scope is considerable.

There are 3 major groups of mathematical models:

2.3.5.4. DETERMINISTIC Models

In a model M_D, if $\{V_d\} = \{M_D \; V_C\}$

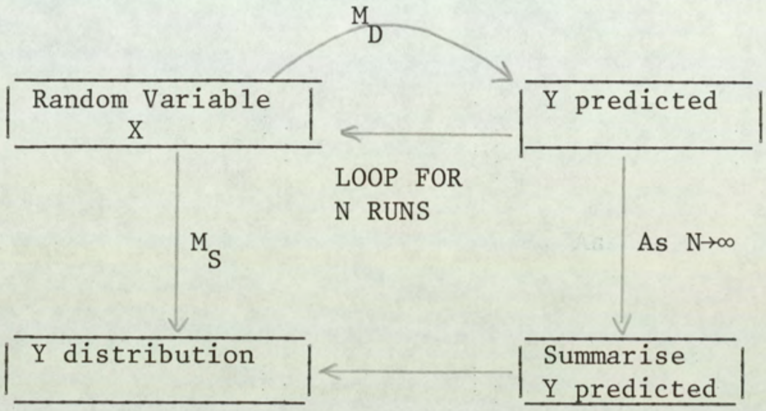
where all variables in V_i are non-random in nature, and the model M_d introduces no random components, the model is said to be DETERMINISTIC. One set of input values will produce one set of output values, reproducibly. Can also be used when random effects are known to be present but considered to be negligible.

2.3.5.5. STOCHASTIC Models

In a model M_S, if $\{V_d\} = \{M_S \;\; V_i\}$

where either some variables in V_i or some part of the modelling process M_s is not defined exactly but by an expectation value and some distribution, the model is said to be STOCHASTIC. One set of input values will produce a set of output values, but not reproducibly.

Consider a variable x, random with normal distribution, standard deviation σ and a set of random numbers distributed normally $\{R\}$. Suppose a prediction of y is made for every case of $x + R_i\sigma$. Every individual case is a deterministic model, but they can be grouped into M_S



RELATIONSHIP BETWEEN M_D and M_S

2.3.5.6. REGRESSION MODELS

These seek to relate observed data in $\{V_i\}$ to $\{V_d\}$ parameters by fitting a surface through the observed data by minimizing the sum of distances between the surface to the points of $\{V_i\}$. This is the LEAST SQUARES Algorithm and is rarely used as a model form in isolation.

2.3.6. SUB MODELS

2.3.6.1. Many tasks considered for modelling would be formidable in terms of an overall approach. It may be convenient to break a problem down into a set of sub models. All submodels of a system must be of the stochastic/regressive or deterministic/regressive type. Sub models can be interactive with other submodels. Consider a model M to be a combination of two models M_A and M_B, then

If $M = M_A + M_B$ the model M is said to be LINEAR in M_A, M_B

If $M = M_A * M_B$ the model M is said to be MULTIPLICATIVE in M_A,M_B

If $M = M_A(M_B)$ the models M_A and M_B are interdependent to form M

2.3.6.2. Consider the problem of a minimum cost/maximum water quality benefit sewage treatment plant site. The treatment plant has to be modelled to predict the quality of effluent through primary, secondary and tertiary treatment, the cost of each phase has to be modelled, and the cost of discharge to various points on available water courses. If E is the input effluent and D is the output effluent, then

$$D = M_T\{M_S(M_P\ E\ )\}$$

where M_P, M_S, M_T are models of primary/secondary and tertiary treatment efficiencies. The cost model $C = M_{CT}(M_{CS}(M_{CP}\{E\}))$ is similar, but requires addition of M_{DT}, or model of discharge transportation costs. The overall model M required is

$$M = \max D + \min C = MAX(M_T(M_S(M_P\{E\}))) + MIN(M_{CT}(M_{CS}(M_{CP}\{E\})) + M_{DT})$$

i.e. the result is a mixed additive/interdependent model. Sensitivity is not usually a linear function. It can be thought of as the gradient component of an n dimensional surface for the variable under consideration. It is more valid to use the term 'conditional sensitivity' and define it as the relative change in a variable for a unit change in input variable V_k for specific values of $\forall V_i \in \{\text{set of input variables}\} \neq V_k$. The sensitivity of a variable with respect to an input variable is conditional on other variables in a system. Often though only a limited number have significant effects.

2.3.7. SENSITIVITY of A MODEL

The sensitivity of a model is defined as the relative change in a predicted value for a unit change in an input variable. For a variable x input to a model predicting y via a model M, the sensitivity of M for y through x is

$$S(y\ M(x)) = \frac{(Y(M(x+1)) - Y(M(x)))}{Y(M(x))}$$

Often S is a well behaved function, but often a small change in x can trigger a different sub model (as in the Steady State model described later when concentration of D.O. falls below 0.4 mg/l) producing a discontinuity in S.

A highly sensitive model could be detrimental in that magnitudes of cause/effect become distorted. A model so insensitive that variations in input produce little variation in prediction is of no use either. It should be remembered that the system itself may be highly sensitive or stationary for some conditions, so similar variations in the model are to be expected.

2.3.8. QUALITY OF A MODEL

It is difficult to quantify as different levels of acquaintance with the model will have different viewpoints. Managements only quality criteria is whether it meets their demands. The formulator knows the implicit and explicit assumptions made, the processes included and excluded etc. One measure used is the amount of variation in data that can be modelled. Ideally, after data observed is subtracted from that predicted, an uncorrelated series of random variants remains. This is usually not so, the amount of correlation not present indicating the 'quality' of the predicted series.

$\{R_i\} = \{O_i\} - \{P\}$, i.e. residuals = observed - predicted

% quality of model = (1.∅ - max autocorrelation coefficient of series R) * 1∅∅

2.3.9. Extension of a Prediction System

2.3.9.1. The main function of models being the application of the predicting process to theoretical situations, some consideration of the validity of extrapolation is required. Models are tested for one set of conditions, a validation population. Usually then the model is used to predict a population at a different point in the set of space-time.

Algorithms for distinguishing groups and testing the significance of any differences have been established for many years [(24),(25)], based on work by Hotelling, Fisher and Mahalanobis. The essence of the statistics is the

Linear Discriminant Function (LDF), being the sum of products of each variable with a weighting factor that seeks to optimize the difference between two groups. Finally, the value of

$$\frac{N_1N_2(N_1+N_2 - P - 1)D^2}{P(N_1+N_2)(N_1+N_2-2)}$$

is calculated, and tested against an 'F' distribution with P and n +n -P-1 degrees of freedom to test significance of difference between two groups. D^2 is the Mahalonobis D^2 [(24),(26)]. This is accepted for testing two known populations. It has to be decided whether a population and a prediction relate to the same underlying predicting mechanism. Then any such prediction can be qualified to assist management in the decision making task.

Consider the context of this project. The procedure to assign probability classes to extensions of models is

1. Measure variables in both populations that are influencing variations in the predicted parameters.
2. Use Discriminant Analysis and Factor Analysis to eliminate composite variables and those variables with a high factor to weighting, but a low Discriminant Analysis weight ((26) suggests <0.01)
3. Classify into high risk/low risk category according to the following scheme:

2.3.9.2. LOW RISK EXTENSIONS: Factor contributions to the LDF from no more than one related factor.

HIGH RISK EXTENSIONS: Contributions from at least two factors related to the predicted population to the LDF.

In the project, it was shown that it is not valid to apply a river BOD-OD model to the Estuary for the period tidal cycle where fresh water flow predominates.

2.3.10. SUMMARY OF GENERAL MODELLING PHILOSOPHY

2.3.10.1. Users of models should consider the following points when interpreting a model system:

Models are only as good as their process formulations.

Simple Models are less accurate than complex models and their predictions allowed greater elasticity in interpretation.

Models often model interacting processes less well than independent parallel or series processes.

Feedback and optimizing models usually require more compitational resource.

A sensitive model is not necessarily an accurate model.

Errors are inherent in every phase, they can only be minimized.

Models are like deep freezes, one only gets out what is put in (GIGO principle), often after a long time interval, but looking different.

Models can only be applied to situations for which they were formulated, extrapolations must be thoughtful.

Models must be allowed to predict without 'assistance' from external agencies.

2.3.10.2. Pollution Models are mostly based on one conservation equation, that of conservation of mass

$$F = \Sigma Q_I\, C_I\, K_I + \Sigma Q_N\, C_N\, K_N - \Sigma Q_A\, C_A\, K_A$$

where suffix I = Input, N = Natural, A = Abstractions, Q is flow, C is concentration, K are kinetic transformation factors.

For conserved materials; $K_I, K_N, K_A = 1$.

For the solute $C_I, C_N, C_A = 1$, and as the solute is conserved

$$F_S = \Sigma Q_I + \Sigma Q_N - \Sigma Q_A = Q$$

This is the second basic equation used, the 'Flow Continuity' equation.

These equations recur frequently in varying guises and are basic physical facts of the system.

2.3.10.3. Some models will now be discussed in greater detail, each one being of historic significance in the progress of water quality modelsin the past fifty years, and covering the spectrum of approaches.

2.4. A Comparative Review of Some More Significant Models

2.4.1. STREETER-PHELPS MODEL

This is the first attempt to quantify the problem of dissolved oxygen levels in water bodies[32], and is of classic standard. The assumptions are simple. Bacteria working to stabilize the organic matter deplete the dissolved oxygen level, at equal rates. The D.O. in turn is replenished by surface absorbtion, reaeration.

$$\frac{dB}{dt} = -K_B B \qquad 2.4.1.A$$

B is BOD remaining, K_B is rate of removal.

$$\frac{dD}{dt} = -K_R D + K_B B \qquad 2.4.1.B.$$

D is D.O. deficit, K_R is rate of reaeration.

At time of introduction of the load under consideration, initial conditions were D_O and B_O. Integrating gives

$$B = B_{Oe}^{-KBt} \qquad 2.4.1.C.$$

$$D = \frac{K_B B}{(K_R - K_B)} \left(e^{-K_B t} - e^{-K_R t}\right) + D.\ e^{-K_B t} \qquad 2.4.1.D.$$

This equation does not relate well to the physical system because of extensive assumptions about constancy. It also refers to a static system, as time is the independent coupling variable. Assuming uniform velocity defined by

$$\frac{dx}{dt} = U \rightarrow x = Ut + XO$$

and changing the variable in the two expressions allows a relation in terms of displacement from point of origin.

When modelling of water quality management was attempted, there was a need to find an expression that would estimate water quality rapidly and effectively, with minimum inputs and concise outputs. So 50 years after formulation the Streeter-Phelps Model was examined and new properties discovered[33].

Differentiation of 2.4.1.C-D setting the derivative to zero and solving, gives the turning points where the maximum deficit is reached, after a time t_c

$$t_c = \frac{1}{(K_R - K_B)} \log \frac{K_R}{K_B} \quad 1 - \frac{(K_R - K_B)}{K_B} \cdot \frac{D_o}{B_o} \qquad 2.4.1.E.$$

$$D_c = \frac{K_B}{K_R} \cdot B_o \, e^{-K_B \cdot t_c} \qquad \text{where } D_c = \text{critical (max) deficit} \qquad 2.4.1.F.$$

The ratio K_R/K_B is the self purification f[34], $D_o B_o$ the deficit load ratio R_o, so

$$t_c = \frac{1}{K_{B(f-1)}} \log f \quad 1 - (f-1)R_o \qquad 2.4.1.G.$$

The long term must be positive for t_c to be defined, and one of the following must hold.

$$f < 1 \quad \text{and} \quad R_o > 0 \qquad 2.4.1.H.$$

or

$$f > 1 \quad \text{and} \quad 0 < R_o < \frac{1}{f-1} \qquad 2.4.1.I$$

to ensure a non negative tc.

It can be shown that by using a linear approximation of the form

$$\overline{D}_c = E + R B_o + AD_o \qquad 2.4.1.J.$$

errors are likely to be less than 6% of $\overline{D}_c$ and often only 1% in practical management problems. The precise values of E, R & A are obtained by minimizing the maximum error.

In a practical problem [37], the resultant equation

$$\overline{D}_c = 0.515 + 0.222 B_o + 0.542 D_o \qquad 2.4.1.K.$$

gave a maximum error of 0.098 p.p.m., where the deficit was allowed to rise up to 4 mg/l.

This form is easily assimilated into a standard optimizing package, and the linear approximation is used as such.

2.4.2. DOBBINS MODEL

Based on the Streeter-Phelps equations, this model represented a significant broadening of factors for inclusion [35]. Sedimentation, runoff sources, benthal layers, photosynthetic action are all included in the model.

Consider the concentration term c(x,t). Then by definition

$$dc = \frac{\partial c}{\partial x} . dx + \frac{\partial c}{\partial t} dt, \text{ so } \frac{dc}{dt} = \frac{\partial c}{\partial x} . \frac{dx}{dt} + \frac{\partial c}{\partial t} \qquad 2.4.2.A.$$

Again assuming $dx/dt = U$, $\frac{dc}{dt} = U \frac{\partial c}{\partial x} + \frac{\partial c}{\partial t}$. 2.4.2.B.

The derivative dc/dt represents the total change of c(x,t), and is due to diffusion and net source terms. Assuming that diffusion is Fickian, i.e. represented by rate of change of concentration gradient, the equation 2.4.2.B becomes

$$E \frac{\partial^2 c}{\partial t^2} + \Sigma S_o - \Sigma S_I = \frac{U\partial c}{\partial x} + \frac{\partial c}{\partial t} \qquad 2.4.2.C.$$

$$\frac{dc}{\partial t} = E \frac{\partial^2 c}{\partial t^2} - \frac{U\partial c}{\partial x} + \Sigma(\Delta S) \qquad 2.4.2.D.$$

Assuming the water body has reached steady state c(x,t) = c(x) and so $\frac{\partial c}{\partial t} = 0$ and reduces to a 2nd order differential equation. To facilitate solution, the following assumptions have to be made on the nature of source/sink terms:

A) The volume of the kinetics of each source/sink term.

B) The constancy of these within the water body being considered.

Dobbins also concluded that neglecting E in a stream gave an error of 0.4% in the final predictions. Although this value was later estimated as 10 times too small by Lynch[36], it is still a small error in relation to the mathematical simplification and the error accepted in field data.

Using the same symbols as in the Streeter-Phelps equation, can be written as

$$E.\frac{d^2B}{dx^2} - U.\frac{dB}{dx} - (K_B + K_N).\ B + B_R \qquad 2.4.2.E$$

$$E.\frac{d^2D}{dx^2} - U.\frac{dD}{dx} - K_R.D + K_B.B + D_B \qquad 2.4.2.F$$

where K_N is rate of decrease in BOD due to all non-oxygen consuming kinetics and D_B is oxygen demand due to benthic action. The D_B term includes photosynthetic action and assumed to be time constant to be consistent. B_R is BOD addition due to run off.

Using the E = 0 approximation, we derive

$$B = B_o e^{-(K_B + K_N)t} + \frac{B_R}{K_B+K_N} \quad 1 - e^{-(K_B+K_N)t} \qquad 2.4.2.G.$$

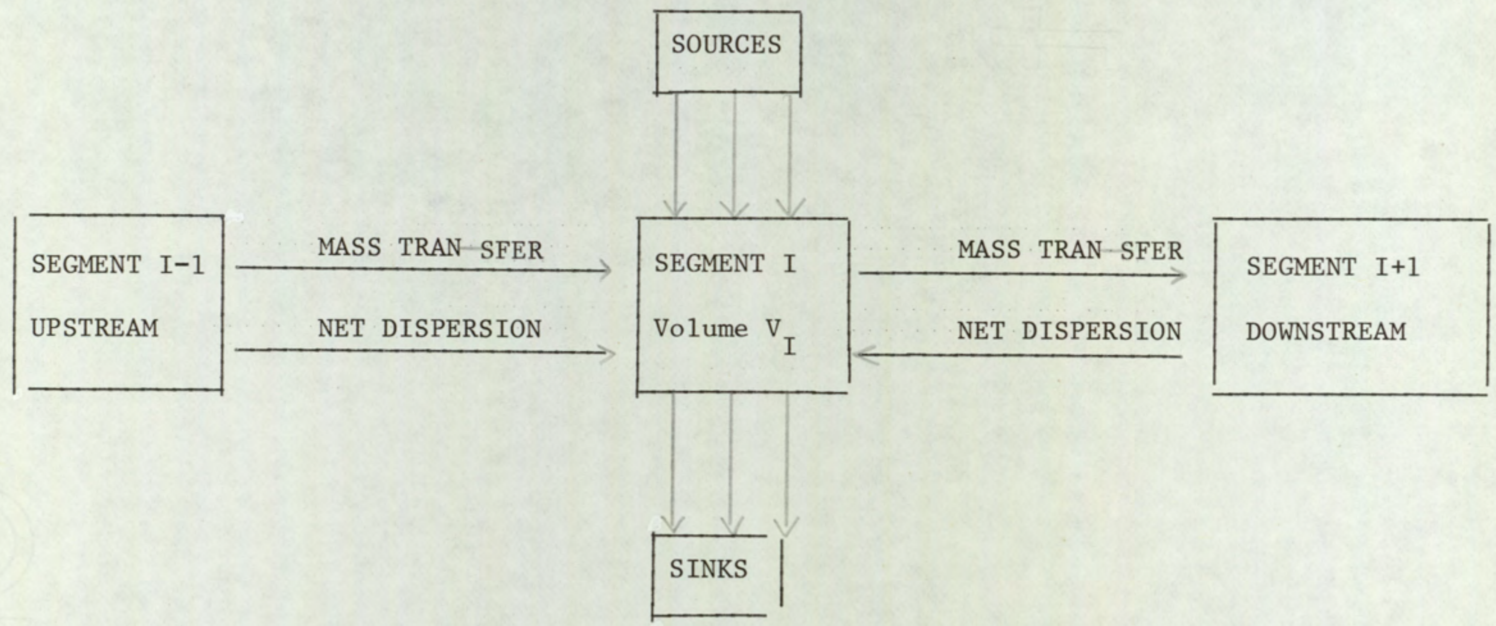
and a similar layer expression for D. If all factors Dobbins introduced are set to zero a 2.4.2.G reduces to the Streeter Phelps Model equations 2.4.1.C & D.

2.4.3. O'CONNORS MODEL

This is a development of the Dobbins Model (see 2.4.2) because it does not require the simplification E = 0 and os is more representative of a real system (37),(38),(39) Applied to the estuary situation, segments have constant parameters and E also allows for tidal advocation. As turbulent diffusion and apparent tidal diffusion are linked phenomena, it is possible to group the two effects. E will, however, be a function of position and time in these cases. The model has been refined by Thomann.

2.4.4. THOMANN'S MODEL

Essentially an extension of the O'Connor Model[40],[41]. Constant conditions within a segment are assumed. The process modelled may be shown by diagram



The equation to model the system is

$$V_i \frac{dC_i}{dt} = \frac{Q_i(C_{i-1}+C_i)}{2} - Q_i + \frac{(C_i+C_{i+1})}{2} + E_i(C_{i-1}-C_i)$$

$$+ E_{i+1}(C_{i+1}-C_i) + V_i\,\Sigma S_o - V_i\Sigma S_I \qquad 2.4.4.A.$$

where Q_i is net flow from segment i-1 to i.

E_i is the diffusion coefficient from segment i-1 to i.

C_i is the concentration, V_i the volume of segment i.

So are source terms, S_I are sink terms, summation to be over the segment only.

Two equations of the form 2.4.4.A. are obtained, one for the BOD process, one for the DO process.

By approximating $\frac{dc_i}{dt} = 0$, i.e. a steady state approximation, the solution becomes simple. The equations can be written in matrix form

$$\underset{\sim}{\mathbf{A}}.\underset{\sim}{c} = \underset{\sim}{b}$$

where $\underset{\sim}{b}$ is the vector of net (source-sink) segment terms.

As the matrix A of equation coefficients is tridiagonal, solutions are relatively easy to accomplish, being $\underset{\sim}{c} = \underset{\sim}{A}^{-1}.\underset{\sim}{b}$

The coupled equations have also been solved without the steady state approximation[42] to simulate tidal cycle variations.

2.4.5. A Distributed Parameter Model[43]

This is a novel approach to the problem of solving an equation like 2.4.2.D. Previous models discussed seek to approximate the differential equations derived by finite difference schemes of various complexities. The approach in this model is to approximate the final solution, not the similar restrictions as the Thomann model (ref. 2.4.4.). To represent segments, a cascade connected electrical network is established. A segment is a network block with voltage/current representing concentration/pollutant transport. The analysis are extended to model the other parameters, and the resultant network is analysed using a standard network analysis program[44],[45]. The state of the work currently makes it inappropriate to most practical applications as problems arise with a network of over 5 segments but there are generality reasons why this work should progress and prove fruitful.

2.4.6. BOX-JENKINS METHOD MODELS

Advances in environmental monitoring techniques in recent times has made bulk data acquisition more viable than early days of water quality management. Developing a model along lines of theory inevitably involve simplifying assumptions. If one could build a model based on actual historic data, then the model would include all sources of variance, the fitting being the limiting factor.

Box and Jenkins Time Series analysis use time as a base[46],[47], as opposed to frequency. An observed time series is a collection of events sequentially dependent, especially in the water quality field.

A frequency based method would seek to resolve all variations into the different frequency components, thus relating them to the varying factors. The time based method seeks to express variations by expressing the observed data as output from a linear filter with random input and transfer functions in series[48].

Models can be built stepwise to cope with variations in time scale[49] and it has been used very successfully[50].

<u>Mathematics of the Box Jenkins Method</u>

Define B, a Backward shift operator, so that $BZ_t = Z_{t-1}$

Define ∇, a Backward different operator, so that

$$\nabla Z_t = Z_t - Z_{t-1}$$

The model is

$$Z_t = \sum_{j=o}^{\infty} w_j a_{t-j}$$

where a_i are a series of random 'shocks', i.e. uncorrelated, w_j are weights and Z_i is the observed value output.
This can be written as

$$Z_t = \sum_{j=1}^{\infty} \{w^1{}_j Z_{t-j}\} + a_t$$

If one can assume that

$$z_t = \sum_{j=1}^{P} \phi_j z_{t-j} + a_t, \text{ so } a_t = \phi(B) z_t$$

then z_i is an autoregressive process (AR) of order p. The current observed value is a linear sum of finite historic observed values and a current shock a_t.

2.4.7. STOCHASTIC MODELS

The first major stochastic model (51),(52) was developed from the Dobbins model described earlier. Mechanism in the Dobbin model were modified for random processes. An additional assumption was made: that BOD/DO distributions could be discretized, i.e. put into small packets of a basic unit Δ. The state m then refers to a concentration of $m\Delta = S_m$. Consider a short time interval δt such that $(\delta t)^2$ is negligible. In that time several changes may occur: polluting loads/DO may increase or decrease, and the other mechanisms considered by Dobbins may change by $i\Delta$ states.

The increase in pollution due to B_o in time δt is B_o. t. Let P_1 be the probability that pollution increases by Δ in δt due to B_o.

$$P_1 = B_o . \delta t/\Delta \quad + O(\delta t)$$

Also $$P_2 = K_N . B_m \delta t/\Delta \quad + O(\delta t)$$

$$P_3 = K_B \, B_m \delta t/\Delta \quad + O(\delta t)$$

$$P_4 = B_B \delta t/\Delta \quad + O(\delta t)$$

$$P_5 \quad K_R D_n \delta t/\Delta \quad + O(\delta t)$$

where P_2 = probability of decrease in pollution due to non-oxygen consuming processes.

P_3 = probability of decrease in pollution due to oxygen consuming bacterial activity.

P_4 = probability of increase in pollution due to benthal demand.

P_5 = probability of decrease in D.O. deficit due to reaeration.

For the case of a moving average (MA) process:

$$\tilde{z}_t = \sum_{j=o}^{q} \theta_j a_{t-j} \text{ , so } \tilde{z}_t = \theta(B)a_t \text{ ,}$$

the current observation is a linear sum of previous deviations from the mean and a current shock a_t.

Combining the two gives an autoregressive moving average (ARMA) process

$$\phi(B)\tilde{z}_t = \theta(B)a_t$$

The mixed model of order (P,q) incorporates a finite number of previous observations, a finite number of previous deviations and a current shock a_t.
This is for a stationary process $\tilde{z}_t$. The models can now be extended to any level or combination required when employed, it has been found to require only a few terms, even though sometimes separated by months in the instances where seasonal trends are important.

There seems little doubt that this kind of model will be used more extensively with the advent of better on-line logger data integrated to management models.

The joint probability that the BOD is in state m and the OD in state n is $P_{m,n}$ (t) at time t.

A difference equation is obtained for $P_{m,n}(t+dt)$ in terms of $P_{m,n}(t)$ and the factors P to P . This can be solved for given boundary conditions to yield expected values and variances. The expectation values (means) were those predicted by the Dobbins Model.

Mouskegian and Krutchkoff[53] extended this model by segmenting a stream and using a cascade approximation that the probability output of one segment could be treated as the probability input into the next downstream segment. This was shown to be a very good sequential model.

2.4.8. STOCHASTIC MODEL OF CUSTER AND KRUTCHKOFF

This is an extension of the Thayer and Krutchkoff model[51],[52], to include estuaries using a random walk mechanism as opposed to the creation/annihilation process. The units of the processes are again in multiples of Δ. Consider a particle of BOD for a time interval of δt in an estuary $x = 0$ to $x = x_i$. In the time it can either move upstream by δx with probability $P_u(t)$, move downstream by δx with probability $P_D(t)$, or be absorbed by some process and last to the BOD system with a time independent probability P_R. By mutual exclusivity, one can write

$$P_u(t) + P_D(t) + P_r = 1$$

If P(m,n) is the probability that a unit of pollution is at $m\delta x$ after a total of n randomly generated steps, the time laps is n t, and it could only be in m x through being at (n-1) t at m-1 and P_D occurring or at m+1 and P_u occurring.

$$\therefore P(m,n) = P(m-1,n)\, P_D(\{n-1\}\delta t) + P(m+1,n) P_u(\{n-1\}\delta t)$$

Bearing in mind the Brownian motion nature of the (54),(55),(56) process the following interpretations are given to each term.

$$P_D(t) = \tfrac{1}{2}\{(1 - K_D\delta t) + U(t)\delta t\} \qquad 2.4.8.A.$$

$$P_U(t) = \tfrac{1}{2}\{(1 - K_D\delta t) - U(t)\delta t\} \qquad 2.4.8.B.$$

$$P_R = K_D\delta t \qquad 2.4.8.C.$$

where K_D = total BOD decay rate = $K_N + K_B$, Utt) is the complete velocity function, i.e. freshwater flow and tidal velocity.

Similar logic for the D.O. process leads to

$$\frac{\partial B}{\partial t}(x,t|to) = E^2\frac{\partial^2 B}{\partial x^2} - U(t)\frac{\partial B}{\partial x} - K_D B(x,t|to) \qquad 2.4.8.D.$$

$$\partial D(x_1 t|to) = E^2\frac{\partial^2 D}{\partial x^2} - U(t)\frac{\partial B}{\partial x} - K_R D(x,t|to)$$

$$+ K_B\, B(x,t|to) \qquad 2.4.8.E.$$

The solution of this for given boundary conditions forms the basis of this model.

2.5. MODELS SELECTED FOR THE PROJECT

3 Models were selected for the project to meet the differing needs of management.

2.5.1. A Steady State Model

Simply constructed and in concept it fulfilled three functions.

A Was yielding results within one year.

B Can be adapted later for cost optimizing

C Can be useful for broad trends and can also be adopted for a regional (as opposed to local) model.

2.5.2. A Time Dependent Model

A one dimensional river (the Usk) discharging into a two dimensional bay (the Severn). Programmed to predict minimum and maximum effect of a variation. Fulfills the management functions of

A Predicting extremes of effects.

B Allowing a suspected reservoir effect of the Severn Bay to be investigated.

C Having considerable spin off in terms of sheer hydrological predictions.

D Being modular and flexible.

2.5.3. A Stochastic Model

A one dimensional segmented estuary model using field data to calculate stochasticity. Fulfilled management functions of:

A Using data acquisitions from planned monitoring system.

B Giving error bounds on predictions.

C Having capability of investigating transient effects.

2.5.4. Each of these models will now be described in detail.

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CHAPTER THREE

THE STEADY STATE MODEL .

CONTENTS-CHAPTER 3

3.1 INTRODUCTION

The Steady State Model is a simple version of a full time dependant model. The model has been employed successfully in the Thames[2] and Severn[3] Estuaries. Its relative simplicity makes it an ideal management tool and with emphasis on input/output simplicity and intuitive comprehension it has been used directly by the WNWDA divisional management team for consent simulations.

This chapter seeks to outline the theory of the basic model and then show how the Steady State Model is derived.

3.2 The Conservation of Volume

This conservation equation is the basic building block of many models and requires careful derivation.There are several similar means of expressing this relationship, also known as the conservation of mass.

3.2.1 Consider a particle of fluid of infinitesmal volume δv, of density λ at any time t. As the particle moves under the influence of various forces, its mass cannot change. Mass is defined as the product of density and volume , it can be seen that

$$\frac{d}{dt}\left[\lambda \, \delta v \right] = 0 \qquad (3.2.1.A)$$

Should compression or expansion take place, this is reflected in λ and δv such that the product remains constant.

3.2.2 Alternatively, consider a closed surface S lying entirely in the body of a fluid and thus enclosing a volume V. if $\underline{u}$ is a unit normal inward for the element ds of S and $\underline{v}$ the velocity vector at the same point , then $-\lambda\underline{u}.\underline{v}.ds$ is the flow out of the surface element in unit time. The volume enclosed by S is constant once S is defined, so V is constant and thus

$$\int_V \lambda.dv \qquad (3.2.2.A)$$

Assuming no creation/annihilation of fluid within V, the computed loss can only be replenished by an equal flow into the boundary, which is, by Gauss's Theorem $\underline{\nabla}\ (\ \lambda.\underline{v}\ ).\delta v$ [4,5]

It can be seen that

$$\frac{\partial}{\partial t}\int_V \lambda.\delta v = \int \underline{v}.\underline{u}\ .ds = -\int_S \underline{\nabla}\ .(\lambda\underline{v}\ )\ \delta v \qquad (3.2.2.B)$$

and therefore $\int \left[\frac{\partial\lambda}{\partial t} + \underline{\nabla}\ .(\lambda.\underline{v}) \right] .\delta v = 0$ (3.2.2.C)

As the surface S integral is arbitrary, V is arbitrary and so

$$\frac{\partial\lambda}{\partial t} + \underline{\nabla}.\ \lambda.\underline{v} = 0 \text{ at every point} \qquad (3.2.2.D)$$

3.2.3 The step 3.2.2.C to 3.2.2.D is important . If an arbitrary volume V existed to satisfy 3.2.2.c, then

$$\int_V A.\delta v = 0 \text{ for any V, and so } \int_V A.\delta v = 0 \qquad (3.2.3.A)$$

Then also $\lim_{V\to 0} \frac{1}{V}\int A.\delta t = 0$,ie $\lim_{V\to 0} \frac{1}{V}.A.V = A$ and not $=0$ (3.2.3.B)

So if the arbitrary V existed, the closed surface S would need to vanish.

Therefore the step in the derivation is justified.

<u>3.2.4</u> Expanding the second term in 3.2.2.D gives

$$\frac{\partial\lambda}{\partial t} + \lambda\, \underline{\nabla}.\underline{v} + \underline{v}\ .(\ \underline{\nabla}.\lambda) = 0 \qquad (3.2.4.A)$$

Rewriting 3.2.2.D now gives $\underline{\nabla}.\underline{v} = \frac{d}{dt}\left[\ \log(1/\lambda)\right]$ (3.2.4.B)

<u>3.2.5</u> Equations 3.2.2.D,3.2.4.A and 3.2.4.B are all forms of the common CONTINUITY EQUATION for a fluid. Shoulld this fluid be incompressible, the above equations further reduce to

$$\underline{\nabla}\ .\ \underline{v}\ = 0 \text{ as } \lambda \text{ is constant} \qquad (3.2.5.A)$$

So for each unit volume it is possible to write the equation

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z} = 0 \qquad (3.2.5.B)$$

where v_x, v_y, v_z are the velocity components along the cartesian axes.

3.2.5.B is the General Equation of conttinuity for an incompressible fluid.

<u>3.2.6</u> The previous theory is now considered in the context of a one dimensional estuarine system at any point x_1

$$\frac{\partial V(x,t)}{\partial t} = Q_o(t) + \int_o^{x_1} q(x,t).dx \quad - \quad A(x,t).\ U(x,t) \qquad (3.2.6.A)$$

Rate of increase of vol in time at x_1	FWF in at head of the system	Sum of inputs between head of system and x_1	Flow out of point being the product of cross-sectional areas and velocity

(V-volumes,A-areas,Q and q-flow inputs,U-velocities in 1-dimension)

Rewriting and diffrentiating with respect to x gives :

$$\frac{\partial}{\partial x}\left(\frac{\partial V}{\partial t}\right) = \frac{\partial}{\partial t}\left(\frac{\partial V}{\partial x}\right) = \frac{\partial Q_o}{\partial x}(t) + q(x,t) - \frac{\partial}{\partial x}\left[A(x,t).U(x,t) \right] \qquad (3.2.6.B)$$

Now $\frac{\partial V}{\partial x} = A$ by definition and $\frac{\partial Q_o}{\partial x}(t) = 0$ as Q_o is the input at a fixed point, so

$$\frac{\partial A}{\partial t}(x,t) = q(x,t) - \frac{\partial}{\partial x}\left[A(x,t).U(x,t)\right] \qquad (3.2.6.C)$$

3.3 The Conservation of Pollutant

Let P be any property of the fluid under consideration. Using very similar logic to section 3.2.1, the equation

$$\frac{d}{dt}\left[\int_V P.\lambda.\delta v \right] = 0 \qquad (3.3.A\ \ )$$

for any volume V is derived.Carrying the temporal differentiation into the integral gives

$$\int_V \frac{dP}{dt} . \lambda . \delta v + \int_V P. \frac{d}{dt}(\lambda.\delta v) \qquad (3.3.B\ \ )$$

The second term is 3.2.1.A and so zero, so $\int_V \frac{dP}{dt}.\lambda.\delta v = 0$

This is the equation of conservation of a property of the fluid , in this case a pollutant being carried within the fluid undergoing other transformations.

3.3.1 By analogous logic, for one dimensional estuarine considerations, using 3.2.6.A , the following equation can be written :

$$\frac{\partial P}{\partial t}(x,t) = Q_o(t).C_o(t) + \int_o^{x_1} P(x,t).dx - \int_A U(x,t).c(x,t).dA \qquad (3.3.1.A)$$

where Q_o is as in 3.2.6, $C_o(t)$ is the concentration of the pollutant in a flow Q_o at time t , p(x,t) is the amount of pollutant added in the interval x=0 to the point under consideration, $x=x_1$. P(x,t) is the absolute mass and c(x,t) the concentration of the pollutant.

Partial Differentiation with respect to x gives the equation

$$\frac{\partial}{\partial x}\left(\frac{\partial P}{\partial t}\right) = \frac{\partial}{\partial x}(Q_o.C_o) + p(x,t) - \frac{\partial}{\partial x}\left[\int U(x,t).\,c(x,t).dA\right] = \frac{\partial}{\partial t}\left(\frac{\partial P}{\partial x}\right) \quad (3.3.1.B)$$

Where $\frac{\partial P}{\partial x}$ is the rate of change of mass of pollutant with repsect to the distance from the head of the system and can be written as A(x,t).c(x,t).

3.3.2 Decay Rates

The conservative equations derived only hold for sustained systems where either decay balances inputs at all times and all points or no decay or creating factors are operating. For most purposes, a first order decay is assumed. The rate of decay is k per unit of time. Equation 3.3.1.B then requires an extra sink term, this being -k.A(x,t).c(x,t). So

$$\frac{\partial}{\partial t}(A.c) = Q_o.C_o + p - \frac{\partial}{\partial x}\left[\int_A U.c.dA\right] - k.A.c \qquad (3.3.2.A)$$

3.3.3 Cross sectional averaging and the source of longitudinal dispersion.

The velocity and concentration terms are functions of both space and time. The dependance on x,y,z,t must be mapped to one on x,t only for a one-dimensional model. Define $\bar{u}$ and $\bar{c}$ as the mean values of u and c(x,y,z,t) over the range of y,z. The resultant is the mean value over the cross-section Similarly define u' and c' as departures from the defined means $\bar{u}$ and $\bar{c}$. Then

$$\int_A U(x,t).c(x,t).dA = \int_A (\bar{u}+u').(\bar{c}+c').dA = \int_A \bar{u}\bar{c}.dA + \int_A \bar{u}c'.dA$$

$$+\int_A u'.\bar{c}\,.dA + \int_A u'.c'.dA \qquad (3.3.3.A)$$

As, by definition, the mean values of u' and c' are zero, two of the terms in 3.3.3.A are zero, and so

$$\int_A u.c.dA = \bar{u}.\bar{c}.A + \int_A u'.c'.\,dA \qquad (3.3.3.B)$$

The term $\bar{u}.\bar{c}.A$ is the net mass transport per unit time. The variations in concentrations over the cross section combine with the variation in velocity to produce the second term. It is this term that produces the dispersion which skews a direct mass transport response function (fig. 3.3.3.C)

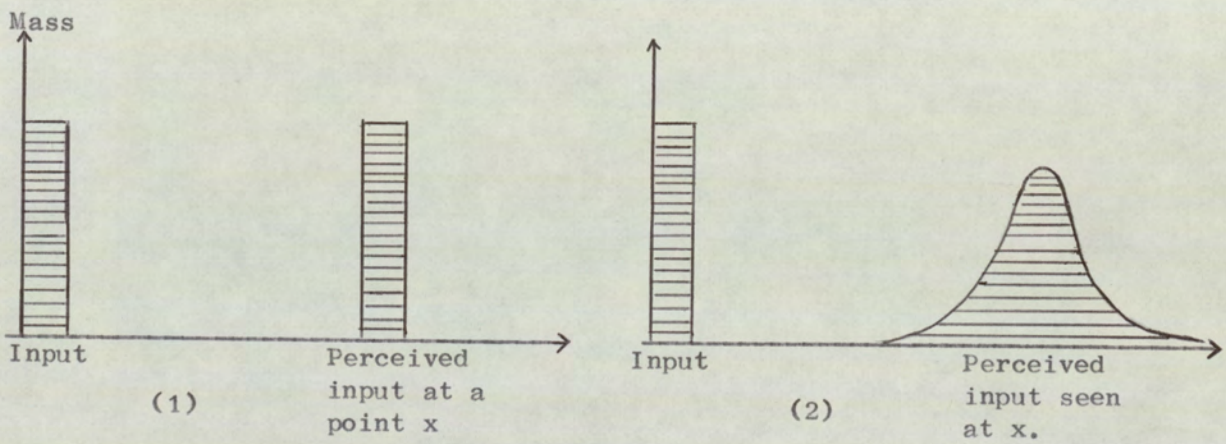

Fig 3.3.3.C Response to input without (1) and with (2) the presence of logitudinal dispersion.

3.3.4. A Dispersion Coefficient D(x,t)

A term may be defined to overcome the problems of solving the diffusive integral in 3.3.3.B. A dispersion coefficient D is defined as in 3.3.4.A in terms of per unit area per unit concentration gradient.

$$\int_A u'.c'.dA = -D(x,t)\frac{\partial c}{\partial x} \qquad (3.3.4.A)$$

The term is negative to show the net transport towards the area of lower concentrations[6].

3.3.5. The final conservation of pollutant equation is then

$$\frac{\partial}{\partial t}[A.c] = p + \frac{\partial}{\partial x}[D.A.\partial c] - \frac{\partial}{\partial x}[A.U.c] - k.A.c \qquad (3.3.5.A)$$

Coupled with 3.2.6.C, these two equations form the basis of the estuary model.

3.4 Tidal Velocities

The velocity u considered previously has been the total ,net, velocity seen in the system.In an estuary, the tidal velocity is an important factor and requires individual consideration. The velocity can be written as

$$U = U_{Tidal} + U_{Fresh\ Water} = U_t + \frac{Q(x,t)}{A(x,t)} \qquad (3.4.A)$$

where Q(x,t) is the sum of the first two right hand terms in 3.2.6.A.

3.2.6.A can now be written as

$$\frac{\partial V}{\partial t} = U.A - U_t.A - A.U \quad ,\text{ie } \frac{\partial V}{\partial t} + A.U_t = 0 \qquad (3.4.B)$$

As $\frac{\partial V}{\partial x} = A$, 3.4.B can now be written as $\frac{\partial V}{\partial t} + \frac{\partial V}{\partial x}.U_t = 0$ (3.4.C)

The interpretation of this is that any particle moving with a particle velocity equal to the tidal velocity has a constant volume upstream of it. A simple proof appears in [7] .

3.5 Combining the conservation equations

Combining the two derived equations ,

$\frac{\partial A}{\partial t} = q - \frac{\partial}{\partial x}\left[A.U\right]$ (3.2.6.C) and $\frac{\partial}{\partial t}\left[A.c\right] = p + \frac{\partial}{\partial x}\left[D.A.\frac{\partial c}{\partial x}\right] - \frac{\partial}{\partial x}\left[A.U.c\right] - k.A.c$

(3.2.5.A) . By expanding the term $\partial/\partial x\left[A.U.c\right]$, substituting for AU from 3.4.A , dividing by A and some rearrangement, results in

$$\frac{\partial c}{\partial t}(x,t) + U_t(x,t).\frac{\partial c}{\partial x}(x,t) = \frac{1}{A}\left[\frac{\partial}{\partial x}(D.A.\partial c) - \frac{\partial}{\partial x}(Q.C) + p\right] - k.c \qquad (3.5.A)$$

3.6 Estuary Salinity

Consider a salt particle moving with tidal velocity U_t ,[7] shows the conditions required for a constant salinity , with p and k both zero, are that the contents of [•...] in 3.5.A vanish, ie

$$D.A.\frac{\partial c}{\partial x} - Q.C = 0 \quad \text{where c is the salinity} \qquad (3.6.A)$$

This necessary condition suggests that the dispersive transport mechanism is balanced by the 'fresh water flow' transport mechanism.A plot of salinity at any point against cumulative volume upstream should result in a well defined curve. This method was developed by the Water Research Centre and used extensively on the Thames[8]. Even if 3.6.A were not strictly the equality shown, but only approximately vanishes, the concentration gradients are similarly small.This implies that particles moving at U_t experience minimal variations when compared to those occuring through time at a fixed point,or those at a fixed point occuring through time. As numerical solutions will eventually be employed to solve this system, it is then desirable to have to model minimum concentration changes.

3.7 Computerisation of the Equations

3.7.1 In order to facilitate a solution, the previously derived equations require modifications to evolve a difference scheme.

The estuary is segmented into N segments, with their boundaries lying at the points $X_i(t)$. These segments are moved with a velocity of U_t to take advantage of the concepts developed in 3.6

$$\frac{\partial X_i}{\partial t} = U_t(x,t) \qquad (3.7.1.A)$$

3.7.2 The conservation of volume equation is also rewritten (cf 3.2.6.A)

Rate of change of volume = Inflow in unit time + input to segment in unit time - outflow from segment in unit time

$$\frac{\partial}{\partial t}\Delta V_i = A_{i-1}(U_{i-1} - U_{t,i-1}) + \Delta Q_i - A_i.(U_i - U_{t,i}) \qquad (3.7.2)$$

The suffix i refers to the point X_i , ΔV_i is the volume of segment i.

A,U and U_t are as before. In 3.4 it was shown that $A_iU_i = U_{t,i}A_i + Q_i$, substituting in 3.7.2. gives

$$\frac{\partial}{\partial t}\Delta V_i = Q_{i-1} + \Delta Q_i + (-Q_i) = 0 \quad \text{(by definition of } Q, \Delta Q\text{)}$$

ie the volume of a segment is constant in time. As a corollary , the volume from x=0 to $x=x_i$ is constant, as it solely composed of fixed volume segments and this allows x_i to be calculated as a function of time [7,p74].

3.7.3 The mixing terms

These are used to simulate the logitudinal dispersion between adjacent segments, representing an equal and opposing interchange of water. The values of F_i are defined by

$$\frac{2\,D_i.A_i}{X_{i+1} - X_{i-1}} = F_i \qquad (3.7.3.A)$$

This representation will later allow A_i to be taken out of the equations.

The dispersion term is most difficult to establish. In practice , a method to estimate the mixing coefficients is used to provide an implicit estimate of dispersion. If there were no turbulent mixing in an estuary , the fresh water input would merely pass through the system in a stratified stream.In this case the assumption of small depth variation no longer holds.Also, were salt detected, it would be at the concentration found in seawater. In most cases,however, there is sufficient mixing to give a homogeneous system. The longitudinal salinity distribution is the result of mixing between the tidal surge and the fresh water head. The mixing coefficients are defined as

$$F_i = \frac{Q_i.(S_i - S_o)}{(S_{i+1} - S_i)} \qquad (3.7.3.B)$$

where S_i is the salinity in the ith segment and S_o is the salinity of the fresh water input at the head of the system.

<u>3.7.4</u> The conservation of pollutant equation (3.5.A) is now reconsidered.

$$\frac{\partial P(x,t)}{\partial t} = \frac{\partial}{\partial t}\left[\Delta V_i . c_i \right] = \Delta V_i . \frac{dc_i}{dt} \qquad (3.7.4.A)$$

$$\int_o^{X_i} p(x,t).dx = P_i + \int_0^{X_{i-1}} p(x,t).dx \qquad (3.7.4.B)$$

where P_i is the amount of pollutant added in X_{i-1} to X_i .

The rate of change of a pollutant in any segment i is :

$\frac{dP}{dt}$ = [Amount introduced through inflow]+[Amount introduced through mixing] +[Amount entering through external sources] - [Amount lost through out flow]-[Amount lost to mixing to next segment]-[Amount extracted by all external sinks]-[Amount lost through decay] (3.7.4.C)

Each of the terms in 3.7.4.C can be quantified in terms of F,c and p as follows :

Inflow into segment	$Q_{i-1}.c_{i-1}$
Inflow through additions	p_i
Loss through decay	$\Delta V_i.k.c_i$
Inflow through mixing	$F_{i-1}.(c_{i-1} - c_i)$
Loss through mixing	$F_i.(c_i - c_{i+1})$
Loss through outflow	$Q_i.c_i$

(the p_i terms the <u>net</u> source in a segment, ie sources-sinks.)

Substituting these into 3.7.4.C gives

$$\frac{dP}{dt} = \left[Q_{i-1}.c_{i-1} + p_i + F_{i-1}.(c_{i-1} - c_i)\right] - \left[k.c_i.\Delta V_i + F_i.(c_i - c_{i+1}) + Q_i.c_i\right] = \Delta V_i \frac{dc_i}{dt} \qquad (3.7.4.D)$$

The diffrential is approximated by $\dfrac{c_i^t - c_i^{t-1}}{\Delta t}$ and this allows 3.7.4.D to be solved using the Crank-Nicholson approximation[9,10].

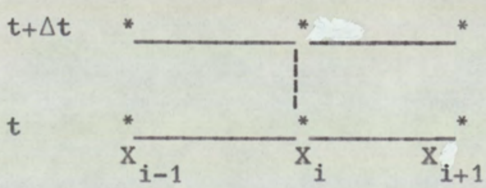

Fig. 3.7.4.E The grid for The Crank Nicholson Method

This method is inherently stable and implicit, with an error order of magnitude $(\Delta t)^2 + (\Delta x)^2$.

The complete time dependant iterative solution can now be written. The last term is the amount of pollutant entering the segment in the time step under consideration.This can have differrent functional representations .

$$\Delta V_i (c_i^t - c_i^{t-1}) = \frac{\Delta t}{2} \Big[(Q_{i-1}^t . c_{i-1}^t - Q_i^t . c_i^t) + (F_{i-1}^t . (c_{i-1}^t - c_i^t)$$

$$+ F_i^t . (c_{i+1}^t - c_i^t)) - k.\Delta V_i c_i^t \Big] + \frac{\Delta t}{2} \Big[(Q_{i-1}^t . c_{i-1}^{t-1}$$

$$- Q_i^t . c_i^{t-1}) + (F_{i-1}^t . (c_{i-1}^{t-1} - c_i^{t-1}) + F_i^t (c_{i+1}^{t-1} - c_i^{t-1})) -$$

$$k.\Delta V_i . c_i^{t-1} \Big] + \int_{t-1}^{t} p_i . dt \qquad (3.7.4.F)$$

If the boundary conditions $C_o(t)$ and $C_N(t)$ are known for all times of the simulations, the equation 3.7.4.F can be solved directly[11] or by the Gauss-Seidel method[12]. An estimate is made for $t=t_o$ if an initial set of field data is not available, and subsequent times are estimated from this.

3.8 <u>Time Averaging</u>

The model thus derived will predict concentration patterns over any time scale. Frequently it is the smaller time scale that is of interest, some knowledge of events within one tidal cycle of some shock transient input can be of great practical benefit. This project, being primarily concerned in the long term effects of variations of discharge consents, required long term trends and a method of time averaging was thus required.

Consider any integrable function $\Upsilon(t)$. A definition of its time-averaged value could be

$$\frac{1}{H} \int_{t_1}^{t_1+H} \Upsilon(t).dt = \bar{\Upsilon}(t) \Big|_{t_1}^{t_1+H} \qquad (3.8.A)$$

where t_1 and t_1+H represent the limits of the periods to be averaged. Using 3.7.4.D and T=one tidal cycle, N the number of whole cycles considered, gives

$$\Delta V_i \int_{t_1}^{t_1+NT} \frac{dc_i}{dt} . dt = \Delta V_i . \bar{c}_i \quad \text{where } \bar{c}_i \text{ is the mean value of } c_i \text{ in the period} \tag{3.8.B}$$

$$\int_{t_1}^{t_1+NT} \left[Q_{i-1} . c_{i-1} + p_i + F_{i-1} . (c_{i-1} - c_i) - k . c_i . \Delta V_i + F_i . (c_{i+1} - c_1) - Q_i c_i \right] . dt$$

$$= \Delta V_i . \bar{c}_i \tag{3.8.C}$$

If the fresh water flow is reasonably consistent, and rates of entry are constant,variations only arise from larger scale fluctuations. If N is 60 or more (ie at leats a lunar cycle) the the effects of the spring-neap tide cycle are included in the averaging and only seasonal factors are excluded.

$$\frac{1}{N.T} \int_{t_1}^{t_1+NT} \frac{dc_i}{dt} . dt = \frac{1}{NT} \left[c_i(t_1+NT) - c_i(t_1) \right] = 0 \text{ as NT is large} \tag{3.8.D}$$

So when substituted in 3.8.C gives

$$\bar{Q}_{i-1} . \bar{c}_{i-1} + \bar{P}_i + \bar{F}_{i-1} . (\bar{c}_{i-1} - \bar{c}_i) - k . \Delta V_i . \bar{c}_i + \bar{F}_i . (\bar{c}_{i+1} - \bar{c}_i) - \bar{Q}_i . \bar{c}_i = 0 \tag{3.8.E}$$

where a bar denoted a time averaged value of the parameter in question. Note that ΔV_i is still constant. This type of system,rewritten as

$$\alpha_{i-1} . \bar{c}_{i-1} + \alpha_i . \bar{c}_i + \alpha_{i+1} . \bar{c}_{i+1} = -\bar{p}_i \tag{3.8.F}$$

this being a band matrix of order m for an m segment estuary and is readily solved.

3.9 The Steady State

Equation 3.8.F is the basis of the stationary state model applied to the Usk Estuary. It will predict the mean value of a pollutant in a moving segment over a period of one lunar cycle. The variation in the neap-spring tide cycle is about 6m ,large by comparison to other british estuaries (eg 2.9m Gt. Ouse, 3.3 Humber) and so the mean value will have an inherently wide confidence limit attached to it when variations from the mean are considered.

3.10 The Chemistry of the Steady State Model

3.10.1 The chemistry may be summarised by figure 3.10.1.A which shows the primary links.

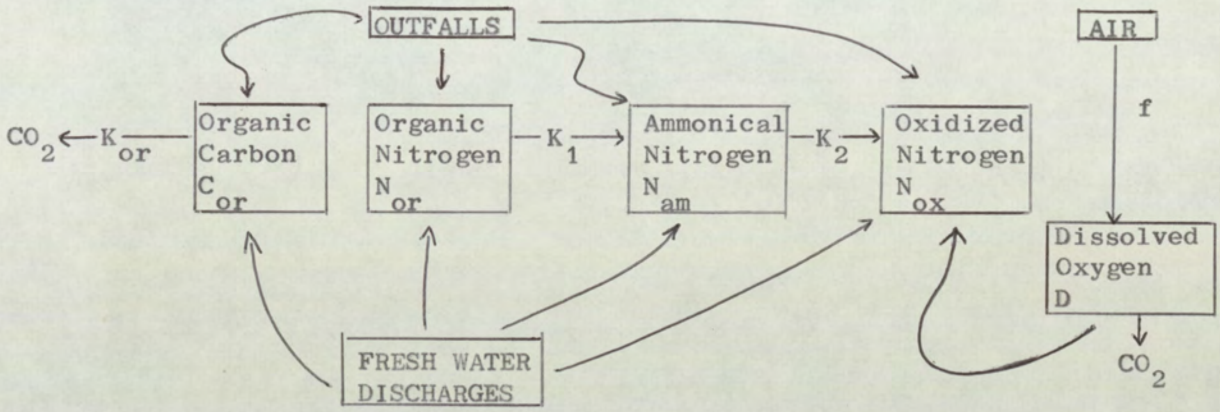

Fig. 3.10.1.A Source - Sink Terms for D.O. and common pollutants

The interconnections show possible reaction paths. These are not purely chemical reactions, with bacteria and related kinetics playing major roles[13] By reducing differing components to oxygen equivalents, direct comparisons are possible.

3.10.2 Carbonaeceous Effluent

The chemical reaction involved is $C + O_2 \text{ -- } CO_2$. Each unit of organic carbon requires 2*16/12 units of oxygen , ie an oxygen equivalent of 8/3 or 2.67 , for complete oxidation.The simplest representation of this oxidation process is the first order kinetics developed by Phelps[14].

$$\frac{dR}{dt} = -K.\,R \quad \text{where R is the residual D.O. deficit} \qquad (3.10.2.A)$$

This simple approach is unsatisfactory in many cases as the rate of reaction is dependant on the form the carbon appears in and which bacteria are available to catalyse the process. One approach used successfully is the method of composite rates[15].In this method, the effluent is considered to be a mix of a number of components, each a source of organic carbon, but each oxidized at an individual rate. Components can be isolated by tracing the percentage of the total oxygen demand excercised after some interval of time. Results show that normal sewage effluent has two main components only , one with a rate of decay 0.2 of the main 'fast' component. The equation is :

$$\text{Oxygen Uptake at time } t = \text{UOD.}\left[1. - p e^{-K_{or}t} - (1-p)\, e^{-K_{or}t/5}\right] \qquad (3.10.2.B)$$

UOD is the ultimate oxygen demand excercised (ie complete oxidation) and p is the proportion of the fast component present in the effluent.

Various experimental oxygen equivalents have been reported, very much dependant on what representation is given to the empirical formula of 'norm' sewage. Figures of 2.79[16] to 3.00[17] have been reported, but it can be shown that a 10 per cent error in this value is negligable[8] on final predictions. A value of 2.89 was used, this being a reported figure in the Thames Study. As oxidation has still been measured on effluents after several months [15], it seems that the primary point is to employ a figure representative of the process during the average retention time of the system under study.

3.10.3 Nitrogenous Effluents

Ammonia is the major component of this group of discharges. The outline reaction is $NH_3 + 2O_2 \longrightarrow HNO_3 + H_2O$, an oxygen equivalent of 2*2*16/14 = 4.57 . This is inclusive of an initial hydrolysis of organic nitrogen to ammonia. The oxidation of nitrite is written as $2HNO_2 + O_2 \rightarrow 2HNO_3$, an oxygen equivalent of 1.14 . Nitrite content of an estuary is often small. At low D.O. levels (<.5ppm) , nitrification is a prevalent factor. The nitrate formed under normal conditions, can be used as a source of oxygen for the breakdown of carbonaceous effluents [8,p.217,19].

$$NO_3 \longrightarrow NO_2 + O^{\bullet} \begin{array}{l} \nearrow N + 2O^{\bullet} \\ \searrow NH_3 + 2O^{\bullet} \end{array} \qquad (3.10.3.A)$$

3.10.4 Difference Equations for Effluent Decomposition

Consider any segment i in the system. The resident amount within it of any one pollutant over a time interval is [the initial content]+[Inflow]+[external input]-[outflow]-[decaying amount] (fig 3.10.1.A).

For organic carbon

$$\Delta V_i . \frac{dC}{dt}_{or,i} = \Delta V_i . I_i - K_{or/c,i} . \Delta V_i . C_{or,i} + C^{Add}_{or,i} \qquad (3.10.4.A)$$

For organic nitrogen

$$\Delta V_i . \frac{dN}{dt}_{or,i} = \Delta V_i . I_i - K_{or/N,i} . \Delta V_i . N_{or,i} + N^{Add}_{or,i} \qquad (3.10.4.B)$$

For ammonical nitrogen

$$\Delta V_i . \frac{dN}{dt}_{am,i} = \Delta V_i . I_i - K_{am/N,i} . \Delta V_i . N_{am,i} - K_{or/N,i} . \Delta V_i . N_{or,i} + N^{Add}_{am,i} \qquad (3.10.4.C)$$

For oxidized nitrogen

$$\Delta V_i . \frac{dN}{dt}_{ox,i} = \Delta V_i . I_i - K_{am/N,i} . \Delta V_i . N_{am,i} + N^{Add}_{ox,i} \qquad (3.10.4.D)$$

For dissolved oxygen

$$\Delta V_i . \frac{dD}{dt} = \Delta V_i . I_i + f_i . R_i [D_{s,i} - D_i] - K_{or/c} . \Delta V_i . C_{or,i} + D^{Add}_i - K_{or/N,i} . \Delta V_i . N_{or,i} \qquad (3.10.4.E)$$

The K's are reaction rates, f_i reaeration rates. The superscript 'Add' refers to additions within the segment i, I_i the influx to each segment. The value I_i is composed of flux into ith segment via dispersion ($F_{i-1}[C_{i-1}-C_i]-F_i[C_i-C_{i+1}]$ plus that due to land water flow ($Q_{i-1}C_{i-1}-Q_iC_i$).

$D_{s,i}$ is the saturation level of dissolved oxygen in segment i, R_i is the surface area of segment i. All these equations are similar to 3.7.4.D and can be solved in that way. When levels of oxygen fall below 0.5ppm (appx) the restricted oxidation of ammonia and the denitrification of oxidized nitrogen has to be catered for. This is accomplished by additional terms in 3.10.4.C-D-E to include additional source and sink terms. Initially the oxygen demand by the ammonia is calculated, and if this is not present or cannot be gained through reaeration, the difference is satisfied by reduction of oxides of nitrogen. In the Usk , indications are that this would not occur with present loadings until the fresh water input were approaching 40 mgd (ie well below Dry Weather Flow levels).

3.11 Modifications for the Usk Estuary

3.11.1 The Steady State Model as described has been applied successfully in two estuaries. The nature of the Usk was considered unusual in two aspects :

1. The number of outfalls and their variable tide-locked nature
2. The long tidal excursion due to the high tides in relation to the whole length of the estuary.

The origional program was to solve for all components iteratively. The model was restructured and made more modular and employed a more efficient method of solution of the four non-interacting components (fast and slow carbon, fast and slow nitrogen). Simulation of varying reaeration is then 50 per cent faster in terms of mill time.

The length of tidal excursion has a pronounced effect on the surface areas of a segment as it oscillates with the tidal motion. The data read in can be optionally moved up and down the estuary with tidal excursion of the segment in question and an adjusted set of data for surface areas employed in the main calculation phase.Comprehensive input/output options provide for online summary of predictions for management presentation.

3.11.2 Tidelocking Outfalls vary widely in the system, the period of no discharge lasting from 2 to 9 hours per tidal cycle. To attempt to cater for the individual dynamics of each outfall would introduce unecessary time dependant elements, and a prohibitive amount of field work. The following approximation was employed :

'A tidelocked discharge effuses from half ebb to half flood'

This implies that the associated tidal excursion segments receive unequal loads.The upstream segments receive the normal load and the downstream ones receive none , or an optional leak rate load only.The reservoir effect of the locking is calculated on the grounds of elapsed time and this is all loaded into the segment opposite the outfall at the time of opening after a period of tidelocking.This reservoir load is that which would have been discharged into the downstream segments.

Each outfall can be simulated as being either tidelocked over a variable period or freeflowing.Results showed that if all outfalls effused throughout the tidal cycle, the D.O. situation would improve.

3.12 Notes on Units

The units of the final predictions of pollutant levels are in concentration units , mg/litre or parts per million , even though the inputs are in imperial units. The predicted level of D.O. is also calculated in terms of percentage of saturation at ambient temperatures and salinities.

The input data units are to some extent arbitrary as long as they are overall consistent.Conversion factors are part of the input parameters and are intended to bring a variety of units into a cohesive system.Depth,surface areas,volumes and temperaturs are in feet, millions of square/cubic feet and deg.C. Salinities are in grams per litre or parts per thousand , head water inflow is in m.g.d.(million gallons per day) and later converted to millions of lds per day by FCTRL. The reaeration rate is in cms/hr. and mixing coefficients in millions of lbs. per day.All the input arrays for discharge additions are in lbs per day,so that on the addition to millions of pounds per day units,concentrations in p.p.m. result.

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Table 3.12.A Three Consistent sets of Input Units

	Imperial	Mixed	Metric
Mixing Coeff.	10^6lbs/day	10^6lbs/day	10^6kg/day
Flow rates	10^6lbs/day	10^6lbs/day	10^6kg/day
Polluting Load	lbs/day	lbs/day	kg/day
Surface Areas	10^6sq.ft	10^6sq.m	10^3sq.m
Volumes	10^6cu.ft	10^6cu.m	10^3cu.m
Re-aeration	ft/day	m/day	m/day
FACTOR	62.3	2200	1

Table 3.12.B Constants and Units currently employed in the Model

Variable Name	Units or Value
NSEG,IPRINT,NC,MAXCNT OMFC,OMSC,OMAM,OMN,OMD	Dimensionless,user specified constants
DOXMIN	mg/l(ppm) set to o.4 ,ie 4% saturation
RKC,RKN	per day rate constants - .183
RKAMM	per day rate constant - .26
RKNO3	per day rate constant - .004
COXNO3	Chemical Equivalents 2.86
OXA	Chemical Equivalents 4.57
ERROR	Solution accuracy - mg/l(ppm)
FFLOW,FFFLOW,F	10^6lbs/day.Input converted by multiplier FCTRL
TEMP	Degrees Centigrade
DISTAN	Units of Distance.
REAER	cms/hr
SA	10^6 sq.ft
V	Cu.ft. FACTOR converts to 10^6lbs/day
DEPTH	Feet
FCADD,SCADD,FNADD,SNADD AMMADD,ANO3ADD,DOXADD	After multiplying by FCTRL,in lbs/day

CHAPTER 4

A TIME DEPENDANT MIXED DIMENSION MODEL FOR AN ESTUARY / BAY SYSTEM

SUCH AS THE USK - SEVERN

Contents

CHAPTER 4

A time dependent, mixed dimension model for the estuary/bay system such as the Usk/Severn

4.1 Introduction

4.1.1 The 'model' consists of a suite of programs which can be composed in various combinations to provide the following models

A. A Hydrodynamic one dimensional model of estuarine flow

B. A Hydrodynamic one dimensional estuary/two dimensional bay flow model

C. A Water Quality model for a one dimensional estuarine system

D. A Water Quality model for a one dimensional estuary/two dimensional bay system.

E. A velocity prediction model for input to the semi-stochastic model

The flexibility available was ideal for the project, where management objectives were broadening constantly with the progress in modelling.

4.1.2 There are four separate models:

F1 - One dimensional estuarine system, hydrodynamics.
F2 - Estuary/bay model, hydrodynamics.
PT1 - One dimensional estuarine system, pollutant transport.
PT - Estuary/bay model, pollutant transport.

The mathematical development for each of the four models is different and autonomous and each topic will be dealt with in isolation.

4.2 The basic equations of nonviscous unsteady flow

4.2.1 Propagation motion of long waves is complex but does have some pattern and is generally mathematically continuous. The coordinate axis are x,y,z with z in the vertical plane, see Fig. 4.2.1A.

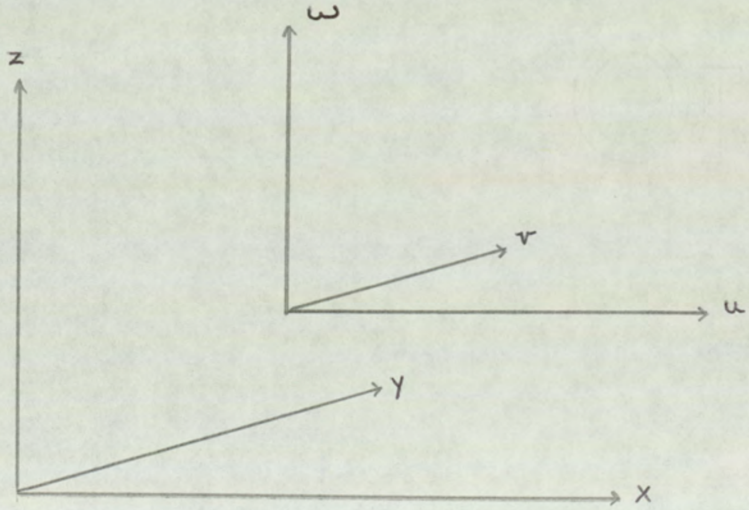


Figure 4.2.1A Coordinate axis in use

The full equations are written as a starting point for development

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z} = -\frac{1}{\rho}\frac{\partial p}{\partial x} + \sum F_x \qquad (4.2.1.A)$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + w\frac{\partial v}{\partial z} = -\frac{1}{\rho}\frac{\partial p}{\partial x} + \sum F_y \qquad (4.2.1.B)$$

$$\frac{\partial w}{\partial t} + u\,\frac{\partial w}{\partial x} + v\,\frac{\partial w}{\partial y} + w\,\frac{\partial w}{\partial z} = -\,\frac{1}{\rho}\,\frac{\partial p}{\partial z} \sum F_z \qquad (4.2.1.C)$$

where u,v,w are velocities in the x,y,z directions, ρ is the density of the medium. F_x, F_y, F_z are components of individual forces parallel to the x,y,z axis and p is the pressure component. The extraneous forces are combinations of earth rotation, tide generation due to extra terrestial bodies and vertical gravity.

The equation of continuity for incompressible flow used is

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \qquad (4.2.1.D)$$

Long waves are defined as waves where the amplitude is small compared to the wavelength. This allows velocities in the ω direction to be neglected and also accelerations are small compared to g. This particularly simplifies equation 4.2.1.C to

$$0 = -\,\frac{1}{\rho}\;\frac{\partial p}{\partial z} + \sum F_z \qquad (4.2.1.E)$$

4.2.2 Two depth variables are used, one relating the depth of water from an unperturbed state, and one relating the perturbation height of water at any one time, figure 4.2.2A.

4.3 The Theory of the Two Dimensional Bay Model

4.3.1 The model was first developed to model nuclear explosion tidal waves[1] and includes all of the major influences found in the project system, and is very responsive.

The nomenclature of fig. 4.2.2A is used. As variations in the ω direction are to be neglected, the velocity components u and v are now altered in definition to velocities averaged in the ω direction, i.e.

$$\bar{u} \text{ or } \bar{v} = \frac{1}{(h + \zeta)} \int_{-h}^{+\zeta} (u \text{ or } v)\, dz \qquad (4.3.1.A)$$

Equations 4.2.1.A and B can now be written as

$$\frac{\partial \bar{u}}{\partial t} + \bar{u}\frac{\partial \bar{u}}{\partial x} + \bar{v}\frac{\partial \bar{u}}{\partial y} = -\frac{1}{\rho}\frac{\partial p}{\partial x} + \sum F_x \qquad (4.3.1.B)$$

$$\frac{\partial \bar{v}}{\partial t} + \bar{u}\frac{\partial \bar{v}}{\partial t} + \bar{v}\frac{\partial \bar{v}}{\partial y} = -\frac{1}{\rho}\frac{\partial p}{\partial y} + \sum F_y \qquad (4.3.1.C)$$

and 4.2.1.E as: $\frac{1}{\rho}\frac{\partial p}{\partial z} = \sum F_z$ (4.3.1.D)

The generating forces must now be itemised to allow solutions to the above.

4.3.2 Forces in the z direction

From 4.3.1.D, $\frac{\partial p}{\partial z} = \rho \sum F_z$, or $p(z) = [\rho\, F_z . z]_{z=h}^{z=\zeta}$.

Assuming uniform density, the pressure is the hydrostatic pressure exerted by a liquid column, and

$$p(z) = g\ \rho(h-z) + p(0) \tag{4.3.2.A}$$

where $p(0)$ is the atmospheric pressure p_a and ρ is water density. 4.3.2.A allows an alternative expression for the right hand derivatives in 4.3.1.B and C to be obtained

$$\frac{\partial p}{\partial x} = g\ \rho\ \frac{\partial h}{\partial x} + \frac{\partial p_a}{\partial x}\ , \quad \frac{\partial p}{\partial y} = g\ \rho\ \frac{\partial h}{\partial y} + \frac{\partial p_a}{\partial y} \tag{4.3.2.B}$$

Generally it can be considered that derivatives of p_a with respect to (x,y,t) are zero. However, in a situation of a storm surge or a transient partial vacuum this would be an insufficient approximation.

In this model, it is assumed

$$\frac{\partial p}{\partial x} \simeq g\ \rho\ \frac{\partial h}{\partial x} \quad \text{and} \quad \frac{\partial p}{\partial y} \simeq g\ \rho\ \frac{\partial h}{\partial y} \tag{4.3.2.C}$$

4.3.3 Components of the Coriolis Acceleration

As the coordinate axes are fixed, the earth's rotation generates a resultant force, manifest by particle acceleration in the (x,y) plane. The Coriolis acceleration allows the use of a local axes, in a moving reference frame, instead of the fixed axes with origin at centre of the earth. Let ω be angular velocity of earth's revolution, θ = angle of latitude.

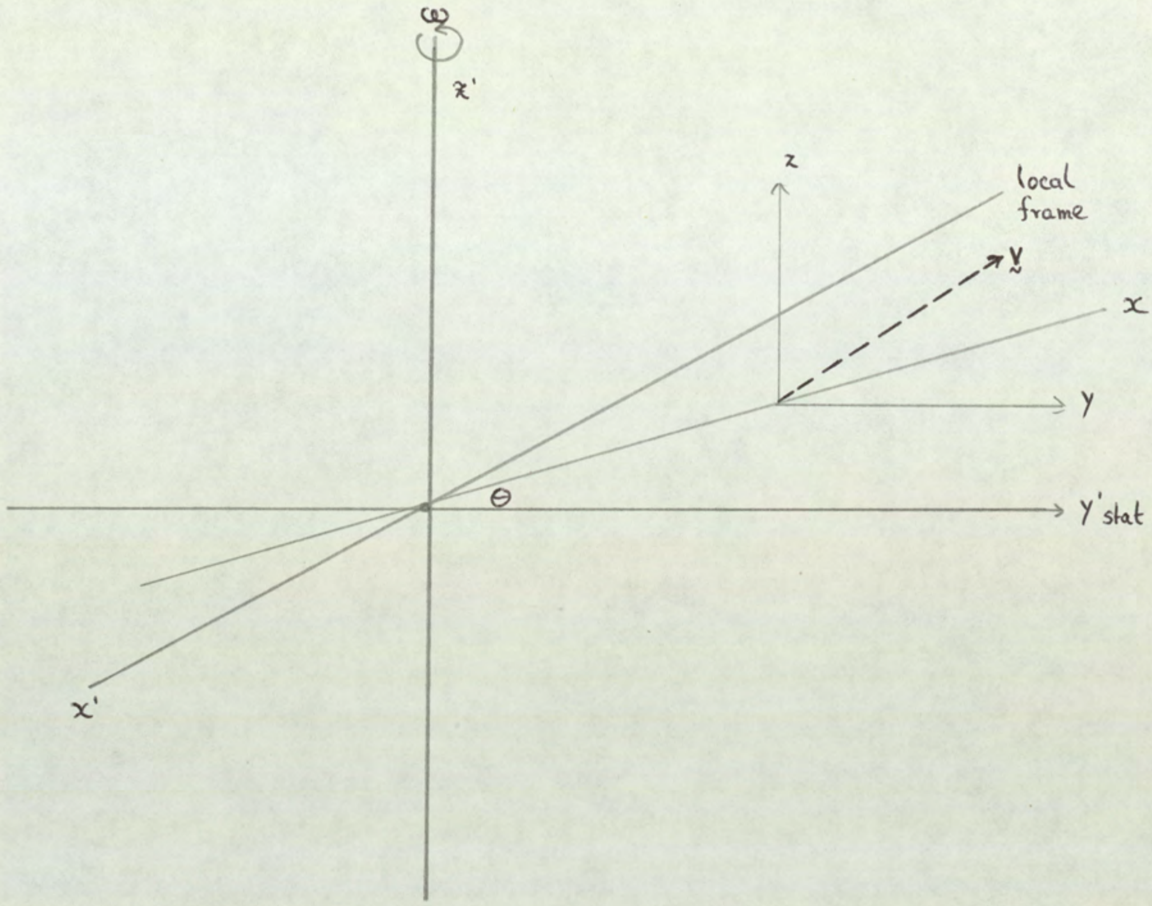


Figure 4.3.3A Relation of two reference frames

When the transformations for acceleration in x and y direction components are obtained, the fixed system (primed reference frame) coordinates may be used instead of the moving frame if centrifugal acceleration and Coriolis acceleration are included. The centrifugal acceleration components are $-\omega^2 x$, $-\omega^2 y$ and are included in gravity factor g.

The Coriolis component of the transformation is

$$v' = -(2\omega \sin\theta)\, v = -\Omega v, \text{ x direction}$$

$$u' = (2\omega \sin\theta)\, u = \Omega u, \text{ y direction}$$

The z component can also be calculated but is negligible[2] in this context.

4.3.4 Frictional Forces from Bottom Friction

In the well established Navier Stokes equation[3],[4],[5]

$$\rho \frac{d\underset{\sim}{v}}{dt} = \nabla p + \mu \, \Delta \, \underset{\sim}{v} + \rho \, \underset{\sim}{F} \tag{4.3.4.A}$$

the term $\mu \, \Delta \, \underset{\sim}{v} = \mu \, \nabla^2 \, \underset{\sim}{v}$ is the friction term defined by

$$\left(\frac{\partial^2}{\partial x^2} \underset{\sim}{v} + \frac{\partial^2}{\partial y^2} \underset{\sim}{v} + \frac{\partial^2}{\partial z^2} \underset{\sim}{v} \right) \tag{4.3.4.B}$$

In the model the u and v components are of prime interest. The coefficient, μ, of dynamic viscosity is replaced by ε_i, coefficients of eddy viscosity. These coefficients ε_i are directional. The friction terms can be written as

$$\varepsilon_H \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) + \varepsilon_v \frac{\partial^2 v}{\partial z^2} \quad \text{(x term)} \tag{4.3.4.C}$$

$$\varepsilon_H \left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) + \varepsilon_v \frac{\partial^2 v}{\partial z^2} \quad \text{(y term)} \tag{4.3.4.D}$$

These two expressions are integrated in the z direction to yield

$$\varepsilon_H \left(\frac{\partial^2 \bar{u}}{\partial x^2} + \frac{\partial^2 \bar{u}}{\partial y^2} \right) + \frac{\varepsilon_v}{\bar{h}+h} \left[\left. \frac{\partial u}{\partial z} \right|_{z=h} - \left. \frac{\partial u}{\partial z} \right|_{z=-\bar{h}} \right] \tag{4.3.4.E}$$

$$\varepsilon_H \left(\frac{\partial^2 \bar{v}}{\partial x^2} + \frac{\partial^2 \bar{v}}{\partial y^2} \right) + \frac{\varepsilon_v}{\bar{h}+h} \left[\left. \frac{\partial v}{\partial z} \right|_{z=h} - \left. \frac{\partial v}{\partial z} \right|_{z=-\bar{h}} \right] \tag{4.3.4.F}$$

where $\bar{u}$, $\bar{v}$ are mean velocities over depth and $\bar{h}$ is the depth with respect to mean water level.

In most physical systems $\varepsilon_v \gg \varepsilon_H$ and terms in ε_H are therefore neglected[6],[7].

The following physical interpretation can now be placed on each term:

$$\varepsilon_v \left(\frac{\partial u}{\partial z}\right)_h = x \text{ component of tangential stress at sea surface} = T_{s,x} \qquad (4.3.4.G)$$

$$\varepsilon_v \left(\frac{\partial v}{\partial z}\right)_h = y \text{ component of tangential stress at sea surface} = T_{s,y} \qquad (4.3.4.H)$$

$$\varepsilon_v \left(\frac{\partial u}{\partial z}\right)_{-\bar{h}} = x \text{ component of tangential stress at sea bottom} = T_{b,x} \qquad (4.3.4.I)$$

$$\varepsilon_v \left(\frac{\partial v}{\partial z}\right)_{-\bar{h}} = y \text{ component of tangential stress at sea bottom} = T_{b,y} \qquad (4.3.4.J)$$

The bottom stress can also be expressed by

$$T_b = \frac{\rho\, g\, \bar{u}^2}{C^2} \quad \text{where C is the De Chezy coefficient.}$$

When applied to the two dimensional system

$$\rho F_x^b = \frac{-\rho\, g}{C^2(\bar{h}+h)} |\underset{\sim}{v}|u \quad \text{and} \quad \rho F_y^b = \frac{-\rho\, g}{C^2(\bar{h}+h)} |\underset{\sim}{v}|u \qquad (4.3.4.K)$$

where F_x^b, F_y^b are the components of F of (4.3.4.A) due to bottom friction. The negative sign is to indicate that friction opposes the direction of motion. u and v are the components of $\underset{\sim}{v}$ and

$$|\underset{\sim}{v}| = +(u^2+u^2)^{\frac{1}{2}}$$

4.3.5 Forces from Wind Stresses

The principles of wind stress are similar to those of 4.3.4. The major extension occurs in shallow water where the vertical velocity distribution is skewed by wind action on the surface. Consequently, the surface gradient is additionally influenced by $T_{b,x}$, $T_{b,y}$. The theory of wind effects is complex and still developing[8],[9],[10], and the approximation

$$T_s = \theta^2 \rho_a V_W^2 \tag{4.3.5.A}$$

where ρ_a = density of air, V_W = wind velocity; is employed.

θ^2 is found experimentally to be about 2.6×10^{-3} for a V_W range of 6 to 20 m/sec (i.e. ~ 13 m.p.h. to 40 m.p.h.). ρ_a can be considered as 1.3 gm/litre, so $\theta^2 \rho_a$ is 3.4×10^{-6}.

For a wind of V_W metres/second at a constant direction degrees to the x-axis, the two components can be written as

$$F_x^s = \theta^2 \rho_a . \frac{V_W^2}{(\bar{h}+h)} . \cos\psi$$

$$\text{and} \quad F_y^s = \theta^2 \rho_a . \frac{V_W^2}{(\bar{h}+h)} . \sin\psi \tag{4.3.4.B}$$

Although θ^2 is usually within the wind range, in shallow systems, where the ratio T_b/T_s becomes significant, it is advisable to calculate a mean value from observation.

4.3.6 Final Theoretical Equations

Summarising the formulae developed in 4.3.2 - 4.3.5 and substituting in 4.3.1.B and C gives

$$\rho \left\{\frac{\partial \bar{u}}{\partial t} + \bar{u}\,\frac{\partial \bar{u}}{\partial x} + \bar{v}\,\frac{\partial \bar{u}}{\partial y} - \Omega\,\bar{v} + \frac{g|\underset{\sim}{\bar{v}}|\bar{u}}{C^2(\bar{h}+h)} - \frac{g\partial h}{\partial x}\right\} = \frac{\theta^2\ \rho_a\ V_W^2 \cos\psi}{(\bar{h}+h)}$$

(4.3.6.A)

$$\rho \left\{\frac{\partial \bar{v}}{\partial t} + v\,\frac{\partial \bar{v}}{\partial t} + \bar{v}\,\frac{\partial \bar{v}}{\partial y} + \Omega\,\bar{u} + \frac{g|\underset{\sim}{\bar{v}}|\bar{v}}{C^2(\bar{h}+h)} - \frac{g\partial h}{\partial y}\right\} = \frac{\theta^2\ \rho_a\ V_W^2 \sin\psi}{(\bar{h}+h)}$$

(4.3.6.B)

$$\frac{\partial}{\partial x}\,(\bar{h}+h)\bar{u} + \frac{\partial}{\partial y}\,(\bar{h}+h)\bar{v} + \frac{\partial h}{\partial t} = 0$$ (two dimensional equation of continuity) (4.3.6.C)

The mean operator from u and v will now be dropped. Any future reference to u and v implies the mean u or mean v over the water depth.

4.4 The Theory of the One dimensional River or Estuary Model

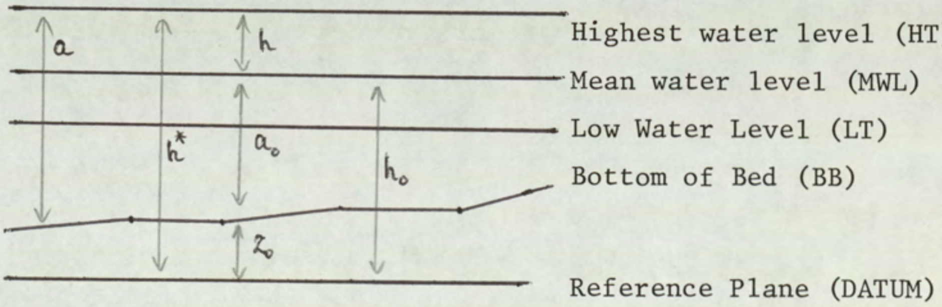
4.4.1 Introduction

A river is a water body where the flow of water is predominantly in one direction and one physical dimension (maximum width) is an order of magnitude smaller than the length. In an estuary situation the long term flow is that of a river with a net throughflow of the 'fresh' water inflow at the tidal limit.

The tidal element of an estuary can be considered to consist of a series of local flow reversals. Most models assume complete vertical mixing, i.e. concentrations of solutes are independent of depths. These allow the system to be modelled in one physical dimension and time. In stratified estuaries, two dimensional models are required to model the skewed vertical distributions and models/observations become more complex[2],[12].

It is possible to compensate to an extent by adjusting the bottom friction. This is because the nature of the tidal propagation is highly dependent, during the flood phase, on the bottom roughness[13]. At slack water, the friction becomes less significant.

The x coordinate is considered to be along the centre line of the estuary, and it is usual to reference all sections to a common datum plane. Figure 4.2.2A shows the relevant notation



Symbols: $h_o = a_o + z_o$: - mean water level with respect to DATUM

h = deviation of water level from mean

$a = a_o + h_{max}$: - depth of high tide

a_o = mean depth of water, averaged over time = $\bar{h}$

Figure 4.2.2A Notation Used

4.4.2 Reduction of the Two Dimensional Equations to One Dimension

The expressions 4.3.6.A, B and C are the basis of the one dimensional equations. All terms in y are neglected, and they can be written as

$$\rho \{\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + g \frac{\partial h^*}{\partial x} + \frac{g|u|u}{C^2.a}\} = \frac{\theta^2 \rho_a V_W^2 \cos \psi}{a} \qquad (4.4.2.A)$$

where $|u|$ = absolute value of u

$$\frac{\partial}{\partial x}(au) + \frac{\partial h^*}{\partial t} = 0, \text{ one dimensional equation of continuity} \qquad (4.4.2.B)$$

Equation 4.3.6.B can be included as

$$\rho \Omega u = -g \rho \frac{\partial h^*}{\partial y} + \frac{\theta^2 \rho_a \sin \psi}{a}, \qquad (4.4.2.C)$$

which would allow small lateral variation. In wider rivers the Coriolis force may perceptably alter the water height without causing an appreciable lateral velocity component, in which case 4.4.2.C will have to be included in further developments. Kelvin wave motion is not considered important in this project context, nor transverse wind components, as the estuary is quite narrow in most areas. So 4.4.2.A can be simplified to

$$\rho \{\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + \frac{g\partial h^*}{x} + \frac{g|u|u}{C^2.a}\} = 0 \qquad (4.4.2.D)$$

This equation, rewritten in terms of conservation of momentum[11], yields

$$\rho \left\{\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + g \frac{\partial z}{\partial x}\right\} = g\left(\frac{\partial z_o}{\partial x} - S_e\right) \qquad (4.4.2.E)$$

where z is the depth above the channel bottom, z_o is the height of channel bottom with reference to datum plane and S_e is the slope of the energy grade line. The energy grade line slope is approximated by

$$S_e = \left(\frac{n}{1.49}\right)^2 \frac{u^2}{R^{4/3}} \qquad (4.4.2.F)$$

where n is Mannings friction factor and R is the hydraulic radius.

4.4.3 The One Dimensional Equation of Continuity

Equation 4.4.2.B requires slight modification to be of use in situations where external sources may be important.

Let $b_n(x,t)$ be the width of the river bed. Let b_s be the 'storage width', i.e. width of water surface so that $b_s \geqslant b_n$. For a.u in 4.4.2.B write Q(x,t), then the equation

$$\frac{\partial Q(x,t)}{\partial x} + b_s(x,t) \frac{\partial h}{\partial t} = 0 \qquad (4.4.3.A)$$

This can be interpreted as the difference in discharge across area a, a + da at the respective points x and x + dx. A factor for external flow disruption will have to be included. If R is the rate of addition through some external agency, the equation

4.4.3.A is modified to read

$$\frac{\partial Q}{\partial x} + b_s \frac{\partial h}{\partial t} + R = 0 \qquad (4.4.3.B)$$

If these flow disruptions are a high proportion of the main channel flow, further energy factors will have to be considered. For example, if a discharge is perpendicular to the main stream flow, the discharged mass has no momentum in the direction of general flow. This has to be assimilated from the energy of motion and thus will result in a degradation of the main flow velocity. In the case of an abstraction the momentum of the abstraction is gradually reduced from the main stream value to zero through bottom or internal friction once the storage area is reached.

4.5 Modification of the Two Dimensional Hydrodynamic Unsteady Flow Equations Prior to Solution

4.5.1 Introduction

The equations to be solved are 4.3.6.A, B and C once the following points have been noted.

A) Variation in barometric pressure in x and y directions are neglieible.

B) Velocities are assumed to be vertically averaged in x and y direction

C) Density of water is unitary and constant. $\rho_{new} = \rho_{air}$.

4.6 Some Properties of Finite Difference Schemes

4.6.1 Introduction

The choice of solution scheme for the coupled system 4.5.1.A, B and C is the major item for a modelling project. This is due to the vast choice of methods available and the underlying applications that discuss options of choice [11, 14, 15]. For any particular problem, the choice of method will depend on local considerations such as:

A) Power of solving tools available: (slide rule - computer)

B) Economics of solving tool used : (free - commercial computing rates)

C) Is the method applicable : (Are underlying assumptions compatible with that of model)

D) Accuracy required : (less accurate methods require less effort)

E) Simulation time scale : (tactical-strategic model?)

4.6.2 Finite Difference Approximation - Order of a scheme

For purpose of discussion, consider the one dimensional wave motion equations:

$$\frac{\partial s}{\partial t} + h\,\frac{\partial u}{\partial x} = 0 \quad \text{and} \quad \frac{\partial u}{\partial t} + g\,\frac{\partial s}{\partial t} + \frac{\partial p_a}{\partial x} = 0 \qquad (4.6.2.A)$$

where $S(x,t)$ = water level, $u(x,t)$ water velocity, $p_a(x,t)$ atmospheric pressure and h = water depth. If a set of initial conditions are established for $x \rightarrow 0$ to x_o then the equations 4.6.2.A give the solution for $\forall\ t > 0$ if boundary definitions are known.

D) S is the deviation of the water level from the mean level.

The principal equations can be written

$$\frac{\partial u}{\partial t} + u\,\frac{\partial u}{\partial x} + v\,\frac{\partial u}{\partial y} - \Omega\,v + g\,\frac{ds}{dx} + \frac{g\;u(u^2+v^2)^{\frac{1}{2}}}{C^2(h+s)} = \frac{W_x}{(h+s)} \qquad (4.5.1.A)$$

$$\frac{\partial v}{\partial t} + u\,\frac{\partial v}{\partial x} + v\,\frac{\partial v}{\partial y} + \Omega\,u + g\,\frac{ds}{dx} + \frac{g\;v(u^2+v^2)}{C^2(h+s)} = \frac{W_y}{(h+s)} \qquad (4.5.1.B)$$

where W_x, W_y are wind stress components.

4.5.2 Integration of the Continuity Equation over Depth

The boundary conditions are at the free water surface (s) and at the bottom (-h):

$$A(s) = \frac{\partial s}{\partial t} + u\,\frac{\partial s}{\partial x} + v\,\frac{\partial s}{\partial y}$$

$$A(-h) = -\,u\,\frac{\partial h}{\partial x} - v\,\frac{\partial h}{\partial y}$$

Integrating 4.3.6.C with respect to the vertical axis gives

$$\frac{\partial s}{\partial t} + \frac{\partial\{(h+s)u\}}{\partial x} + \frac{\partial\{(h+s)v\}}{\partial y} = 0 \qquad (4.5.2.A)$$

If (4.6.2.A) is replaced by a finite difference scheme of step size Δx and Δt then the solution will be known at the set of points

$$S_F = \{(x,t);\ \ x = n_x\ \Delta x,\ x \leq x_o;\ \ t = n_y\ \Delta t,\ t \leq t_o;\ \ n_x,\ n_y\ \varepsilon\ Z^+\}$$

where x_o defines the end of the simulation space field and to the upper time limit. Had 4.6.2.A been solve analytically, the solution will be known at the set of points

$$S_A = \{(x,t),\ x = R_x x_o,\ t = R_t t_o,\ 0 \leq R_x \leq 1;\ \ 0 \leq R_y \leq 1;\ \ R_y\ \varepsilon\ \ \}$$

i.e. S_A is an infinite set and S_F is a subset of S_A, and as Δx, $\Delta t \rightarrow 0$, $S_F \rightarrow S_A$. Also, if U_A is the analytical solution and U_F the finite difference solution, as

$$\Delta x,\ \Delta t \rightarrow 0,\ \ U_F \rightarrow U_A$$

The practice Δx, Δt remains finite and so

$$U_F = U_A + U_D \qquad (4.6.2.B)$$

where U_D represents the difference in the two solutions through the approximation. It is important to investigate U_D, as for any acceptable method it must have three properties:

1. U_D must not be a monotonically increasing function over time.

2. U_D must be capable of estimation and minimisation.

3. U_D must tend to zero as Δx, $\Delta t \to 0$.

To generalise, D_D is the difference approximation operator, operating on U_D, equal to F_D, the forcing functions. 4.6.2.A is now rewritten

$$D_D\ U_D = F_D$$

$$\text{Where } D = \begin{vmatrix} \frac{\partial}{\partial t} & h\frac{\partial}{\partial x} \\ g\frac{\partial}{\partial x} & \frac{\partial}{\partial t} \end{vmatrix}, \quad U = \begin{vmatrix} S \\ U \end{vmatrix}, \quad F = \begin{vmatrix} 0 \\ -\frac{\partial p_a}{\partial x} \end{vmatrix} \tag{4.6.2.C}$$

it is possible to write 4.6.2.D for the analytical solution D U = F (4.6.2.D). The norm is defined as $N[f_1 - f_2]$ and is the maximum difference between functions f_1 and f_2 over the area under consideration. The order of the finite difference approximation can now be defined as 'p' in

$$\begin{aligned} N[D_D\ U_D - DU] &\lesssim q(\Delta x)^p \\ N[F_D - F] &\lesssim r(\Delta x)^p \end{aligned} \tag{4.6.2.E}$$

using 4.6.2.C and D and where q, r are > 0, finite and constant.

It can be shown that similar logic allows an expression for the order of the approximation to be written as[16]:

$$N[U_D - U] \lessapprox s(\Delta x)^p \text{ where s is similar to q,r} \qquad (4.6.2.F)$$

the difference scheme used is stable.

Also, if the properties of U_D are correct, then $U_D \rightarrow U_A$. This can also be established rigidly[16]. The order of an approximation is useful when critically reviewing different schemes, but give no guide to the accuracy of such a method.

4.6.3 Stability of a Difference Scheme

A scheme must be stable in the sense that as $t \rightarrow \infty$, the error remains bounded, and as the mesh is refined, i.e. Δx, $\Delta t \rightarrow 0$ the scheme must tend to the continuous analytical solution. A simple method[17],[18] for investigating stability is to use Fourier expansions of the error element U_D and then calculate the amplification.

Using the lowest order scheme and omitting the forcing element of 4.6.2.A, a difference equation can be written as

$$S_m^{n+1} - S_m^n + \frac{h}{2}\frac{\Delta t}{\Delta x} \cdot \{U_{m+1}^{n+1} - U_{m-1}^{n+1}\} = 0 \qquad (4.6.3.A)$$

$$U_m^{n+1} - U_m^n + \frac{g}{2}\frac{\Delta t}{\Delta x} \quad \{S_{m+1}^{n+1} - S_{m-1}^{n+1}\} = 0 \qquad (4.6.3.B)$$

If $U = \{{U \atop S}\}$, then a vector $\delta \underset{\sim}{U}$ exist at time zero (arbitrary) and in the spatial field $x = 0 \rightarrow x_o$. This vector can be decomposed into a finite Fourier Series:

$$\delta \underset{\sim}{U}(x) = \sum_n \underset{\sim}{A}_n . e^{i\sigma_n x} \quad \text{of } x_o/\Delta x = N \text{ terms, wave number } \sigma_n \tag{4.6.3.C}$$

The system is linear, so only one term need be considered. A_n is time dependent and must have the general form $U_n^* \, e^{i\beta_n t}$ to satisfy the t = 0 Fourier series (4.6.3.C).

So at a grid point (x,t), $\delta U(x,t) = \begin{matrix} U^* \\ S^* \end{matrix} e^{i\beta_t Q i\sigma x}$

If it is assumed that errors are fluctuations superimposed on the true solution, having deducted the real solution should leave only the error perturbation. Rewriting 4.6.3.A-B in Fourier terms and deducting the whole expression including perturbations, gives

$$(e^{i\beta t} - 1)\, S^* + \frac{h}{2} \frac{\Delta t}{\Delta x} \left(e^{i(\beta t+\sigma x)} - e^{-i(\beta t+\sigma x)}\right) U^* = 0 \tag{4.6.3.D}$$

$$(e^{i\beta t} - 1)\, U^* + \frac{g}{2} \frac{\Delta t}{\Delta x} \left(e^{i(\beta t+\sigma x)} - e^{-i(\beta t+\sigma x)}\right) S^* = 0 \tag{4.6.3.E}$$

writing $E = e^{i\beta t}$, as S^* and U^* do not vanish identically, and as 4.3.6.D-E are two homogeneous equations in S^* and U^*, the determinant must vanish, giving a quadratic in E, solving

$$e^{i\beta t} = \{1 \pm i \cdot \left(\frac{\Delta t}{\Delta x}\right) \sqrt{gh}\, \sin(\sigma x)\} / \{1 + \left(\frac{\Delta t}{\Delta x}\right)^2 gh \sin^2(\sigma x)\} < 1 \tag{4.6.3.F}$$

Therefore the errors introduced at time t = 0 will decay without restriction on Δx or Δt and the scheme is said to be unconditionally stable.

These equations are only of interest at grid points. The two series 4.3.6.G can be substituted into 4.6.3.A-B and written as

$$[\underset{\sim}{A}]\ \underset{\sim m}{U}^{n+1} = [\underset{\sim}{B}]\ \underset{\sim m}{U}^{n} \qquad (4.6.3.H)$$

where $\underset{\sim j}{U}^{i} = \begin{vmatrix} S^i_j \\ U^i_j \end{vmatrix}$, $\underset{\sim}{A} = \begin{vmatrix} 1 & i\,\frac{\Delta t}{\Delta x}\,h\,\sin(\sigma\Delta x) \\ i\,\frac{\Delta t}{\Delta x}\,g\,\sin(\sigma\Delta x) & \end{vmatrix}$,

$\underset{\sim}{B} = \begin{vmatrix} 1 & 0 \\ 0 & 1 \end{vmatrix}$, σ is the wave number

Equation 4.3.6.H can be written as $\underset{\sim m}{U}^{n+1} = [\underset{\sim}{P}]\ \underset{\sim m}{U}^{n}$ where $[\underset{\sim}{P}] = [\underset{\sim}{A}]^{-1}.[\underset{\sim}{B}]$ and [P] is the Amplification Matrix[14],[19],[20].

The condition for stability can be expressed in terms of behaviour of the Amplification matrix.

$[P(\Delta t,\sigma)]^n$ must be bounded for $0 < t < t_o$, for wave numbers σ

If R is the spectral radius of P[14],

$$R(\Delta t,\sigma)^n \leq N[P(\Delta t,\sigma)^n] \leq N[P(\Delta t,\sigma)]^n . \qquad (4.6.3.I)$$

The stability condition is now that there exists a K, > 1, such that

$$R(\Delta t,\sigma)^n < K \quad \text{for } 0 < n\Delta t < t_o$$

$$R(\Delta t,\sigma) < K^{1/n} = K^{\Delta t/t_o} \qquad (4.6.3.J)$$

for t in the interval 0 to , $K^{\Delta t/t_o}$ is bounded by $1 + K' \Delta t$. Bearing the definition of spectral radius in mind, the Van Neumann Stability Criteria is written as

$$|\lambda_i| < 1 + O(\Delta t) \quad \text{where } \lambda_i \text{ are eigenvalues of P} \qquad (4.6.3.K)$$

4.6.4 Schemes for Inertia Terms Stability

It can be advantageous to use off centred differences, using a weighting function θ as a measure of the eccentricity [14]. Consider

$$\frac{\partial s}{\partial t} + h \frac{\partial u}{\partial x} = 0 \text{ and } \frac{\partial u}{\partial t} + g \frac{\partial s}{\partial x} + U_o \frac{\partial u}{\partial x} = 0 \qquad (4.6.4.A)$$

where $g \frac{\partial s}{\partial x}$ is small and U_o is the basic mean flow rate and large compared to the fluctuations. This equation can be written as

$$U_m^{n+1} - U_m^n + \frac{1}{2} (\frac{\Delta t}{\Delta x}) U_o \{(1-\theta) [U_{m+2}^{n+1} - U_m^{n+1}] + \theta[U_m^{n+1} - U_{m-2}^{n+1}]\} = 0$$

(4.6.4.B)

Applying the derivation of amplification matrices from 4.6.3, the amplification factor (a|x| matrix) is

$$\lambda = 1/\{1 - \frac{1}{2} (\frac{\Delta t}{\Delta x}) U_o \{(1-2\theta) \cos(2\sigma\Delta x) - (1-2\theta)$$

$$+ i \sin (2\sigma\Delta x)\}\} \qquad (4.6.4.C)$$

Two possibilities have to be considered, $U_o < 0$ and $U_o > 0$. If $U_o < 0$ then the denominator is > 1 if $0 < \theta < \frac{1}{2}$ and so satisfies the necessary and sufficient stability condition (4.3.6.K)

If $U_o > 0$ then $(1-2\theta)$ has to be negative to change the - sign outside to + to make the denominator positive. This occurs for $\frac{1}{2} < \theta < 1$. For $\theta = \frac{1}{2}$ the sufficient condition is met in both limits.

Using this analysis, it is possible to predict instability. On boundaries instabilities have been predicted and found[1]. If economic forces permit, all finite difference schemes can be programmed for off centre differences. These can then be switched in and investigated as required.

4.7 'Computerisation' of the Two Dimensional Wave Equation

4.7.1 Introduction

The computational model of the two dimensional wave equation will now be developed, based on the sections 4.3, 4.5 and 4.6.

Three indices are required to locate a variable in the x/y field and in time $U(i,j,k) = U(i\Delta x,\ j\Delta y,\ k\Delta t)$ is the notation employed.

A simplifying assumption is that $\Delta x = \Delta y = \Delta H$. $(x,y) = (i\Delta x,\ j\Delta y)$ is a grid point in the two dimensional field.

$$i = 0, \pm \tfrac{1}{2}, \pm 1, \pm 3/2$$

$$j = 0, \pm \tfrac{1}{2}, \pm 1, \pm 3/2$$

$$k = 0, \tfrac{1}{2}, 1, 3/2$$

Some averaging operators are defined as A^x, A^y, A^{xy} and two difference operators D^x, D^y

$$\bar{f}^x(i,j) = A^x[f^x] = \tfrac{1}{2}[f(i+\tfrac{1}{2})j) + f(i-\tfrac{1}{2},j)] \quad (4.7.1.A)$$

$$\bar{f}^y(i,j) = A^y[f^y] = \tfrac{1}{2}[f(i,j+\tfrac{1}{2}) + f(i,j-\tfrac{1}{2})] \quad (4.7.1.B)$$

$$\bar{\bar{f}}(i,j) = A^{xy}[f] = \tfrac{1}{4}[f(i-\tfrac{1}{2},j-\tfrac{1}{2}) + f(i-\tfrac{1}{2},j+\tfrac{1}{2}) + f(i+\tfrac{1}{2},j-\tfrac{1}{2}) + f(i+\tfrac{1}{2}, j+\tfrac{1}{2})] \quad (4.7.1.C)$$

$$f_x = D^x[f^x] = [f(i+\tfrac{1}{2},j) - f(i-\tfrac{1}{2},j)] \quad (4.7.1.D)$$

$$f_y = D^y[f^y] = [f(i,j+\tfrac{1}{2}) - f(i,j-\tfrac{1}{2})] \quad (4.7.1.E)$$

4.7.2 Outline of the Solution Scheme

A staggered spatial grid is used (Figure 4.7.2.A) so that u, v; s and h are located at different points. This ensures that the current variable acted on in time is always central[21].

TABLE 4.7.2.A Grid point references of variables

Variable	x grid reference of description	y grid reference of description
s-water level	Integer values of I	Integer values of J
u-x component of velocities	$I + \frac{1}{2}$	J
v-y component of velocities	I	$J + \frac{1}{2}$
H-depth	$I + \frac{1}{2}$	$J + \frac{1}{2}$

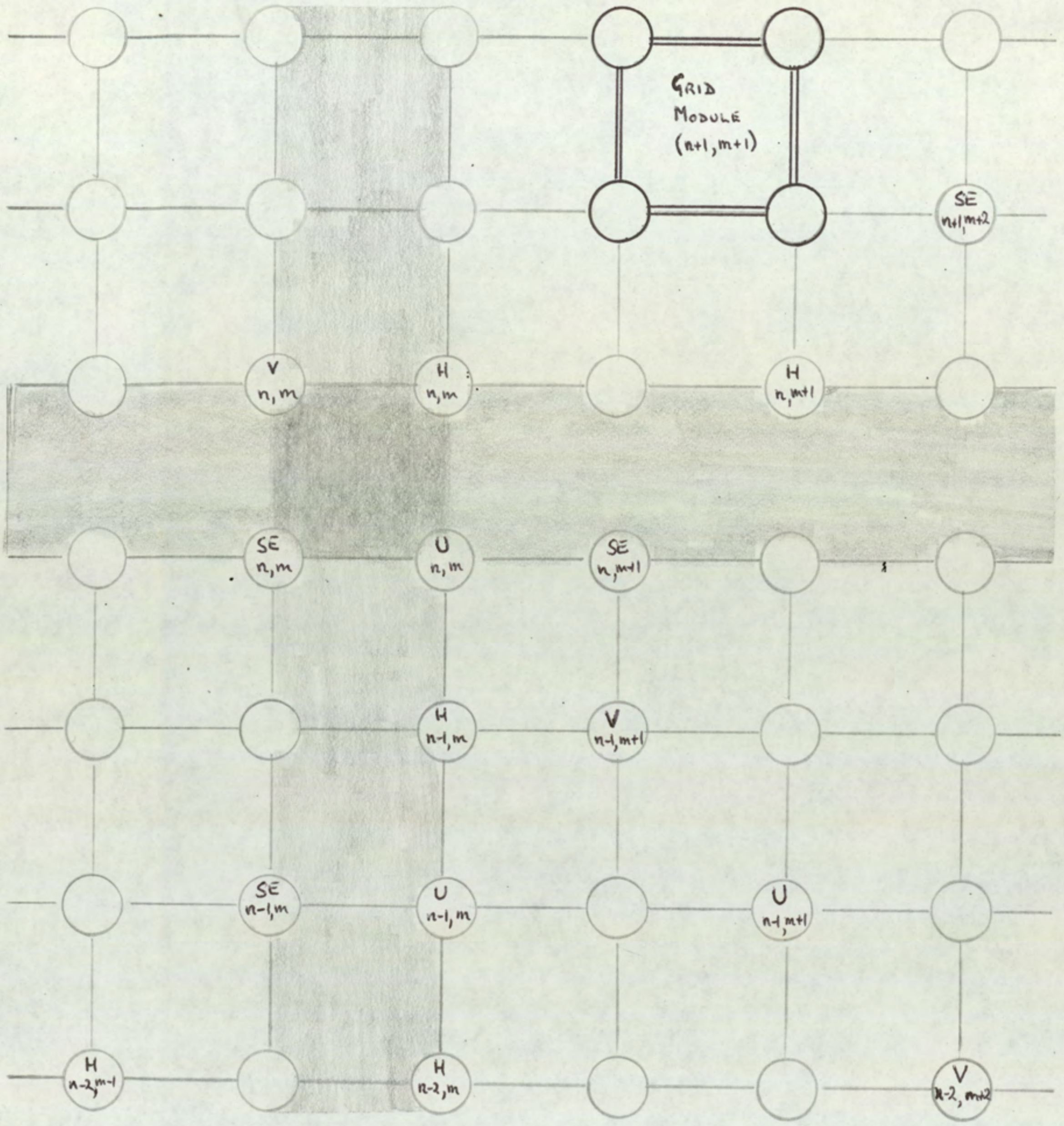


FIGURE 4.7.2.A Staggered Grid used in the Computational Model

A double time step method is employed, i.e. to obtain $t + \Delta t$ from t, firstly $t + \Delta t/2$ is estimated, then this is used to obtain $t + \Delta t$. This allows spatial derivatives and Coriolis forces to be alternately calculated forwards or backwards in time:

$U,V,S(i,j,k+\frac{1}{2})$ are obtained from $U,V,S(i,j,k)$ by a method implicit in S and U and explicit in V, then $U,V,S(i,j,k+1)$ are obtained from $U,V,S(i,j,k+\frac{1}{2})$ by a method implicit in S and V and explicit in U.

4.7.3 The Multioperational Method

Firstly, values of U, V, S are obtained at plus one half time step, then at plus one time step. A minimum of six equations are thus required, U and V are determined from the equations 4.5.1.A and B while S is obtained from 4.5.2.A, the equation of continuity. Derivatives are replaced by finite differences with the exception of the velocity gradients in the convection terms. Wind and Barometric effects are lumped in the function W and Bottom Roughness/Friction in the function F. Superscripts refer to time

$U(i+\frac{1}{2},j,k+\frac{1}{2})$ from S, U, $V(i,j,k)$ explicit in V, implicit in U and S, at $(i+\frac{1}{2},j)$:

$$U^{(k+\frac{1}{2})} = U^{(k)} + \frac{\Delta t}{2} \{\Omega \,\bar{\bar{V}}^{(k)} - U^{k+\frac{1}{2}} \left|\frac{\partial u}{\partial x}\right|^{(k)} - \bar{\bar{V}}^{(k)} \left|\frac{\partial u}{\partial y}\right|^{(k)} - \frac{g}{\Delta H} S_x^{(k+\frac{1}{2})}\} - F(x)^{(k)} - W(x)^{(k+\frac{1}{2})} \qquad (4.7.3.A)$$

$S(i,k,k+\frac{1}{2})$ from $S,U,V(i,j,k)$ explicit in V, implicit in U and S, at (i,j):

$$S^{(k+\frac{1}{2})} = S^{(k)} - \frac{\Delta t}{2\Delta H}\{[(\bar{h}^{y} + \bar{s}^{x})\, u]_{x}^{(k+\frac{1}{2})} - [(\bar{h}^{x} + \bar{s}^{y})v]_{y}^{(k)}\}$$

(4.7.3.B)

$V(i,j+\frac{1}{2},k+\frac{1}{2})$ from $S,U,V(i,j,k)$:

$$V^{(k+\frac{1}{2})} = V^{(k)} - \frac{\Delta t}{2}\{\Omega\, \bar{\bar{U}}^{(k+\frac{1}{2})} - \bar{\bar{U}}^{(k+\frac{1}{2})} \left|\frac{\partial v}{\partial x}\right|^{(k)} - \frac{g}{\Delta H}\, S_{y}^{(k)}\} - F(y)^{(k+\frac{1}{2})} - W(y)^{(k)}$$

(4.7.3.C)

Then, for the second phase from $k+\frac{1}{2}$ to $k+1$

$U(i+\frac{1}{2},j,k+1)$:

$$U^{(k+1)} = U^{(k+\frac{1}{2})} + \frac{\Delta b}{2}\{\Omega\, \bar{\bar{V}}^{(k+1)} - U^{(k+1)} \left|\frac{\partial u}{\partial x}\right|^{(k+\frac{1}{2})} -$$

$$\bar{\bar{V}}^{(k+1)} \left|\frac{\partial u}{\partial y}\right|^{(k+\frac{1}{2})} - \frac{g}{\Delta H}\, S_{x}^{(k+\frac{1}{2})}\} - F(x)^{(k+1)} - W(x)^{k+\frac{1}{2}}$$

(4.7.3.D)

$S(i,j,k+1)$ from continuity equation:

$$S^{(k+1)} = S^{(k+\frac{1}{2})} - \frac{\Delta t}{2\Delta H}\{([\bar{h}^{y} + \bar{s}^{x}]u)_{x}^{(k+\frac{1}{2})} - ([\bar{h}_{x} + \bar{S}^{y}]V)_{y}^{(k+1)}\}$$

(4.7.3.E)

$V(i,j+\frac{1}{2},k+1)$:

$$V^{(k+1)} = V^{(k+\frac{1}{2})} - \frac{\Delta t}{2}\{\Omega\,\bar{\bar{U}}^{(k+\frac{1}{2})} - \bar{\bar{U}}^{(k+\frac{1}{2})}\left|\frac{\partial v}{\partial x}\right|^{k+\frac{1}{2})} - V^{(k+1)}\left|\frac{\partial v}{\partial y}\right|^{k+\frac{1}{2}}$$

$$- \frac{g}{\Delta H} S_y^{(k+1)}\} - F(y)^{(k+\frac{1}{2})} - W(y)^{(k+1)} \qquad (4.7.3.F)$$

The equations 4.7.3.A - 4.7.3.F have now to be solved numerically. To simplify the essential development of a solution scheme, the functions F and W are omitted at this stage. The terms still in differential form are also omitted and developed later.

Consider equations 4.7.3.A and B. Each equation contains three unknowns at time $n+\frac{1}{2}$ in line j. 4.7.3.A can be rewritten as

$$B(i+\tfrac{1}{2},j,k) = -\frac{g}{2}\frac{\Delta t}{\Delta H} S_{i+\frac{1}{2}}^{(k+\frac{1}{2})} + U_{i+\frac{1}{2}}^{(k+\frac{1}{2})} + \frac{g}{2}\frac{\Delta t}{\Delta H} S_{i+1}^{(k+\frac{1}{2})} = U^k + \frac{\Delta t}{2}\Omega\,\bar{\bar{V}}^{(k)}$$

$$(4.7.3.G)$$

and the continuity condition 4.7.3.B is

$$A(i,j,k) = -\frac{\Delta t}{2\Delta H}[(\bar{h}_y + \bar{S}_x)u]_{i-\frac{1}{2}}^{(k+\frac{1}{2})} + S_i^{(k+\frac{1}{2})} + \frac{\Delta t}{2\Delta H}[(\bar{h}_y + \bar{S}_x)u]_{i+\frac{1}{2}}^{(k+\frac{1}{2})} = S^{(k)}$$

$$- \frac{\Delta t}{2\Delta H}[(\bar{h}_x + \bar{S}_y)V]_j^{(k)} \qquad (4.7.3.H)$$

So in 4.7.3.G for each velocity point $U_{i+.}$ there is one equation in 3 unknowns, similarly for S in 4.7.3.H. If there are ℓ water level points on the line j, velocities are known at boundaries, ℓ water levels and $\ell-1$ velocities have to be calculated from $2\ell-1$ expressions.

Now define r_i as follows:

$$r_i = \frac{g}{2}\frac{\Delta t}{\Delta H} = r_{i+1} \quad , \quad r_{i\pm\frac{1}{2}} = \frac{\Delta t}{2\Delta H}(\bar{h}_y + \bar{S}_x)_{i\pm\frac{1}{2}} \qquad (4.7.3.I)$$

For a line j in the y direction of the water field, 4.3.7.G and H can be written in matrix form

$$M.N^{(k+\frac{1}{2})} = \alpha^{(k)} + \beta^{(k+\frac{1}{2})} \qquad (4.7.3.J)$$

assuming $U^{k+\frac{1}{2}}_{s-\frac{1}{2}}$ and $U^{k+\frac{1}{2}}_{e+\frac{1}{2}}$ are <u>known</u> velocities at start and end (i.e. boundaries) of water field. The matrix M is a banded matrix and N,α,β are column vectors

$$M = \begin{vmatrix} 1 & r_{s+\frac{1}{2}} & 0 & 0 & . \\ -r_s & 1 & r_{s+1} & 0 & . \\ 0 & -r_{s+\frac{1}{2}} & 1 & r_{s+3/2} & . \\ 0 & . & . & . & . \\ 0 & . & -r_{e-1} & 1 & r_e \\ 0 & . & . & -r_{e-\frac{1}{2}} & 1 \end{vmatrix}$$

Dimension of $M = \ell \times \ell$
where $\ell = 2\,(e - s + 1)$
e = end boundary index
s = start boundary index

N is a column vector at time $k+\frac{1}{2}$ = $[S_s \; U_{s+\frac{1}{2}} \; S_{s+1} \cdots S_{e-1} \; U_{e-\frac{1}{2}} \; S_e]$

α is a column vector at time k = $[A_s \; B_{s+\frac{1}{2}} \; A_{s+1} \cdots A_{e-1} \; B_{e-\frac{1}{2}} \; A_e]$

β is a column vector at time $k+\frac{1}{2}$ = $[r_{s-\frac{1}{2}} U_{s-\frac{1}{2}} \quad 0 \quad 0 \ldots . \; 0 \quad 0$

$r_{e+.} \; U_{e+\frac{1}{2}}]$ (4.7.3.K)

The matrix N is the solution and one possible method is to find M^{-1} and use the matrix relation

$$N^{(k+\frac{1}{2})} = M^{-1}[\alpha^{(k)}+\beta^{(k+\frac{1}{2})}]$$ (where bracketed superscripts are non arithmetic powers)

However, a band matrix is a special case of a square matrix and so a simpler method for solution similar to that used in the steady state model is employed.

To solve 4.7.3.J the following recursion equations can be used:

$$S_i^{k+\frac{1}{2}} = \pi_i \; U_{i+\frac{1}{2}}^{(k+\frac{1}{2})} + \mu_i \qquad (4.7.3.L)$$

$$U_{i-\frac{1}{2}}^{k+\frac{1}{2}} = \psi_{i-1} \; S_i^{(k+\frac{1}{2})} + \Phi_{i-1} \qquad (4.7.3.M)$$

where values of π, μ, ψ and Φ can be obtained from considerations of the terms involved and 4.7.3.L/M.

$$\pi_i = -r_{i+\frac{1}{2}}/\{1-r_{i-.} \; \psi_{i-1}\} \qquad (4.7.3.N)$$

$$\mu_i = \{A_i^{(k)} + r_{i-\frac{1}{2}} \; \Phi_{i-1}\}/\{1 - r_{i-\frac{1}{2}} \; \psi_{i-1}\} \qquad (4.7.3.O)$$

$$\psi_i = -r_{i+1}/\{1 - r_i \; \pi_i\} \qquad (4.7.3.P)$$

$$\Phi_i = \{B_{i+\frac{1}{2}}^{(k)} + r_i/\mu_i\}/\{1 - r_i \; \pi_i\} \qquad (4.7.3.Q)$$

The values of these recursion coefficients are calculated initially at ℓ and then backwards until s is reached. $U_{e+\frac{1}{2}}^{(k+\frac{1}{2})}$ is a known velocity so π_e, μ_e are the last factors to be calculated. This allows 4.7.3.L to be used to calculate $S_e^{k+\frac{1}{2}}$ as the velocity is known. The result of this allows 4.7.3.M to calculate $U_{e-\frac{1}{2}}^{k+\frac{1}{2}}$ to be calculated, which, on reducing the index can again be used to calculate S_{e-1} from 4.7.3.L etc until the boundary s is reached. U and S for $k+\frac{1}{2}$ are then known. This allows 4.7.3.C to be used to calculate V at $k+\frac{1}{2}$ as only $U^{k+\frac{1}{2}}$ is required for the Coriolis term.

Equations 4.7.3.D and E can be used in an identical manner to solve for $U^{(k+1)}$ and $S^{(k+1)}$ in the time step $k+\frac{1}{2}$ to $k+1$. Then 4.7.3.F is used to determine $V^{(k+1)}$.

All the previous theory was developed for a variable c(i,j,k) with j constant. This j has to be cycled also to provide a complete solution at each time step. An alternative method can be developed initially, varying j for a fixed i. Ideally, both methods are programmed and used alternatively.

The method is quite economic in terms of computer store and runtime as the solution method is one of substitution and there are no iterative sections (see 4.7.4). Further, only two levels of solution have to be stored, at k and $k+\frac{1}{2}$ or $k+\frac{1}{2}$ and k, for each of the three variables.

4.7.4 Non Linear Terms in Continuity Equation

The terms $[(\bar{h}^y + \bar{S}^x)u]_x^{k+\frac{1}{2}}$ in 4.7.3.B and $[(\bar{h}^x + \bar{S}^y)V]_y^{k+1}$ in 4.7.3.E contain non linearities in the context of equations 4.7.3.A-F.

The initial value of $S^{k+\frac{1}{2}}$ calculated from 4.7.3.B uses the non-linear term at time k and not $k+\frac{1}{2}$ as should be the case. The solution of this part can be made iteratively until the values $S^{k+\frac{1}{2}}$ converge, each step approximating the $\bar{S}^x$ at $k+\frac{1}{2}$ better than the previous cycle. This does introduce an undesirable element into the model. Iterative procedures need not converge. In practice it is uneconomical in additional computer time to allow more than two iterations.

4.7.5 Convection/Inertial Terms

These are the terms in 4.7.3.A,C,D,F and are in spatial derivative form. To estimate these, available grid points have to be used. Therefore

$$\left|\frac{\partial u}{\partial x}\right|^{(k)} = \frac{1}{2\Delta H}\{U(i+3/2,j,k) - U(i-\tfrac{1}{2},j,k)\} \qquad (4.7.5.A)$$

$$\left|\frac{\partial u}{\partial y}\right|^{(k)} = \frac{1}{2\Delta H}\{U(i+\tfrac{1}{2},j+1,k) - U(i+\tfrac{1}{2},j-1,k)\} \qquad (4.7.5.B)$$

$$\left|\frac{\partial v}{\partial x}\right|^{(k)} = \frac{1}{2\Delta H}\{V(i,j+3/2,k) - V(i,j-\tfrac{1}{2},k)\} \qquad (4.7.5.C)$$

These terms cannot be taken central in time as the matrix M would be filled out to more than a 3 element band matrix and to minimize error, spatial derivatives are taken at a lower velocity in the higher time levels.

It has been shown that by use of an alternate network by which S is computed at the same location as h, allowing velocity derivatives to be spaced over ΔH[22],[23]. This would double store requirements of the model and also may create oscillations between the two systems. The magnitude of these terms is small and so a simpler approximation is employed. The terms involved in 4.7.3.A, C, D and F are the approximations used.

4.7.6 Bottom Friction Effect Terms

The Bottom Friction Term is approximated using Chezy coefficients assigned at values where S is defined from 4.3.4:

$$F(s)^{(h)} = \frac{g\Delta t}{2} U^{(k)} \left[(U^{(k)})^2 + (\overline{\overline{V}}^{(k)})^2 \right]^{\frac{1}{2}} / \{ [\overline{h}^y + \overline{S}^{x(k)}][\overline{C}^x]^2 \}$$

$$\text{at } (i+\tfrac{1}{2}, j) \qquad (4.7.6.A)$$

$$F(y)^{(k+\frac{1}{2})} = \frac{g\Delta t}{2} V^{(k+\frac{1}{2})} \left[(\overline{\overline{U}}^{(k+\frac{1}{2})})^2 + (V^{(k)})^2 \right]^{\frac{1}{2}} / \{ [\overline{h}^x + \overline{S}^{y(k+\frac{1}{2})}][\overline{C}^y]^2 \}$$

$$\text{at } (i, j+\tfrac{1}{2}) \qquad (4.7.6.B)$$

$$F(x)^{(k+1)} = \frac{g\Delta t}{2} U^{(k+1)} \left[(U^{(k+\frac{1}{2})})^2 + (\overline{\overline{V}}^{(k+1)})^2 \right]^{\frac{1}{2}} / \{ [\overline{h}^y + \overline{S}^{x(k+1)}][\overline{C}^x]^2 \}$$

$$\text{at } (i+\tfrac{1}{2}, k) \qquad (4.7.6.C)$$

$$F(y)^{(k+1)} = \frac{g\Delta t}{2} V^{(k+\frac{1}{2})} \left[(\bar{\bar{U}}^{(k+\frac{1}{2})})^2 + (V^{(k+\frac{1}{2})})^2 \right]^{\frac{1}{2}} / \{[\bar{h}^x + \bar{S}^{y(k+\frac{1}{2})}][\bar{C}^y]^2\}$$

$$\text{at } (i,j+\tfrac{1}{2}) \qquad 4.7.6.D)$$

4.7.7 Wind Forcing Terms

Using the expressions developed in 4.3.5 an expression is obtained for the wind component.

Shear stress $T = C\, \rho_a\, |V|^2$ where C is the drag coefficient

Wing drag component in x direction $WX = T^x\, \rho_\omega^{-1}$,

y direction $WY = T^y\, \rho_\omega^{-1}$.

Terms are $WX/\{\bar{h}^x + \bar{S}^x\}$ and $WY/\{\bar{h}^y + \bar{S}^y\}$

4.7.8 Summary and Final Computational Model

The terms discussed after the development of the basic method can all be included in the recursive factors developed in 4.7.3.N-Q. The method developed for the x-direction is identical to that for the y-direction. The formulae so developed still cannot be translated directly for a high level language as fractional indices are used. All indices could be multiplied by 2 to ensure pure integer indices. This would be wasteful of storage as each lattice would be sparsely determined. It is more efficient to use three different coordinate systems, one for U and V, one for water levels S and one for depths h (Figure 4.7.8.A, compare Figure 4.7.2.A).

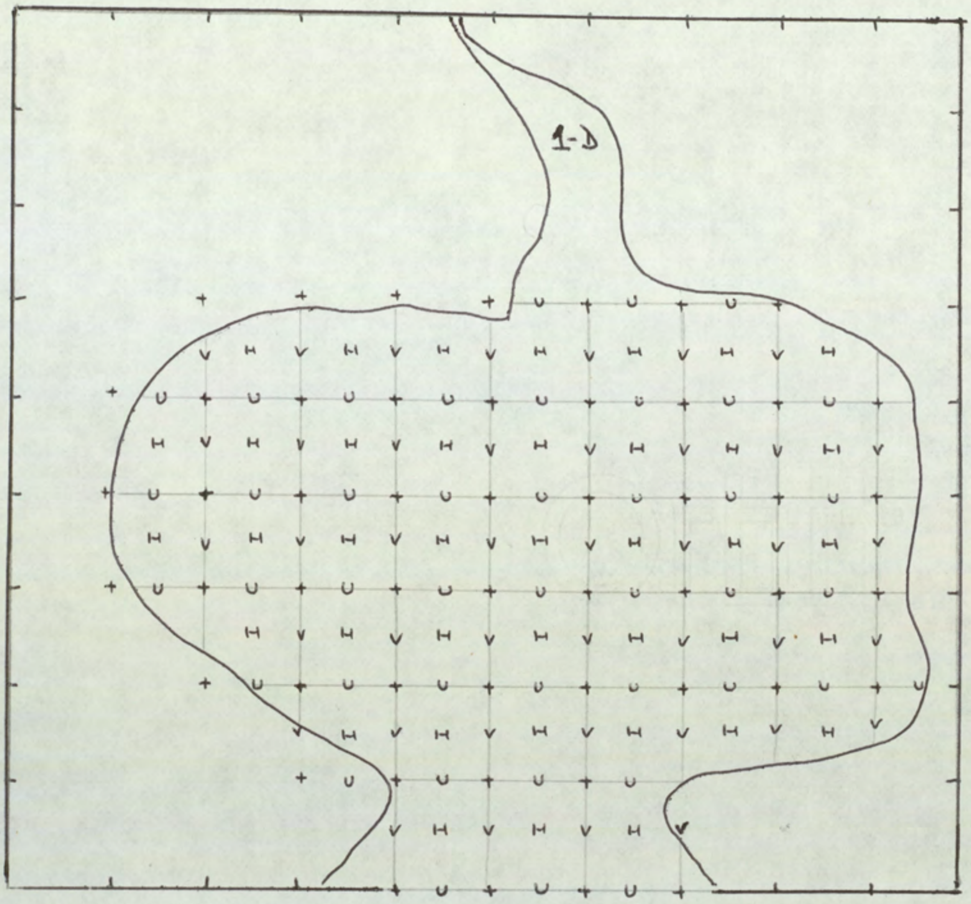


FIGURE 4.7.8.A "Grid Systems for U/V, SE, H in Bay"

The remaining equations of this section used the following notation:

H(N,M) = value of array at col N, row M, invariant with respect to time

SE(N,M,T) = water level at time T

$\frac{\Delta t}{\Delta H}$ = , the ratio of two steps

The recursive equations are, for each row N

$$SE(N,M,T+ t/2) = -P(M).U(N,M,T+\Delta t/2)+Q(M) \qquad (4.7.8.A)$$

$$U(N,T,T+ t/2) = -R(M-1).SE(N,M,T+\Delta t/2)+S(M-1) \qquad (4.7.8.B)$$

where P,R,Q,S are defined by the relationships:

$$P(M) = \frac{\Delta}{2}\ [H(N,M)+H(N-1,M)+SE(N,M,T)+SE(N,M+1,T)]/F(M) \qquad (4.7.8.C)$$

$$Q(M) = \{A(M) + \frac{\Delta}{2}\ [H(N,M-1)+H(N-1,M-1)+SE(N,M-1,T)+SE(N,M,T)]S(M-1)\}/F(M)$$

$$(4.7.8.D)$$

$$F(M) = \{1 + \frac{\Delta}{2}\ [H(N,M-1)+H(N-1,M-1)+SE(N,M-1,T)+SE(N,M,T)]R(M-1)\}$$

$$(4.7.8.E)$$

$$R(M) = \Delta.g/G(M) \qquad (4.7.8.F)$$

$$S(M) = \{B(M)+\Delta.g.Q(M)\}/G(M) \qquad (4.7.8.G)$$

$$G(M) = \{1 + \Delta[g.P(M)+(1-\alpha(N,M))(U(N,M+1,T)-U(N,M,T))+\alpha(N,M)(U(N,M,T)- U(N,M-1,T))]\} \qquad (4.7.8.H)$$

$$A(M) = SE(N,M,T) - \frac{\Delta}{2}\ [\{H(N,M)+H(N,M-1)+SE(N,M,T)+SE(N,M+1,T)\} V(N,M,T)+ \{H(N-1,M-1)+H(N-1,M)+SE(N,M,T)+SE(N-1,M,T)\} V(N-1,M,T)] \qquad (4.7.8.I)$$

$$B(M) = U(N,M,T) + \frac{\Delta t.\Omega}{4} - \frac{\Delta}{4}\ \{(1-\gamma(N,M))(U(N+1,M,T)-U(N,M,T)) +\gamma(N,M)(U(N,M,T)-U(N-1,M,T))\}.\{\frac{1}{4}[V(N,M,T)+V(N,M+1,T) +V(N-1,M+1,T)]\}-8.\Delta t.g.U(N,M,T)\ [U(N,M,T)^2+[\frac{1}{4}\{V(N,M,T) +V(N,M+1,T)+V(N-1,M,T)+V(N-1,M+1,T)\}]^2]^{\frac{1}{2}}/[(SE(N,M,T) +SE(N,M+1mT)+H(N,M)+H(N,M-1)).(C(N,M)+C(N+1,M))^2] \qquad (4.7.8.J)$$

To calculate $V(N,M,T+\Delta t/2)$ explicitly, use

$$UAV(N,M,T+\Delta t/2) = \frac{1}{4}[U(N,M,T+\Delta t/2)+U(N+1,M,T+\Delta t/2)+U(N+1,M+1,T+\Delta t/2) +U(N,M-1,T+\Delta t/2)] \qquad (4.7.8.K)$$

$$V(N,M,T+ t/2) = \{V(N,M,T)-[\Delta t.\Omega+\Delta(1-\delta(N,M))(V(N,M+1,T) -V(N,M,T))-\Delta.\delta(N,M)(V(N,M,T)-V(N,M-1,T))].UAV(N,M,T+\Delta t/2)-\Delta g(SE(N+1,M,T+\Delta t/2) -SE(N,M,T+\Delta t/2))\}/D(M) \tag{4.7.8.L}$$

$$D(M) = 1+(8.\Delta t.g[V(N,M,T)^2+[UAV(N,M,T+\Delta t/2)]^2]^{\frac{1}{2}}/[(SE(N,M,T +\Delta t/2)+SE(N+1,M,T+\Delta t/2)+H(N,M-1)+H(N,M))]. (C(N,M)+C(N+1,M))^2]+[\Delta.(1-\beta(N,M)) (V(N+1,M,T)-V(N,M,T))+\Delta\beta(N,M)(V(N,M,T) +V(N-1,M,T))] \tag{4.7.8.M}$$

where $\alpha(N,M) = \beta(N,M) = \delta(N,M) = \frac{1}{2}$ for central differences.

Equations 4.7.8.A-M are the computational model for the step T to $T+\Delta t/2$. For the step $T+\Delta t/2$ to $T+\Delta t$ the recursion formula 4.7.8.A and B is written

$$SE(N,M,T+\Delta t) = -P2(N).V(N,M,T+\Delta t)+Q2(M) \tag{4.7.8.N}$$

$$V(N-1,M,T+\Delta t) = -R2(N-1).SE(N,M,T+\Delta t)+S2(N-1) \tag{4.7.8.O}$$

where P2, Q2, R2, S2 are similar to P,Q,R,S and defined by

$$P2(N) = \frac{\Delta}{2}[H(N,M)+H(N,M-1)+SE(N,M,T+\Delta t/2)+SE(N+1,M,T+\Delta t/2)]/F2(N) \tag{4.7.8.P}$$

$$Q2(N) = \{A2(N) + \frac{\Delta}{2}[H(N-1,M)+H(N-1,M-1)+SE(N,M,T+\Delta t/2) +SE(N+1,M,T+\Delta t/2)]S(N-1)\}/F2(N) \tag{4.7.8.Q}$$

$$F2(N) = \{1 + \frac{\Delta}{2}\ [H(N-1,M)+H(N-1,M-1)+SE(N,M,T+\Delta t/2) +SE(N+1,M,T+\Delta t/2]R2(N-1)\} \tag{4.7.8.R}$$

$$R2(N) = \Delta.g/G2(N) \tag{4.7.8.S}$$

$$S2(N) = \{B2(N)+\Delta.g.Q2(N)\}/G2(N) \tag{4.7.8.T}$$

$$G2(N) = \{1+\Delta.[g.P2(N)+(1-\beta(N,m))(V(N+1,M,T+\Delta t/2) -V(N,M,T+\Delta t/2))+\beta(N,M)(V(N,M,T+\Delta t/2) -V(N-1,M,T+\Delta t/2))]\} \tag{4.7.8.U}$$

$$A2(N) = SE(N,M,T+\Delta t/2) - \frac{\Delta}{2}\ (H(N,M)+H(N-1,M)+SE(N,M,T+\Delta t/2) +SE(N,M+1,T+\Delta t/2))U(N,M,T+\Delta t/2)+ \frac{\Delta}{2}\ (H(N,M-1) +H(N-1,M-1)+SE(N,M-1,T+\Delta t/2)+SE(N,M,T+\Delta t/2)). U(N,M-1,T+\Delta t/2) \tag{4.7.8.V}$$

$$B2(N) = V(N,M,T+\Delta t/2)-(\Omega.\Delta t+\Delta(1-\delta(N,M))(V(N,M+1,T+\Delta t/2) -V(N,M,T+\Delta t/2)).(UAV(N,M,T+\Delta t/2)))-V(N,M,T+\Delta t/2). [8.\Delta t.g\ V(N,M,T+\Delta t/2)^2+[UAV(N,M,T+\Delta t/2)]^2\}^{\frac{1}{2}}]/\{ SE(N,M,T+\Delta t/2)+SE(N+1,M,T+\Delta t/2)+H(N,M)+H(N,M-1)). (C(N,M)+C(N+1,M))^2 \tag{4.7.8.W}$$

$$UAV(N,M,T+\Delta t/2) = \frac{1}{4}\ U(N,M-1,T+\Delta t/2)+U(N+1,M-1,T+\Delta t/2) +U(N+1),M,T+\Delta t/2)+U(N,M,T+\Delta t/2) \tag{4.7.8.X}$$

To finally calculate $U(N,M,T+\Delta t)$ explicitly, use:

$$U(N,M,T+\Delta t)=\{U(N,M,T+\Delta t/2)+[\Omega.\Delta t-(1-\gamma(N,M)).\Delta.(U(N+1,M,T +\Delta t/2)-U(N,M,T+\Delta t/2))-\gamma(N,M).\Delta.(U(N,M,T+\Delta t/2) -U(N-1,M,T+\Delta t/2)).VAV(N,M,T+\Delta t)]-\Delta.(SE(N,M+1,T+\Delta t) -SE(N,M,T+\ t)\}/\{1+8.\Delta t.g[U(N,M,T+\Delta t/2)^2 +(VAV(N,M,T+\Delta t))^2]/[\{SE(N,M,T+\Delta t)+H(N,M) +SE(N,M+1,T+\Delta t)+H(N-1,M)\}.\{C(N,M)+C(N,M+1)\}^2] +\Delta.[(1-\alpha(N,M))(U(N,M+1,T+\Delta t/2)-U(N,M,T+\Delta t/2)) +\alpha(N,M)(U(N,M,T+\Delta t/2)-U(N,M-1,T+\Delta t/2)]] \tag{4.7.8.Y}$$

$$VAV(N,M,T+ t) = \frac{1}{4}\{V(N,M,T+\Delta t)+V(N,M+1,T+\Delta t)+V(N+1,M,T+\Delta t)$$
$$+V(N-1,M+1,T+\Delta t)\} \qquad (4.7.8.Z)$$

4.7.9 Assumptions and Boundary Conditions

It is assumed that boundaries are closed, i.e. that the boundary remains fixed in space implying that there is always a finite depth of water remaining.

At a closed bound, the velocity perpendicular to the bound can be taken as zero, i.e. there is no flooding over the bound. Also, the maximum deviation of SE from H at this point should be less than H so that $|H-SE|$ remains positive. This is a necessary condition for definition of a fixed boundary, as if SE = H then water depth = 0 and boundary can be moved forward until $|H-SE| > 0$ again.

The boundary passes through locations at which the depth H is given.

At a bay/ocean interface the forcing function provides the boundary condition for SE at those points included in the boundary.

At a river/bay interface the velocity of outflow of the river can be calculated from the output of the river hydrodynamic model. This velocity can be split into components to provide boundary conditions for U and V at each interface point.

Alternatively, for a flood phase, an average of the velocities around the river/bay interface is used to compute inflow volumes to the river phase.

4.7.10 Stability Criteria

Using the criteria developed in 4.6 for stability, the basic equations should give unconditional stability[1]. The additional terms are all stable and it seems reasonable to postulate that a method composed of stable sub methods is also stable. It has been shown that this is not necessarily so[24], and instead the Courant Stability condition is used to provide a guide; although not perhaps the most strict criterion for efficiency:

$$\frac{\Delta H}{\Delta t} > \sqrt{2g\, S_m} \quad \text{where } S_m \text{ is maximum depth expected in any of the field} \tag{4.7.10.A}$$

In the Severn model, ΔH = 2 miles ~ 10,000 ft, S_m = 50 ft, giving a Δt of 180 seconds. Some numerical experiments have been carried out on the complete model in the absence of a comprehensive stability analysis[1].

Early testing in the Severn showed that the approximation of a closed bound above Avonmouth, where the Severn Estuary width < ½ mile, the bore that develops to some extent on all tides in this area is, by nature of the boundary, reflected.

The reflected wave interferes with the hinter waves causing amplification and consequent stability. This was eliminated by using a rounded boundary that would prevent the accentuation of the bore wave.

4.8 Computerisation of the One dimensional River Model

4.8.1 Introduction

The theory applied in section 4.7 could be simplified for one dimension and strictly applied to the one dimensional case. For a flow in a prismatic channel, other methods are available to provide a rapid, stable and accurate method of solution. A characteristic method is used in this computational model[29].

Equation 4.4.2.E is written

$$\frac{\partial u}{\partial t} + u\,\frac{\partial u}{\partial x} + g\,\frac{\partial z}{\partial x} = g\{S_z - S_e\} \qquad (4.8.1.A)$$

where S_z = slope of bed of river, $S_e = \left(\frac{M_F}{1.49}\right)^2 \cdot \frac{u^2}{R^{4/3}}$, M_F = Manning friction factor

Equation 4.4.2.B is $\frac{\partial Q}{\partial x} + b\,\frac{\partial z}{\partial t} = 0$ (4.8.1.B)

where $Q = A(x,t).U(x,t)$ is the flow and b = top width of channel.

The method employs the relationship for a function [25] A(b,c)

$$dA = \frac{\partial A}{\partial b} \,.\, db + \frac{\partial A}{\partial c} \,.\, dc \qquad (4.8.1.C)$$

4.8.2 Computational Model of the One Dimensional Equations

Equation 4.8.1.B is multiplied by a factor β and added to (4.8.1.A)

$$\frac{\partial u}{\partial t} + u\,\frac{\partial u}{\partial x} + g\,\frac{\partial z}{\partial x} + \beta\,\frac{\partial \theta}{\partial x} + \beta\, b\,\frac{\partial z}{\partial t} = g\{S_z - S_e\} \qquad (4.8.2.A)$$

is so chosen as to make u and z perfect differentials. Using (4.8.1.C) gives

$$du = \frac{\partial u}{\partial x}\cdot dx + \frac{\partial u}{\partial t}\cdot dt, \quad dz = \frac{\partial z}{\partial x}\cdot dx + \frac{\partial z}{\partial t}\cdot dt \qquad (4.8.2.B)$$

and $$\frac{du}{dt} = \frac{\partial u}{\partial x}\cdot\frac{dx}{dt} + \frac{\partial u}{\partial t}\,, \qquad \frac{dz}{dt} = \frac{\partial z}{\partial t}\cdot\frac{dx}{dt} + \frac{\partial z}{\partial t} \qquad (4.8.2.C)$$

If β is chosen as $\pm\,\sqrt{g\bar{D}}^{-1}$ where $\bar{D}$ = mean depth = area/ width B, 4.8.2.A becomes

$$\frac{du}{dt} + \beta\,\frac{dz}{dt} = g\{S_z - S_e\} \qquad (4.8.2.D)$$

which is satisfied along the characteristic lines

$$\frac{dx}{dt} = U \pm (gD)^{\frac{1}{2}} \qquad (4.8.2.E)$$

The theory for the above is briefly thus:

Let $z(x,t)$, $u(x,t)$ be the solution of the tidal equations in one dimension, equations 4.8.1.A and B. It is generally possible to derive a unique solution for this kind of equation[26],[27]. $z(x,t)$ defines a surface in the z,x,t space, $u(x,t)$ one in the u,x,t. Consider $x = x(\alpha)$, $t = t(\alpha)$ in the x,t plane. They are curves ψ_x and ψ_t in the surfaces of $u(x,t)$ and $z(x,t)$ and usually exist. For these curves, using 4.8.1.C, the derivatives are

$$\frac{dz}{d\alpha} = \left|\frac{\partial z}{\partial x}\right|_\alpha \cdot \frac{dx}{d\alpha} + \left|\frac{\partial z}{\partial t}\right|_\alpha \cdot \frac{dt}{d\alpha}\ , \quad \frac{du}{d\alpha} = \left|\frac{\partial u}{\partial x}\right|_\alpha \cdot \frac{dx}{d\alpha} + \left|\frac{\partial u}{\partial t}\right|_\alpha \cdot \frac{dt}{d\alpha} \tag{4.8.2.F}$$

Also, let z and u be on the curves $x(\alpha)$ and $t(\alpha)$. Then $z(\alpha)$ represents a curve $\psi_x(\alpha)$ in the space z,x,t and $u(\alpha)$ represents a curve $\psi_t(\alpha)$ in the space u,x,t. The problem is: Can one determine solutions such that ψ_x and ψ_t lie in the integral spaces z,x,t and u,x,t? This is the classic Cauchy Boundary Problem.

For the solution of this, all partial derivatives of u and z have to be known. This can be calcuatted from a determinant of rank 4 of coefficients of $\partial z/\partial x$, $\partial z/\partial t$, $\partial u/\partial x$, $\partial u/\partial t$ to yield

$$\delta = b\ dx^2 - u.b.dt.dx - g\ A\ dt^2 \tag{4.8.2.9}$$

If $\delta = 0$ the equations cannot be used to derive the derivatives. However, when $\delta = 0$ then

$$0 = \left(\frac{dx}{dt}\right)^2 - u\,\frac{dx}{dt} + \frac{gA}{b} \qquad (4.8.2.H)$$

and using the quadratic solution formula, gives $\frac{dx}{dt} = u \pm \sqrt{gA/b}$

(4.8.2.I)

This is the solution of the characteristic lines ψ_x and ψ_t. In deep rivers where the tidal amplitude only creates small values of u then it is permissible to write

$$\frac{dx}{dt} \simeq \pm \sqrt{gA/b} = \pm C_c = \sqrt{gH_c} \qquad (4.8.2.J)$$

4.8.3 Solving the Computational Model

The basic space for solution is the x,t space. A grid is established, figure 4.8.3.A. The x = 0 point is the upstream end of the model, usually beyond all tidal influences. Advancing time is in the positive y plane

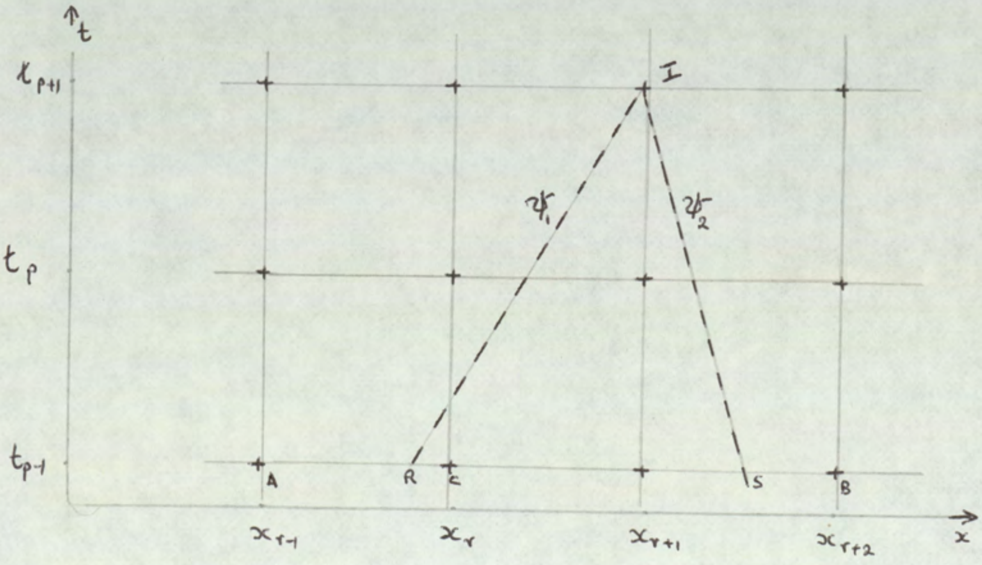


FIGURE 4.8.3.A Coordinate Grid for One Dimensional Computational Model

Suppose the values of $z(x,0)$ and $u(x,0)$ are known, and each are continuous. The x grid is defined by a set of points x_i like A, B in Figure 4.8.3.A, each separated by a distance Δx. Consider the point $x = 0$ where either z is known, or Q which can be converted to a height estimate. Equation 4.8.2.I can be used to calculate the slopes of the two characteristic lines

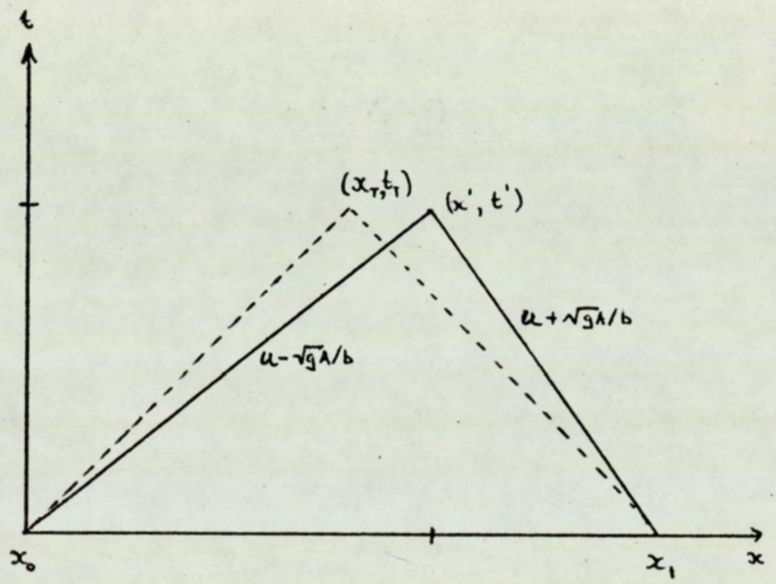


FIGURE 4.8.3.B Method of Characteristics -First Estimates

It can be seen that two segments of origin $x = 0$ and x_1 will intersect at (x', t'). If Δx is sufficiently small, these lines will be sufficiently close to the true characteristic intersection at (x_T, t_T). Once the intersection is located, values of z and u can be calculated.

Writing 4.8.2.D as a difference equation for the graphical construction of Figure 4.8.3.A gives

$$\frac{U_I - U_R}{(t_o+\Delta t)-t_o} + \beta_c \frac{Z_I - Z_R}{(t_o+\Delta t)-t_o} = g(S_z - S_e) \text{ for } \psi_1 \tag{4.8.3.A}$$

$$\frac{U_I - U_s}{\Delta t} - \beta_c \frac{Z_I - Z_s}{\Delta t} = g(S_z - S_e) \text{ for } \psi_2 \tag{4.8.3.B}$$

These terms with subscript I are unknown, so 4.8.3.A and B are rewritten with them as the subjects

$$U_I + \beta_c Z_I = \alpha(\psi_1) = g.\Delta t.\Delta S + \beta_c.Z_R + U_R \tag{4.8.3.C}$$

$$U_I - \beta_c Z_I = \alpha(\psi_2) = g.\Delta t.\Delta S - \beta_c Z_S + U_S \tag{4.8.3.D}$$

The values of U_I and Z_I can be obtained by solving 4.8.3.C and D, to give

$$U_I = \tfrac{1}{2} (\alpha(\psi_1) + \alpha(\psi_2)), \quad Z_I = \frac{g\beta_c}{2} \{\alpha(\psi_1) - \alpha(\psi_2)\} \tag{4.8.3.E}$$

The only problem is the determination of U_R, U_S, Z_R, Z_S as they will not occur on a previous intersection point. However, values of these are known at A, B and C and other points on the grid of x by the previous step. Four linearly interpolated equations are of value here:

$$Z_R = Z_C + (Z_A - Z_C)\ (U_C + C_C)\ (\Delta t/\Delta x) \qquad (4.8.3.F)$$

$$Z_S = Z_C + (Z_C - Z_B)\ (U_C - C_C)\ (\Delta t/\Delta x) \qquad (4.8.3.G)$$

$$U_R = U_C + (U_A - U_C)\ (U_C + C_C)\ (\Delta t/\Delta x) \qquad (4.8.3.H)$$

$$U_S = U_C + (U_C - U_B)\ (U_C - C_C)\ (\Delta t/\Delta x) \qquad (4.8.3.I)$$

to determine the points required for the application of 4.8.3.E. The two limiting assumptions used so far are:

1. Characteristic curves are linear
2. Flow remains subcritical

The movement of a particle can be seen through a series of motions along characteristic lines, the criterion for time steps for a geometric stability is seen to be $(\frac{\Delta t}{\Delta x})\cdot(U+C) < 1$ otherwise equations 4.8.3.F-I are extrapolations and give rise to instability. This condition includes the U-C component also. Within the constraints outlined, the equations are stable and converge as $\Delta x, \Delta t \to 0$[28].

4.8.4 Boundary Conditions

Referring to the geometric interpretation of the method of characteristics, boundaries are those spatial locations where only one characteristic can be calculated, ψ_1 at the downstream end and ψ_2 at upstream end of a section. However, at least one is available and so if either Z or U (or Q) is given at a boundary, the existing characteristic can be used to solve for U or Z respectively.

The theory further assumes prismatic channels. Most natural systems will not meet this requirement. They can usually be segmented into such subsystems though. Information can then be passed from one section to another using Kirchoff type laws. Having accepted this segmentation, a small additional programming effort would yield a model capable of handling intricate networks often found in estuary situations.

4.8.5 Solutions at Nodes

A node is a point in the system where one or more prismatic channels meet another set of one or more prismatic channels.

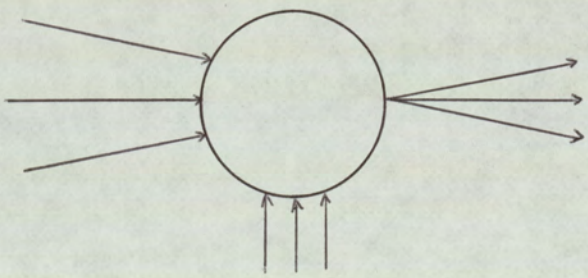


FIGURE 4.8.5.A A General Node

A node can be treated as a spatial point where boundary conditions only are given. Only one characteristic line can be found. It is possible to write the general node continuity equations from this and a simple conservation.

Suppose there are i inflow branches and j outflow branches. For the i inflowing branches the characteristic is given by (4.8.3.C)

$$\alpha_i(\psi_{i,1}) = U_{i,1} + \beta_c z \qquad (4.8.5.B)$$

For the j outflowing branches the characteristic is given by (4.8.3.D)

$$\alpha_j(\psi_{j,2}) = U_{j,1} - \beta_c z \qquad (4.8.5.C)$$

The depth z is common as all water levels are assumed equal. Let $A_{i,1}$ and $A_{j,2}$ be the cross-sectional areas at end of inflowing and start of outflowing segments. The Kirchoff Law can be used: inflow + input = outflow.

$$\sum_i U_{i,1}.A_{i,1} + Q = \sum_j U_{j,2}.A_{j,2} \qquad (4.8.5.D)$$

The unknowns are the sets $\{U_{i,1}\}$, $\{U_{j,2}\}$ and the depth Y, making i + j + 1 unknowns. As there are i equations 4.8.5.B and j of 4.8.5.C, there are i+j+1 equations and the system can be solved.

$$\text{The depth at node } z = \frac{\{\sum_i\{\alpha_i(\psi_{i,1}).A_{i,1}\} - \sum_j\{\alpha_j(\psi_{j,2}).A_{2,i}\}\}+Q}{\{\sum_i \beta_c A_i + \sum_j \beta_c A_j\}} \qquad (4.8.5.E)$$

This additional model for nodes combined with the previous method allows most types of systems to be modelled.

4.9 The Pollutant Transport Phase

4.9.1 Introduction

In sections 4.2 to 4.8 a complete one and two dimensional hydrodynamic model is described. This generates sufficient predictive data to answer the primary question : given a particle introduced into the system at a known point , to what point will it be transported and how long will it remain in the system ? In its broader context, the problem is one of a series of polluting inputs interacting with each other and the environment and also subject to the forces of turbulent flow. Most mathematical models employ the basic diffusive equations. The model to be outlined does not, it seeks to describe mathematically what is essentially an intuitive model,more in the sense of a hydraulic model[34].

4.9.2 Classical Theory of Transport Equations

The theory is included to provide a reference point for the model used. The customary starting point is the'convective diffusion'or'conservation of mass in turbulent flow' equation[30][31]:

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} + v\frac{\partial c}{\partial y} + w\frac{\partial c}{\partial z} = \frac{\partial}{\partial x}(E_x.\frac{\partial c}{\partial x}) + \frac{\partial}{\partial y}(E_y.\frac{\partial c}{\partial y}) + \frac{\partial}{\partial z}(E_z.\frac{\partial c}{\partial z}) + \Delta S \quad (4.9.2.A)$$

where $c(x,y,z,t)$ is the complete concentration distribution of a pollutant or other substance;u,v,w are the x,y,z direction velocity components; E_x,E_y,E_z are the turbulent diffusion coefficients and ΔS is the net rate of addition of the substance to the system.
If ($\bar{c} - c(x,y,z,t)$) is permanently small over the solution space, then $\partial c/\partial t$ is also consistently small and the system can be said to have reached pseudo Steady State. If the magnitude of this term is not negligible non-steady conditions prevail.Few natural system cannot be grouped into some form of steady state description.

The next three terms in 4.9.2.A represent the mass transport by convection. Molecular diffusion processes are small in an environment of turbulent flow. The first three right hand terms of 4.9.2.A represent the diffusive turbulent flow.The scalar diffusion coefficients are less accurate than the use of tensors[32].However,lack of detail to estimate these tensors usually precludes their use. Also the process itself is still the subject of debate and it seems that scalar approximations will continue to be widely used in practical studies[33].

Despite modern methods of numerical and analytical problem solving, it is generally impossible to solve 4.9.2.A either way.The first simplification is the reduction to two dimensions, by assuming a completely vertically mixed system.Writing 4.9.2.A in mass terms and for two dimensions gives

$$\frac{\partial(cz)}{\partial t} + \frac{\partial}{\partial x}(ucz) + \frac{\partial}{\partial y}(vcz) = \frac{\partial}{\partial x}\left(D_x . z\frac{\partial c}{\partial x}\right) + \frac{\partial}{\partial y}\left(D_y . z\frac{\partial c}{\partial y}\right) + R_{add} - R_{abs} \quad (4.9.2.B)$$

where z is depth;u,v the velocities in the x and y direction;D_x and D_y the longitudinal and transverse dispersion coefficients averaged over the point depths,R_{add} and R_{abs} being the rate of mass additions and abstractions . Usually further simplifications have to be made,two frequently used are

a. Averaging over a tidal cylcle or longer period

b. Reducing 4.9.2.B further to a one dimensional system

In one dimension, the simplest forms of coupled equations used for the first order decay BOD/OD mechanism is

$$\frac{\partial B}{\partial t} + u\frac{\partial B}{\partial x} = - K_B . B + \frac{\partial}{\partial x}\left(E\frac{\partial B}{\partial x}\right)$$

$$\frac{\partial D}{\partial t} + u\frac{\partial D}{\partial x} = - K_B . B + K_R(D_s - D) + \frac{\partial}{\partial x}\left(E.\frac{\partial D}{\partial x}\right)$$

where B,D are concentrations of BOD and DO;K_B,K_R are rates of BOD decay and DO re-aeration,D_s is the saturation concentration of DO and E is the lumped coefficient of dispersion.These coupled equations have been widely used[36][37][38] and there is a good choice of implicit or explicit schemes available for their solution[39].

4.9.3 Conceptual Basis of the One Dimensional Phase

The one dimensional flow phase is divided into the same segmentation scheme as for the one dimensional hydrodynamic program. These segments have stationary common interfaces - nodes. Up to three upstream segments can connect directly to one downstream segment. The final lone downstream segment then interfaces with the two dimensional part of the model over a variable area.

Internally ,however, the segment is not split into fixed,pre-determined grid points. The water in a general segment is divided into volumes of water positioned sequentially along the segment axis. The initial segmentation scheme is taken from the hydrodynamic phase. Flow from one segment to another is simulated by taking a volume from the end most element of the outflow segment and creating a new inflow element in the one or more receiving segments. The magnitude of these moves is determined by the predicted velocities from the F1/F2 models and node data. This simulates the convective step. The body of water is generally at its new location.The content of pollutant of this body is now diffused from its new location.

The concentration in a new element is the same as that of the origional element in a previous segment. Elements staying in a segment are kept in that sequence although some merging has to be triggered to keep the number of elements down. New positions can be computed using elemental volume and channel area together with mean velocities. This method reduces numerical dispersion in the convective step and removes channel geometry constraints. In the purely one dimensional phase , the boundary conditions are constant pollutant concentrations at the downstream end of the most seaward segment.If the one dimensional model is connected to a two dimensional phase, the boundary conditions are the time varying predicted concentrations at the bay/river interface. The method has been fi ld tested and appears a wholly satisfactory method[40].

4.9.4 Conceptual Basis of the Two Dimensional Phase

An identical grid to that used in the hydrodynamic program is used with the associated velocity vectors.The motion of a set of marker particles is tracked, with a superimposed diffusive step.The base time segment is a tidal phase, ie a flood or ebb phase whatever their actual duration.

Initially , a grid point is considered at the end of the current tidal phase.The particle is moved backwards in time steps over the two-dimensional grid using the computed velocity vectors from the hydrodynamic program. This defines a theoretical position at the start of the current phase. The nearest grid point is located and the particle assigned to it . This simulates again the pure convective terms of the motion. The diffusive estimate is less satisfactory. To allow diffusion, the original concentration is not that taken for the marker particle , but the nearest neighbour average is used. This implies a diffusive step of order of one grid spacing per phase. Often this restriction is not representative of the actual process in terms of over or understating the physical reality.A greater rate can be achieved by averaging over a greater number of neighbours.This can be simply implemented by extending a search for neighbour points. An alternative way to increase diffusion is to sub-divide the tidal phase into shorter time steps and perform the diffusive step as often as required to estimate the physical processes. . To simulate steps smaller than one grid point, there has to be a finer grid , laid either explicitly or implicitly computed from the coarser grid. Any extension of the simpler nearest neighbour assumption carries serious computing overheads.

During the backward convection of marker particles , several abnormal situations may occur.

The marker could move from the bay into the river, where it is trapped for the whole phase (as the time base is a tidal phase). Alternatively the river could introduce a marker point, in which case the bay concentration is that of the emergent particle at the time of entering the bay.

The marker could cross the ocean/bay interface.If it moves to the ocean the marker is lost and ceases to be of interest as a distinguishable entity.Moving the other way introduces to the bay grid the marker at ocean concentration.

The marker may attempt to cross a bay/land interface.This is the main inaccuracy introduced by the use of coarse grids and numerical dispersion The marker is located to the nearest grid point at the time of the attempted transfer and retained there for the duration of the phase. Any boundary crossing event is further recorded and,if it includes the one dimensional phase,is used for the one dimensional transport calculations.

The convective velocity of each marker particle is inevitably required at points other than on the intersections of the u/v prediction grid. Ideally, a linear or higher order interpolative process should be employed to estimate these cases. For a large grid system with an irregular geometry the exception clauses and interpolation other than simple linear create large computing overheads.

4.9.5 Conceptual Basis of the One Dimensional River System and the Two Dimensional Bay System Matching Program

There are two phases per tidal cycle , and as the transport routines are also particle movers within phases, the two principal routines need to be matched at all times.
During a flood phase, ie a period of net tidal inflow, the events in the bay are relevant to those occuring in the river. There is a time delay between the parameters from the bay being relevant in the river phase, so the entire bay phase is simulated without reference to the one dimensional phase. Concentration parameters crossing into the river are stored at regular intervals (currently every parameter is retained on an hourly basis).When the entire bay phase is completed, the river phase is run with inputs from the bay phase into the single interface segment.
During an ebb phase, the position is reversed.The net flow is out of the river to the bay, and so parameters in the river are of interest to the bay, but only after a transport time delay. The river phase is run in its entirety, and output concentrations stored as required. These are the variable boundary conditions (along with the constant bay/ocean parameters) for the bay phase. To facilitate easier starting up of the scheme, the user has to supply steering information as to the initial phase.This merely determines which program is run first and sets switches without pre-inspection of velocity data.

4.9.6 Diffusion and Pseudo-Diffusion

4.9.6.1 The one dimensional phase uses only the longitudinal dispersion coefficient.This is the parameter D_x in 4.9.2.B and E in equation 4.9.2.C & D. The lumped parameter is sufficient in many cases, especially where tidal amplitude is large so that turbulence ensures genuine vertical and transverse mixing.Also a situation where there are numerous discharges ensures that the effects of an erroneous value,or one in which the mixing assumption does not hold, will be reduced.Literature cites cases where adoption of the assumptions leads to placing of the ultimate DO deficit twice as far from the outfall as was found in practice[41]. In this situation a model of one more dimension is usually applied. Multi-dimensional models allow direction dependant diffusion and so can simulate a'streamed' effluent or similar effect.
In the program, the value of 'EFOR' determines the level of diffusion (ref. App. C).A single value is applied for the model as a whole and can be used as a tuning parameter.If this value is zero, convection only occurs, but molecular diffusion could become an important process.This is the situation in certain areas of many systems where because of geometry the water mass is essentially stationary for a large portion of each phase.Evaluation of the diffusive parameter is usually the most difficult part of any validation phases, and often recourse to predictive formulae is the only practical solution.
Predictive investigations suffer from one of two restrictions : either they are for idealised situations and therefore of doubtful applicability to real-world cases, or they are for specific systems in which case their transfer to another system opens areas of doubt.Both however, are invaluable aids to providing a base for the investigator either for tuning or for local field measurements.

Many studies attempted to find a method for predicting diffusion from more easily determined parameters[42][43]. Turbulence in natural systems compounded by variable geometries are more suited to semi-analytical methods[44][45], and a general predictive form often used is

$$D = K.z.\sqrt{g.z.S_e} \qquad 4.9.6.A$$

where z is the depth, K is a constant, S_e is the energy grade line slope. The values of K are reported widely varying. For vertical and transverse directions, the values of 0.07 and 0.23 have now gained widespread acceptance. The unanimity of these two values is eclipsed by the diverse values accepted for the longitudinal coefficient. Values varying from 10 to 400 have been cited[45]. If a sound estimate is available for a system , then using D for tuning the model provides a useful guide to the accuracy of the rest of the model. The newly obtained value of D should then be qualified when quoted. As the internal grid in this model is moveable, there are advantages to the two step explicit method used: convection and diffusion can be simulated in isolation and tuned independantly, numerical errors are reduced due to lesser elements of inter-grid point estimation.

4.9.6.2 Pseudo-Diffusion

This is a numerical phenomena that occurs in fixed grid systems using the serial method (ie convection then diffusion). In a fixed grid, Δt and Δx are selected , and for steady, non-turbulent flow $\Delta x = U.\ \Delta t$. Consider a unit concentration function such as fig. 4.9.6.A . For the two step method, the distribution in fig. 4.9.6.A will predict a distribution as in fig.4.9.6.B, which is the pure convective step (solid lines)

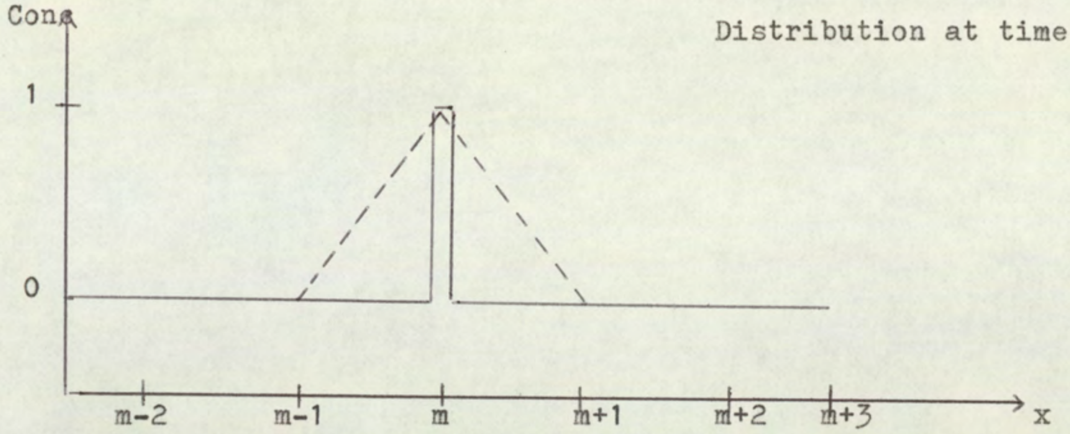


Fig. 4.9.6.A Concentration Distribution at time t .

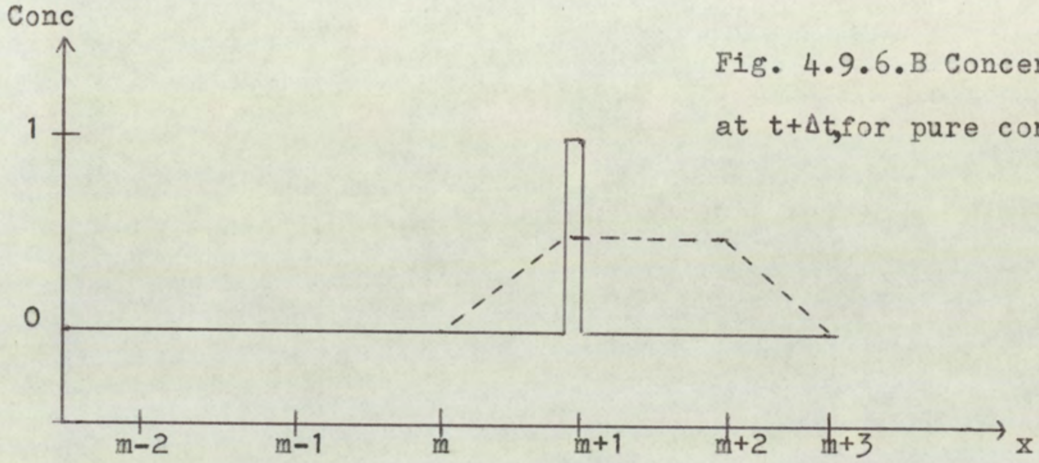


Fig. 4.9.6.B Concentration at t+Δt, for pure convection

However, in physically realistic situations ,$\Delta x \neq U.\Delta t$. Assume that $\Delta x = n.U.\Delta t$ .For a particle to have arrived at a grid point m+1 at a time t+Δt , it would not have origonated from m unless n=1(ideal case) If n>0 the particle was originally at m+(n-1)/n , and if n<0 , at point m+1-(1/n)(all in terms of units of grid points).As the distribution in fig. 4.9.6.A is only known at the grid points, the assumption that intermediate points are estimated by interpolation has to be accepted :

$$C(x) = (1/n)C_m + (1 - 1/n)C_{m+1} \qquad 4.9.6.B$$

and the dotted distributions are implicitly assumed.Consequently, the dotted origonal distribution in fig. 4.9.6.A gives rise to the dotted distribution in fig. 4.9.6.B. This can be seem to retard the pure convective step by seeming to introduce a diffusive effect into that step.The magnitude of the effect depend on the value of n.

A maximum pseudo-diffusive effect occurs for n=2. E_p is the coefficient of pseudo-diffusion and analogous to E in 4.9.2.C.

$$\mathrm{Max}(E_p) = \frac{(\Delta x)^2}{8.\Delta t} \qquad 4.9.6.C$$

However, as the stability of the physical diffusion term E is limited by $(\Delta x)^2/2.\Delta t$, the values of Δx and Δt cannot be so defined to reduce E_p as desired.

At best, $E = (\Delta x)^2/2.\Delta t$, and applying this to 4.9.6.C gives

$$\mathrm{Max}(E_p) = E/4 = \min(\mathrm{Max}(E_p))$$

Therefore, worst possible conditions to give n=2 with best possible choice of E implies diffusion errors of 25%.

In a two step explicit method with a variable grid system, the grid is subdivided dynamically so that n=1 , by a variable Δ x. This ensures that no interpolation is required as the value is known exactly at the grid point.

As the sequel to disposing of pseudo-diffusion , small segments tend to accumulate at end points of the fixed segment system. Occasionally these have to be merged to keep computing requirements within limits and this involves an element of averaging and smearing of concentration elements - as in pseudo-diffusion. This process is not subject to any constraints other than practicality, and in any event, this effect generates a coefficient of $\ll E/4$ in normal systems.

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CHAPTER 5

A Stochastic Model of Water Quality in Estuaries and Streams

CONTENTS

5.0 Introduction

This chapter details the Stochastic Model investigated for the Usk Estuary. The basic principles are detailed in 2.4.7 and 2.4.8 , the reasons for choice appearing in 2.5.3 . It was hoped to provide via the results of the model , some analysis of the diversity of values for the predicted parameters, possibly with a view to imposing percentile defined standards on major effluent inputs to the system.

5.1 BOD/DO as a Discrete Interactive Process

In the model about to be considered , it is implicitly assumed that BOD and DO (or more strictly , virtual units of oxygen deficit) occur and interact in parcels of Δ units of concentration . This allows levels of occurrence of the two components to be classed into states and thus given appropiate state numbers for use in a probablistic context.

$$m_1 = \text{concBOD} / \Delta \qquad m_2 = \text{conc DO} / \Delta$$

This parameter Δ is measureable within a system.BOD can only decay in parcels of Δ and OD can only be created in similar parcels, as the decay /depletion on a normal reaction path , is a 1 : 1 process.

5.2 Assumptions of the Model

Deterministic processes are a subset of all processes. All processes are stochastic in nature , and as such, the deterministic process is an idealized system. For the formulation of this model, the various processes of the Dobbins' Model are reformed from being considered as deterministic to stochastic.

5.2.1 Consider a small time step δt . Consider the base unit of BOD /OD changes as Δ . This restricts any reaction to integer multiples of the base unit Δ in terms of quantities. The actual change may occur via any combination of the processes previously formulated. All such transformations are considered to be statistically independant events. Only first order changes are considered important within the time span δt. This assumes a predominance of first order rate kinetics for all the relevant chemical and biological reactions.
The probability of a transformation of Δ or one of its multiples is directly proportional to the magnitude of the time slice δt. This implies that a small δt is required as the assumed linearity is oftne not present over larger δt.

5.2.2 The major processes to be stochastized are outlined in 2.4.7 to 2.4.8.

5.3 <u>The Random Walk</u>

5.3.1 A random walk is a cumulative series of individual Bernoulli trials which[1],[2], when considered in toto will reproduce a stochastic process with an overall general effect , for example - diffusion.

5.3.2 An estuary is considered to be a field where a procession of random walks occur in small time steps. As the time step is decreased , in the limit $\delta t \dashrightarrow 0$ the process observed Brownian motion or Wiener Process becomes evident[3].

5.3.3 Consider any time step δt, and one unit of BOD. This unit can move upstream, downstream, remain or be degraded. The remaining probability is near zero. The relevant probabilities for the other processes are defined by:

$$r(x,t), s(x,t), q(x,t)$$

with the constraint of : $r + s + t = 1$ (5.3.3.a)

The probability q is the sum of all degradative processes. For each unit lost through the q process , a virtual unit of oxygen deficiency (OD) is created. This unit of OD carries on with the virtual random walk until it is 'neutralized by a 'real' DO unit from the re-aeration process , or until it is abstracted from the system.

5.4 <u>The Segmented Estuary in terms of Probabilities - BOD</u>

5.4.1 Consider a segmented estuary of N segments. Let p(m,n) be the probability that a 'particle' of Δ BOD is in segment m (or a point $m\delta x$

where δx is the segment length) after n Bernoulli Trials, or a random walk of n steps. To be at this point $m\delta x$ at time $n\delta t$, the particle must, at the time $(n-1)\delta t$ have been either at $(m-1)\delta x$ and moved positively, or at $(m+1)\delta x$ and moved negatively. So

$$p(m,n) = p(m-1,n-1)r(x,t) + p(m+1,n-1)s(x,t) \tag{5.4.1.A}$$

for $m,n \in \mathbb{Z}^+ \quad -\infty < m < +\infty \quad 0 \leqslant n < \infty$

For consistency with the physical reality of Brownian Motion, define the following terms :

$$\text{Diffusion Coefficient } E = \frac{1}{2} \cdot \frac{(\delta x)^2}{\delta t} \tag{5.4.1.B}$$

$$r(x,t) = \frac{1}{2}\left[(1-K.\delta t) + U(t).\frac{\delta t}{\delta x} \right] \tag{5.4.1.C}$$

$$s(x,t) = \frac{1}{2}\left[(1-K.\delta t) - U(t).\frac{\delta t}{\delta x} \right] \tag{5.4.1.D}$$

$$q(t) = K.\delta t \tag{5.4.1.E}$$

Where $U(t)$ is the compound velocity function. K is the rate sum of all the degradative processes. Equation 5.4.1.A is now written as

$$p(m,n) = \frac{1}{2}.p(m-1,n-1)\left[(1-K.\delta t)+U(t)\frac{\delta t}{\delta x}\right] + \frac{1}{2}.p(m+1,n-1)\left[(1-K.\delta t)-U(t)\frac{\delta t}{\delta x}\right] + K.\delta t \tag{5.4.1.F}$$

5.4.2 Boundary Conditions

The unit under consideration entered the field at a time t_0 (where $t_o = n_o.\delta t$)

Boundary conditions for the system are :

$p(0,n_o) = 1$ ie at point of entry to the system

$p(m,n_o) = 0$ for $m \neq 0$

$p(m,n) = 0$ if $n < n_o$

and where $n_o = t_o / \delta t$

5.5 The segmented Estuary in terms of probabilities - The Oxygen Deficit.

5.5.1 Absorbing one unit of BOD generates one unit of oxygen deficit (OD) through the 1 to 1 stoichiometry of the reaction. This unit is reabsorbed by re-aeration in a similar manner to the BOD decay through bacteria. If $\Upsilon(m,n)$ is the probability of a unit of OD at a point $m\delta x$ at a time $n\delta t$, then analogous logic to section 5.4.1 leads to

$$\Upsilon(m,n) = \Upsilon(m-1,n-1)r'(x,t) + \Upsilon(m+1,n-1)s'(x,t) + p(m,n).K_d.\delta t \quad (5.5.1.A)$$

The extra term is the creation of OD through BOD decay. K_d is the de-oxygenation rate constant (or of BOD decay).

The corresponding transition probabilities to 5.4.1.B-D are :

$$r'(x,t) = \frac{1}{2}\left[(1-K_r\delta t) + U(t).\frac{\delta t}{\delta x} \right] \quad (5.5.1.B)$$

$$s'(x,t) = \frac{1}{2}\left[(1-K_r\delta t) - U(t).\frac{\delta t}{\delta x} \right. \quad (5.5.1.C)$$

$$q'(t) = K_r.\delta t \quad (5.5.1.D)$$

5.5.2. Boundary Conditions

The initial condition is the absence of any OD due to the absence of BOD for decay to produce the OD , so

$$\Upsilon(m,n) = 0 \quad \text{for } \forall m \text{ if } n < n_o$$

5.6 The limit of the probabilistic expressions

As $\delta t \rightarrow 0$ the probabilities p and Υ approach a continuous density distribution B and D respectively. Re-writing 5.4.1.A and 5.5.1.A :

$$B(x,t+\delta t) = B(x-\delta x,t)r(t) + B(x+\delta x,t)s(t) \qquad (5.6.A)$$

$$D(x,t+\delta t) = D(x-\delta x,t)r'(t) + D(x+\delta x,t)s'(t) + B(x,t)K_d\delta t \qquad (5.6.B)$$

These expressions are expanded in a Taylor series about the point (x,t).

Then incorporating 5.4.1.B gives

$$\frac{\partial B(x,t)}{\partial t} = E\frac{\partial^2 B}{\partial x^2} - U(t)\frac{\partial B}{\partial x} - K.B(x,t) \qquad (5.6.C)$$

$$\frac{\partial D(x,t)}{\partial t} = E\frac{\partial^2 D}{\partial x^2} - U(t)\frac{\partial D}{\partial x} - K_r.D(x,t) + K_b.B(x,t) \qquad (5.6.D)$$

where K_b is the rate of BOD decay through oxidation processes.

5.7 Analytical Results

Some trials have been conducted using 5.6.C and 5.6.D to simulate a pollutant situation [4]. The broad results were that the random fluctuations of input loadings are of marginal significance in the mean levels of BOD/OD in a large diffuse system. The fluctuations do however, markedly influence the nature of the departure from the means. The compuational model is now considered to consist of two phases , one to estimate the mean distribution, and the second phase to estimate the departure from the means.

5.8 The Difference Equations for BOD/OD in an Estuary

5.8.1 The problem is the solution of the set of equations 5.4.1.A and 5.5.1.A

$$p(m,n+1) = p(m-1,n)r((m-1)\delta x, n\delta t) + p(m+1,n)s((m+1)\delta x,n\delta t) \qquad (5.8.1.A)$$

for the range $-\infty < x < +\infty$, $0 \leq t < \infty$

and

$$\Upsilon(m,n+1) = \Upsilon(m-1,n)r'((m-1)\delta x,n\delta t)+\Upsilon(m+1,n)s'((m+1)\delta x,n\delta t) \qquad (5.8.1.B)$$

$$+ p(m,n)K_{d(x,t)} \cdot \delta t$$

Now 5.8.1.A-B could be solved iteratively using the associated expressions 5.4.1.B-5.4.1.E , 5.5.1.B-5.5.1.D . However, this solution would have to be effected for each particle in isolation , then convoluted to produce a probability density function (pdf).

5.8.2 It should be remembered that $\Upsilon(m,n)$ is infact $\Upsilon(m, n|n_o)$, i.e. a probability of a unit introduced only at $n_o.\delta t$. Different introductory times $n_o.\delta t$ will generally affect the final generated pdf. However, as all particles are identical , there is no need to label individual ones , so using this explicit scheme over determines the system.

5.8.3 Prior to analytical consideration, the difference equations 5.8.1.A-B must be considered in the limit of $\delta x \rightarrow 0$. Writing the function B for the limit of p and D for the limit of Υ , where B and D are the respective pdf's :

$$B(x,t+\delta t) = B(x-\delta x,t|t_o)r(x-\delta x) + B(x+\delta x,t|t_o)s(x+\delta x,t) \qquad (5.8.3.A)$$

$$D(x,t+\delta t|t_o) = D(x-\delta x,t|t_o)r'(x-\delta x,t)+D(x+\delta x,t|t_o)s'(x+\delta x,t) + B(x,t|t_o)\,K_d(x,t).\delta t \qquad (5.8.3.B)$$

Consider the term $b(x,t+\delta t)$. Using a Taylor series expansion about the point (x,t) gives

$$B(x,t) + \delta t\ .\frac{\partial f}{\partial t} + (\delta t)^2.\frac{\partial^2 f}{\partial t^2} + \ldots. \qquad (5.8.3.C)$$

Expanding each term in 5.8.3.A in a Taylor series up to the point whwre terms are of the order δt^2 yields

$$\left[B(x,t) + \frac{\partial f}{\partial t}.\delta t + \frac{\partial^2 f}{\partial t^2} . (\delta t)^2 + \dots \right] = \left[B(x,t) - \delta x.\frac{\partial f}{\partial x} + \frac{(\delta x)^2}{2}.\frac{\partial^2 f}{\partial x^2} + ..\right]$$

$$* \left[r(x,t) - \delta x.\frac{\partial r}{\partial x} + \frac{(\delta x)^2}{2}.\frac{\partial^2 r}{\partial x^2} + \dots\right]+\left[B(x,t) + \delta x.\frac{\partial f}{\partial x} + \frac{(\delta x)^2}{2}.\frac{\partial^2 f}{\partial x^2} + \dots\right] * \left[\right.$$

$$\left. s(x,t) + \frac{\partial s}{\partial x}.\delta x + \frac{(\delta x)^2}{2}.\frac{\partial^2 s}{\partial x^2} + \dots \right] \qquad (5.8.3.D)$$

For $B(x,t)$ write B, multiplying out, grouping like terms,dividing by δt,and neglecting terms of order 2 and upwards , the above is reduced to :

$$\frac{\partial B}{\partial t} + \frac{\delta t}{2}.\frac{\partial^2 b}{\partial t^2} = B \left[\frac{(r+s-1)}{\delta t} + \frac{\delta x}{\delta t} \left(\frac{\partial s}{\partial x} - \frac{\partial r}{\partial x} \right) + \frac{(\delta x)^2}{2.\delta t}\left[\frac{\partial^2 r}{\partial x^2} + \frac{\partial^2 s}{\partial x^2} \right] \right] +$$

$$+ \frac{\partial B}{\partial t}\left[\frac{\partial x}{\partial t}(r-s) + \frac{(\delta x)^2}{2\ \delta t}\left[\frac{\partial r}{\partial x} + \frac{\partial s}{\partial x}\right]\right] + \frac{\partial^2 B}{\partial x^2}\left[\frac{(\delta x)^2}{2\delta t} (r+s) \right] \qquad (5.8.3.E)$$

Now the expression 5.4.1.B is used to replace $(\delta x)^2/\delta t$, 5.4.1.C-D for $r+s$ and $(r+s-1)/\delta t$ and derivatives of r and s , e.g.

$$\frac{\partial^2 r}{\partial x^2} + \frac{\partial^2 s}{\partial x^2} = - \frac{\partial^2 K}{\partial x^2} . \delta t \qquad (5.8.3.F)$$

Substituting the various expressions for r and s into 5.8.3.E gives

$$\frac{\partial B}{\partial t} + \frac{\delta t}{2}.\frac{\partial^2 B}{\partial t^2} = B\left[-K-\frac{\partial U}{\partial x} - E\frac{\partial^2 K}{\partial x^2}.\delta t\right]+\frac{\partial B}{\partial x}\left[-U-2E.\frac{\partial K}{\partial x}.\delta t\right]+\frac{\partial^2 B}{\partial x^2}\left[E(1-K\delta t)\right] \qquad (5.8.3.G)$$

This , in the limit $\delta x,\delta t \to 0$ yields , for a well behaved function :

$$\frac{\partial B}{\partial t} = -KB - B\frac{\partial U}{\partial x} + E\frac{\partial^2 B}{\partial x^2} \qquad (5.8.3.H)$$

This is the Fokker-Planck equation for a continuous diffusion process with a non conservative substance. Identical application to the OD phase leads to the expression

$$\frac{\partial D}{\partial t} = -K_r D - D\frac{\partial U}{\partial x} - U\partial D + E\frac{\partial^2 D}{\partial x^2} + BK_d \qquad (5.8.3.I)$$

5.9 Interpretation of the Probability Density Functions (pdf's)

5.9.1. Equations 5.8.3.H-I are the pdf's for one particle of BOD or OD introduced at the point x_o , being found at the point x after an elapsed time since introduction of t .

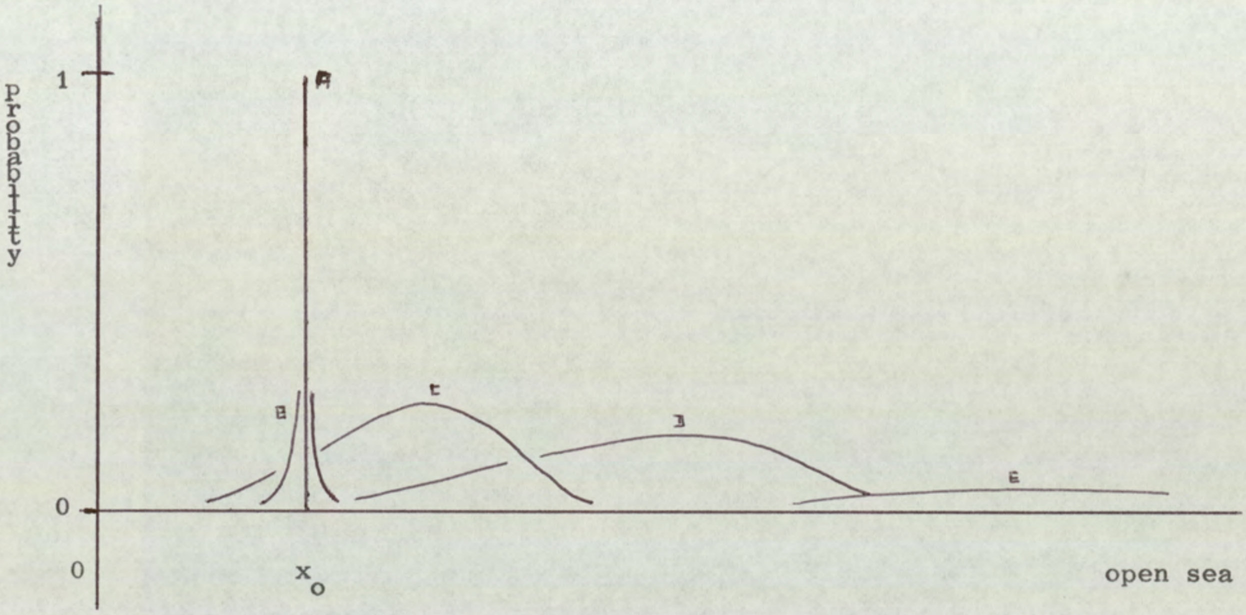

Figure 5.9.1.A PDF's of BOD at various elapsed times

Curve	Interpretation
a	At the instant of discharge , t=0 (on high water,say)
b	Soon after discharge , at typically t< 2 hours
c	Some time after discharge , 2< t < 10 hours typically
d	Considerable time after discharge , 1 day < t < 30 days typically
e	Long enough after discharge for all effects to be removed,in most estuaries t>30 days.

Note : Times given for a typical estuary of 50 km length , with steady flow and 20 days retention for a headwater discharge.

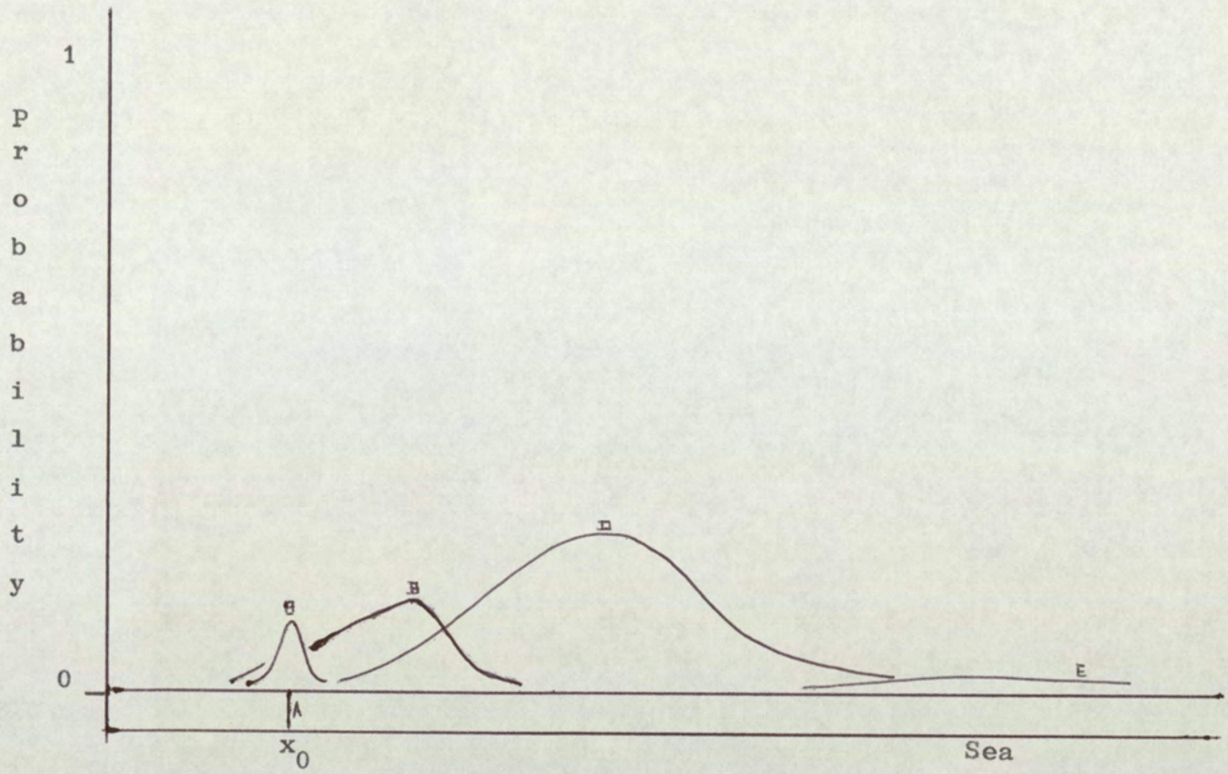

Figure 5.9.1.B The Probability Density Function for O.D. at similar Elapsed Times

Fig. 5.9.1.A shows that at the time of discharge, the probability function has a value , and up to small elapsed times before appreciable transformation occurs, behaves much like the Dirac δ function (curve A and B).

$$\text{i.e.} \int_{-\infty}^{+\infty} \delta(f).dx \quad = 1$$

Eventually tidal distortion, fresh water input variability will skew the distribution , and the decay to produce O.D. will steadily reduce the area under the BOD curve (C and D) . After a long period relative to the tidal retention of the system has elapsed , the effect of the input is lost altogether and so the PDF tends to zero (curve E).

Fig. 5.9.1.B shows the curve of the coupled OD PDF. Assume at the initial discharge point , there is no OD in the system , and the discharge itself introduces no direct OD. Initially then the PDF is zero, until the BOD begins to be utilized. The PDF slowy rises to a peak at a time when the rate of change of BOD has reached a maximum . The OD curve is also modified by the varied

re-aeration process , which is independant of the process of BOD decay.

5.9.2. The term -KB is the loss of BOD due to decay through oxygen consuming and other processes .

The term $-K_R D$ is the rate of re-aeration term. The larger the oxygen deficit, the more prominent the re-aeration factor becomes. If, for some inputs under certain conditions, the system becomes supersaturated , the re-aeration process reverses until a zero deficit results.

The term $+K_D B$ is the rate of biological utilization of BOD to create OD units

5.9.3 The term $E\frac{\partial^2 B}{\partial x^2}$ and $E\frac{\partial^2 D}{\partial x^2}$ represnt the net difference in rates of diffusion between the segment 'slices' δx . There is an inherent assumption here; that is diffusion for real BOD and virtual OD particles are identical . Infact the actual differences in molecular diffusivities are insignificant.

It is of interest to simulate a situation with E=0 , then with E at a measured value. The additional 'smearing' of a pollutant provides a valuable source of dilution for meeting effluent standards in rivers and estuaries.

5.9.4 The term $-U\frac{\partial B}{\partial x}$ and $-U\frac{\partial D}{\partial x}$ represent the inputs due to bulked (ie fresh and tidal) flow . The differenc between input and output of a slice is the term ∂ /∂x.

5.9.5. The terms $B\frac{\partial U}{\partial x}$ and $D\frac{\partial U}{\partial x}$ allow for the compressability of the medium. This term is sufficiently small to be neglected in the case where the medium is water.

5.9.6. All the previous calculations have shown to yield is a probability density function for one particle of BOD in isolation. The problem of aggregating many particle systems and converting the resultant PDF into meaningful terms remains.

5.10 Continuous Sources

5.10.1 If the time interval between sucessive BOD units entering the system , δt , tends to zero , the limit is an approximation to a continuous source.The total effect can then be obtained by convoluting the individual effects.

Let $A_K(m,n)$ be the probability that there are K BOD units at $m\delta x, n\delta t$, and $B_K(m,n)$ similar for OD units.

Then , for a continuous source , S , for BOD using equation 5.8.1.A :

$$T_A(m,n,s) = \sum_{k=0} A_K(m,n).S^k \qquad (5.10.1.A)$$

$$\simeq \prod_{i=0}^{n} \left[(1-p(m,n|i))S^0+p(m,n|i)S^1\right]$$

$$= \prod_{i=0}^{n} \left[(1-p(m,n|i))+p(m,n|i)S\right] \qquad (5.10.1.B)$$

Similarly, for OD using 5.8.1.B :

$$T_B(m,n,s) = \prod_{i=0}^{n} \left[(1-\Upsilon(m,n|i))+\Upsilon(m,n|i)S\right] \qquad (5.10.1.C)$$

5.10.2 For calculation of the probability of any one particular state , (m_A,n_A) BOD and (m_B,n_B) OD , the equations 5.10.1.B and 5.10.1.C are fully expanded and the product coefficient

$$T_A(m_A,n_A).T_B(m_B,n_B)$$

is the probability of that particular state occurring from the set on input conditions of the system.This product-coefficient will have a maximum value, and the state this reflects is the expected state of the system for the input conditions.

5.10.2 For a function F(x) , the expected value is defined as [5]

$$E(g[x]) = \int_{-\infty}^{+\infty} g(x)F(x).dx \qquad (5.10.2.A)$$

(g(x) is a function of x so that the integral exists over the range of integration). In the context of 5.10.1.B-C , E is the mean value

$$E_A(k) = T_A'(s)\Big|_{s=1} \quad \& \quad E_B(k) = T_B'(s)\Big|_{s=1}$$

(as probabilites for s=0 are zero). Consequently , for 5.10.1.B

$$\frac{\partial T_A}{\partial s} = \sum_{l=0}^{n} p(m,n|l) . \prod_{\substack{i=0\\ i\neq l}} [(1-p(m,n|i))+p(m,n|i)s] \qquad (5.10.2.B)$$

$$\text{and } \frac{\partial T_A}{\partial s}\Big|_{s=1} = \sum_{l=0}^{n} p(m,n|l) \qquad (5.10.2.C)$$

$$\text{Similarly , } \frac{\partial T_B}{\partial s}\Big|_{s=1} = \sum_{l=0}^{n} \lambda(m,n|l) \qquad (5.10.2.D)$$

5.10.3 For a function F(x), the variance is defined as $E(x^2)$ .In the context of 5.10.1.B-C this is equal to

$T_A''(s)\Big|_{s=1} + T_A'(s)\Big|_{s=1} - (T'(s))^2\Big|_{s=1}$ with an identical expression for T_B.

To calculate $T_A''(s)$, the expression 5.10.2.B is differentiated to yield

$$\frac{\partial^2 T_A}{\partial s^2} = \sum_{l=0}^{n} p(m,n|l) . \sum_{\substack{i=0\\ i\neq l}}^{n} p(m,n|i) . \prod_{\substack{j=0\\ j\neq l\\ j\neq i}}^{n} [(1-p(m,n|j)) + p(m,n|j)s] \qquad (5.10.3.A)$$

$$\text{and so } \frac{\partial^2 T_A}{\partial s^2}\Bigg|_{s=1} = 0 \text{ , then } E_A(T_A^2) = \sum_{l=0}^{n} p(m,n|l) - \sum_{l=0}^{n} [p(m,n|l)]^2 \qquad (5.10.3.B)$$

and similarly for the OD expression.

5.10.4. <u>Summarising</u>

5.10.4 Summarising

$$\text{Mean(BOD State)} = \sum_{l=0}^{n} p(m,n|l) \qquad (5.10.4.A)$$

$$\text{Variance(BOD State)} = \sum_{l=0}^{n} p(m,n|l) - \sum_{l=0}^{n} [p(m,n|l)]^2 \qquad (5.10.4.B)$$

$$\text{Mean(OD State)} = \sum_{l=0}^{n} \Upsilon(m,n|l) \qquad (5.10.4.C)$$

$$\text{Variance(OD State)} = \sum_{l=0}^{n} \Upsilon(m,n|l) - \sum_{l=0}^{n} [\Upsilon(m\ n|l)]^2 \qquad (5.10.4.D)$$

Allowing $\delta t, \delta x$ to decrease so that $1/2 \,.(\delta x)^2/\delta t = E(x,t)$ and the process becomes continuous, n becomes large due to the very large number of states available for the system to occupy. As $\sum \Upsilon$ and $\sum p$ are each bounded by unity , the actual individual values of each probability state must decrease. Both sums also have the loer bound of zero. Therefore, in the variance terms above (5.10.4.B and D) the individual values of each state squared decrease very quickly and so

$$\text{Variance(BOD/OD State)} \xrightarrow[\text{limit } \delta x, \delta t \to 0]{} \text{Mean (BOD/OD State)} \qquad (5.10.4.E)$$

5.10.5 These independant Bernoulli Trials with non uniform probabilities can be shown to converge to a Poisson Distribution[6]. This has to be further generalised in that , on the assumption of particle independance, as individual distributions are Poisson , and convoluting Poisson distributions gives a further Poisson distribution. The total concentraion distributions are Poisson with expectancy equal to the sum of the individual expectancies.

$$\sum_{\substack{\text{all states}\\ \text{all particles}}} . \Pi \text{ (Poisson Probabilities)} = \text{Total PDF (Poisson)}$$

$$\text{Mean Total Concentration} = \frac{\sum \text{(Individual Mean Concs)}}{\text{number of states}}$$

$$= \Delta . \frac{\sum \text{(Individual Mean Probabilities)}}{\text{number of states}}$$

This holds for any pollutant that behaves independantly and has a finite probability distribution. The only singularity may arise at a point of discharge, at a high rate of discharge (curve A, fig. 5.9.1.A).

The concentration variance is similarly the product of the state probability variance and Δ^2 .

Consequently , a knowledge of the mean and Δ yields the complete anticipated solution to a general estuarine pollutant dispersion problem.

5.11 Solutions for the Mean

5.11.1 Equations 5.8.3.H and 5.8.3.I are rewritten in the concentrations sense with the notes of 5.9 incorporated :

$$\frac{\partial B}{\partial t} = E_B(x,t).\frac{\partial^2 B}{\partial x^2} - U(x,t).\frac{\partial B}{\partial x} - K(x,t)B + L(x,t) + F(x,t) \qquad (5.11.1.A)$$

$$\frac{\partial D}{\partial t} = E_D(x,t).\frac{\partial^2 D}{\partial x^2} - U(x,t).\frac{\partial D}{\partial x} - K_R(x,t)D + K_B B + D_B(x,t) - P_S(x,t) \qquad (5.11.1.B)$$

with the following key

$E_B(x,t)$	Diffusion Coefficient for BOD
$E_D(x,t)$	Diffusion Coefficient for OD
$U(x,t)$	Total Velocity
$K(x,t)$	Total rate of BOD decay
$L(x,t)$	Land run off rate of addition of BOD load
$F(x,t)$	Point Sources of BOD , discharges
$K_R(x,t)$	Rate of re-aeration
$K_B(x,t)$	Rate of oxidation of BOD to produce OD
$D_B(x,t)$	Rate of increase of OD due to benthal demand
$P_s(x,t)$	Rate of decrease of OD due to photosynthetic production

5.11.2 For most purposes , $E_B = E_D$. The term $U(x,t)$ can be estimated in different ways. It can be measured in the field , aprroximated in the form

$$U(x,t)= U_F(x) + U_T(x) . \sin(\alpha t)$$

where U_F= fresh water velocity, U_T = maximum tidal velocity, α is tidal frequency.

The term $K(x,t)$ includes the term $K_B(x,t)$ and the rate of biological utilization through non-oxidation pathways. These are great over-simplifications of the pathways the reactions are thought to take[7,8].

5.11.3 Briefly , some outline reactions thought to predomiate are :

$$BOD(carbonaeceous) \xrightarrow{O_2} CO_2 + (OD)$$

$$BOD(nitrogenous\ ) \xrightarrow{O_2} NO_2 + (OD)$$

$$NO_2 + O^{\bullet} \longrightarrow NO_3 + (OD)$$

$$(OD) \longrightarrow \text{Waste products}$$

$$CO_2 + NO_3 \longrightarrow \text{Biomass}$$

$$(Biomass) \xrightarrow{light} -(OD)$$

$$(Biomass) \xrightarrow{death} BOD(carb.) + BOD(nitr.)$$

5.11.4 The equations 5.11.1.A-B have to be solved simultaneously as they are coupled . They are simultaneous first order differential equations[9]:

$$\frac{dB}{dt} = f_1(B,D,t) \text{ and } \frac{dD}{dt} = f_2(B,D,t)$$

As such they can be solved using a wide variety of methods. Ideally , a method requiring only knowledge of the current step , with insensitivity to start up errors, has some error estimation inbuilt and requires little storage and minimal computation time is sought.

5.11.5 A fourth order Runge Kutta method modified by Merson is best suited to meet the previously listed ideal requirements. The main necessary condition is that the functions are expandable as a Taylor series (see 5.6) For a step size h from (x_o,y_o) to (x_o+h,y_o) , calculate the following :

$$c_o = \frac{h}{3} f(x_o,y_o) \qquad (5.11.5.A)$$

$$c_1 = \frac{h}{3} f(x_o+h/3,y_o+c_o) \qquad (5.11.5.B)$$

$$c_2 = \frac{h}{3} f(x_o+h/3,y_o+c_1/2+c_o/2) \qquad (5.11.5.C)$$

$$c_3 = \frac{h}{3} f(x_o+h/2,y_o+9c_2/8+3c_o/8) \qquad (5.11.5.D)$$

$$c_4 = \frac{h}{3} f(x_o+h\ \ ,y_o+6c_3-9c_2/2+3c_o/2) \qquad (5.11.5.E)$$

Then , finally

$$y_1 = y_o + 1/2\ (\ c_o + 4c_3 + c_4) + O(h^5) \qquad (5.11.5.F)$$

An estimation of the truncation error E is given by [9]

$$0.2c_o + 0.8c_3 - 0.9c_2 - 0.1c_4 \qquad (5.11.5.G)$$

Then , if $E > mE_A$ where m is a multiple of E_A , the allowed error , the step size must be reduced. Alternatively , if $E < E_A/m$, the step size is altered to a multiple of h to reduce computational effort . So if the alterations in step size reduce the computational effort by only 20% , the above method becomes efficient despite the additonal evaluation of the function .

5.12 Computational Considerations for Expected Value Computation

The computational aspects of the stochastic model can be considered in 3 sections:

1. Input of parameters and segmentation of the system
2. Stepwise time progressive computation of expected values of BOD/OD
3. Occasional reporting of values,presentation of results and controlling program efficiency.

Details of the coded procedures are given in Appendix D. Two principal versions are available . The first uses the functional representation of velocity, the second one has a link routine to abstract estimated velocity data from the data bank accumulated from simulations of the Fischer Model F1 or F2.

The program is arithmetically bound , that is to say , a large proportion of the entire run/mill time is used by the arithmetic unit in the large number of function evaluations that are required. This was a prohibitive feature in the Usk Estuary where the large tidal prism with a high tidal velocity result in small time steps and consequently uneconomic running costs.

5.13 Predicting Δ - The Stochastic Coefficient

5.13.1 Doubt still surrounds the exact nature of Δ[10], although it is now accepted to be a physical factor and not limited to the purely discrete interpretation often employed [11,12]. If one restricts changes to multiples of Δ then the ability to handle low order and high order changes of state is lost. However, having shown that the process is a continuous stochastic one ,and that Δ enters as a scale factor in the variance only, the stochastic coefficient can be defined as

> "The constant of proportionality of a change of mean concentration reflected in the variance."

5.13.2 No restriction on Δ has been placed, as to minimum levels of change involved.Because of the coupling of the process and their inherent first order kinetic common factor , it is reasonable to assume that

$$\Delta_{BOD} \cong \Delta_{DO} = \Delta_{OD}$$

However, it was found that there is an order of magnitude difference between the two coefficients [13]. This demonstrates that Δ is a function of the process itself , not only of its stochasticity.

5.13.3 Practically , Δ must be determined for each site from field data using

$$\Delta.(\text{mean}) = (\text{variance})$$

Care must be taken when using field data for an estimation of Δ . No other major parameters should be in a state of flux , and data included should exclude diurnal effects. Dependance on turbulence,temperature and initial conditions has been demonstrated[12]. Furthermore , only data outside the anaerobic region can be included. The chemistry of the model postulated fails below 5% saturation of dissolved oxygen.

5.13.4 Multiple expressions can be calculated using regressive techniques.These allow Δ to be expressed as a function of major state variables. For the Ohio for example , it was found that

$$\Delta(OD) = -0.2 + 0.0823(\text{sample station no.}) + 0.002(\text{time of day})$$

$$+0.0078(\text{temperature}) - 0.0081(\text{BOD}) - 0.0088(\text{staion no.})^2$$
$$+0.00004(\text{time of day})^2$$

However, applying Student -t tests to the coefficients revealed that only two terms were significant , the .0823 and .0088 . This is a direct consequence of the restriction placed on the inclusion of data under 5.13.3 . If all data were included , the above expression would have all significant coefficients [15].

5.14 VERIFICATION of the Model

The model has been verified on the Potomac and Delaware Estuaries . All the preceeding theory can be applied equally to river systems , with the added simplification of the velocity representation. Further validation work has been carried out on the Ohio [14,15]. The volume of data required to perform adequate validation was never available in the Usk system. The model was not extensively used during the project because of the high costs of running while connected to a bureau facility. When in-house computer power is available the routines will be regenerated and validated.

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Chapter 6

Estuary Parameters and Sources

Chapter 6

CONTENTS

6.0 Introduction

6.0.1 A necessary step for theory is from conceptual framework to reality and acceptability. Unless a model proposed for a situation can duplicate a known baseline situation, it has no value in the decision making process. Conversly , having duplicated some known condition, extensions of established constraints are not guaranteed to produce equally valid results. However , they will be more thoughtful and of considerable use provided the underlying assumptions of the prediction process are impressed on the use of those predictions. Few models have all the data they require, most have considerably less than scientific approaches require. Scarcity of data undermines the confidence of validation and this must be reflected when considering the applicability of any projections.

6.0.2 Establishing a base-line data set can cause severe problems. There are undoubted advantages to designing 'saturation' surveys of a complete system for short, intensive periods. Information thus gained is of course highly specific to a set of conditions , and for a relatively short space of time, usually one tidal cycle. There is an advantage in that recorded fluctuations are functions of the main system and there are no long term variants to generate noise on measured signals. This allows inter-relationships and interpretation of data to be more efficient and reliable.

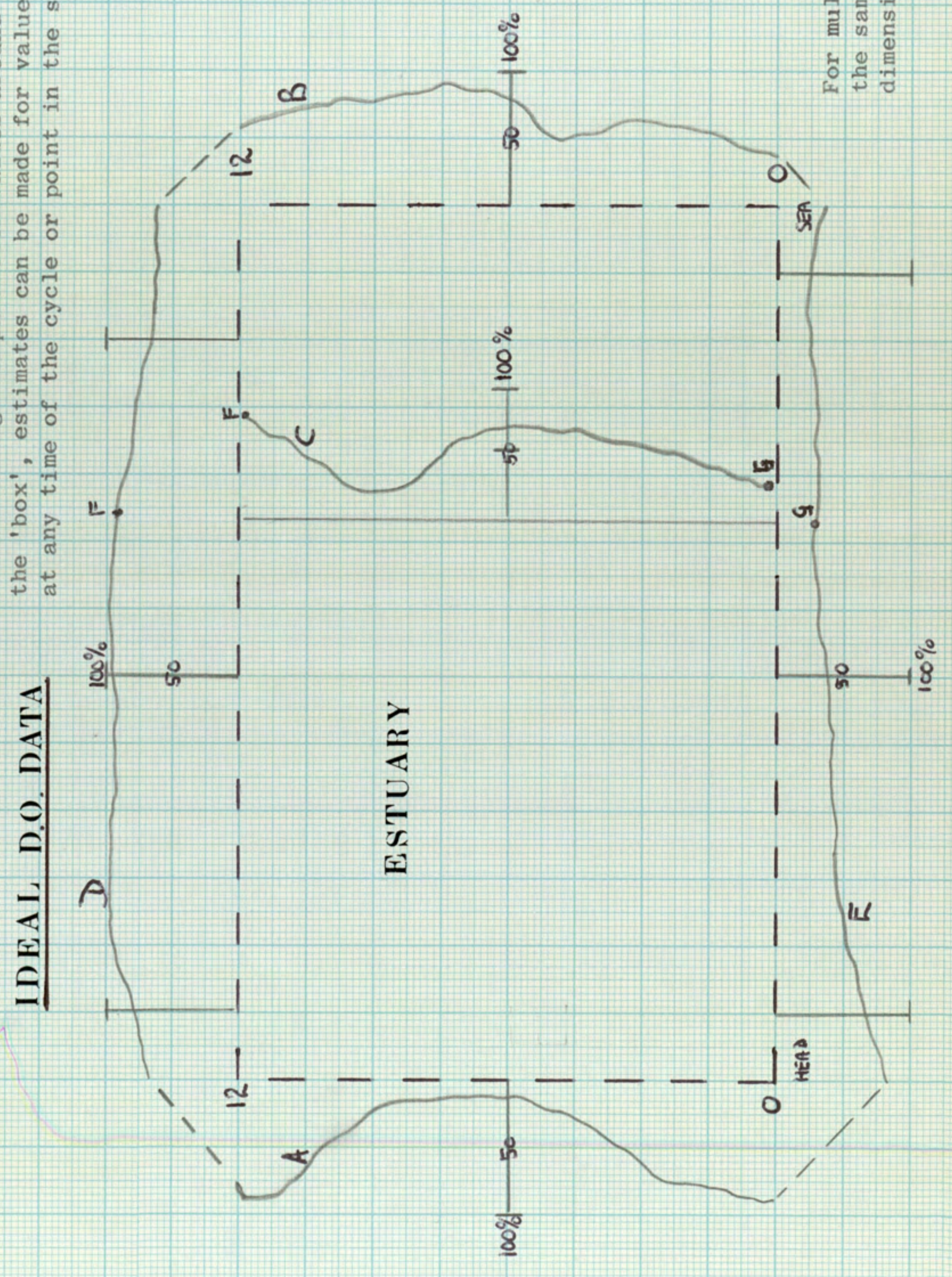
6.0.3 Ideally, for a given parameter, the least data required is that indicated in fig. 6.0.A. For a one dimensional (ie. longitudinal) data collection excercise , consider the estuary as a time-space box of length 1 Estuary and width 1 Tidal Cycle. The least desired data is represented by the lines A,B,D and E . A is the upstream , B the seaward boundary condition. At these points data can usually be gathered with fixed survey stations. At time 0 , nominally Low Water, the line E then represents parameter levels at low water throughout the estuary. Line D represents the high water trace. Lines D and E can be obtained by manning a large number of points for a short period of time or using a highly mobile survey station. A fast boat capable of 20 knots can cover a 17 mile estuary in 90-100 minutes with about 20 samples. Because of the high and low water slack time lag , this can be a very effective method of freezing the system. Conventional boats are often restricted for low water access, because of drawing several feet that are not always available at low water. Two alternatives to be considered are the use of inflatables drawing less than one foot and the use of helicopters. Capital costs of a 12' inflatable with 25HP motor are about £1500 . Helicopters are extremely flexible and can cover a long area. They are restricted in that ground support is often required and hire costs can be about £100 to £300 per hour[4].

Line C represents data acquired by use of an additional fixed survey station.Apart from the additional vertical trace on the data map,each station also provides correlative readings for the tide extreme freezes (at points F and G , fig. 6.0.A).

Figure 6.0.A - Ideal Data Collection

From knowledge of parameter values around the outside the 'box', estimates can be made for values inside at any time of the cycle or point in the system.

For multi-dimensional models, the same concept in 3 and 4 dimensions can be applied .



6.0.4 Instrumentation should be calibrated in the field if possible as particularly Dissolved Oxygen Meters suffer from drift. As a matter of routine , 'Winkler' D.O.'s should also be collected at regular intervals. Braystoke Flowmeters and E.I.L. Salinometers tended to require less maintenance,although Flowmeter Control Units were prone to minor faults. Any B.O.D. Samples should be returned to the Laboratory as soon as possible.Some cooling should be available to prevent initial incubation.

6.0.5 Another matter to be resolved at the planning stage of the project is the required accuracy of the model. The level of accuracy required will affect the depth modelling to be attempted, the type of model used and the validating procedures (which tend to be man-power intensive). The only constraint specified in the project was the acceptability of the Steady State Model. This would be deemed acceptable if the error remained within 10% of base line data. Idealised time dependent model validation required a field effort in excess of that available after re-organisation of the Water Industry.

6.0.6 The criteria of fit is a further subject of choice. What statistical measure should be regarded as acceptable? Usually at least one state variable is left open as a degree of freedom to 'tune' the model to any degree of accuracy desired to an observed data line. Using this method, the term 'accuracy' becomes relatively arbitrary as the tuning parameter becomes less a reflection of the physical measure it relates and more of a 'best-fit' type weighting coefficient. In this study it was decided to use best available estimates for all parameters and accept any lack of fit as faults in the underlying model philosophy.

6.0.7 Fig 6.0.B shows three typical predictive curves to one observed base-line. In terms of total sum of squares of the predictive data to the field data, all three fits are acceptable. Line i) could be considered as a good fit, the general trend is reproduced with an apparent lag in space. If the fit were weighted in terms of volumes or seaward distance then it would be a goodness of fit well below 5%. Without weighting the 10% criteria would be satisfied. Line ii) is on the whole a much closer fit and well within 1% - 2% for the most part. However at the seward boundary divergence is rapid and the whole match loses its attractions. Even unweighted the fitting is pushed beyond the 10% goodness usually required. Rather than attempt to tune this type of deviation, the basic cause should be modified (in this case, seaward boundary conditions are the first parameters to consider for modification). Line iii) has a totally acceptable fit in terms of percentage deviation, at no point do projected and actual curves differ by more than 10%, and is this a situation where the model predicts within the confidence limits of the field data. However, closed examination of the line shows regular projected variation which does not occur in the observed data. This indicates incorrect use of the particular model in terms of sphere of applicability or numerical instabilities. Again the model should be examined rather than using a 'tuner' to amplify or dampen the output to fit.

6.0.8 The following criteria are commonly employed for an observed data set x

(xp - predictive set)

a) $\text{Max} \left| x - xp \right| < E$ Absolute error

b) $\text{Max} \left| \frac{x - xp}{x} \right| < E$ Relative error

c) Combinations of Absolute and Relative error

d) $\sum (x - xp)^2 < E$ Absolute sum of squares

e) $\sum \frac{(x - xp)^2}{x} < E$ Relative sum of squares.

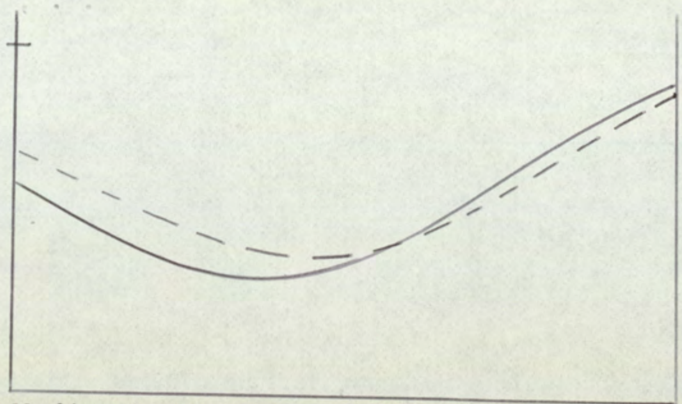
f) $x - n\sigma < xp < x + n\sigma$ Within confidence limits.

All these are for one dimensional goodness of fit. For simultaneous multiple fitting each of the above definitions can be extended to more than one variable. In the multi-dimensional case the minimisation of the multiple least squares is used most frequently. The emergence of Cluster Analysis [6] [12] has recently made available a new set of statistics to judge groupings. As the differing parameters fitted may have differing relative importance, it may be required to weight each parameter prior to calculating the goodness of fit statistic.

For time series prediction, the residuals can be further analysed using the Durbin-Watson Statistic[14] to test for trends in residuals (which theoretically should be randomly distributed). The Steady State Model uses a) for its convergence test, and the Stochastic Model method c).

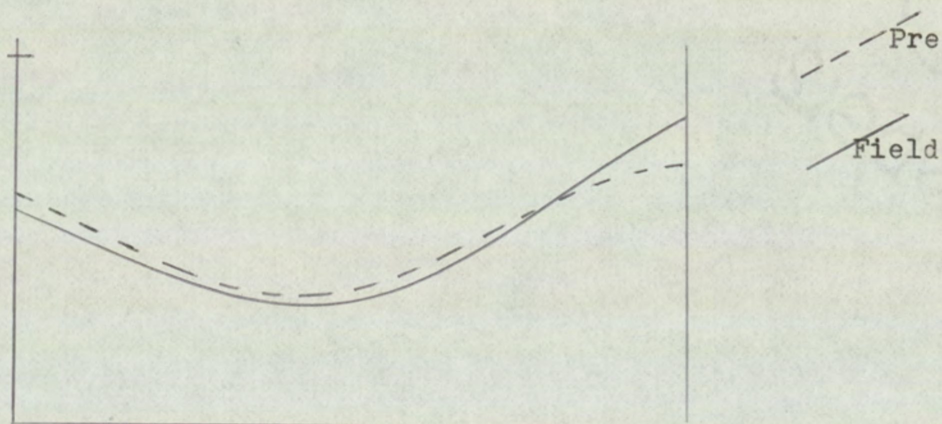
Fig. 6.0.B Fitting a Prediction to a Measured Data Set.

I)



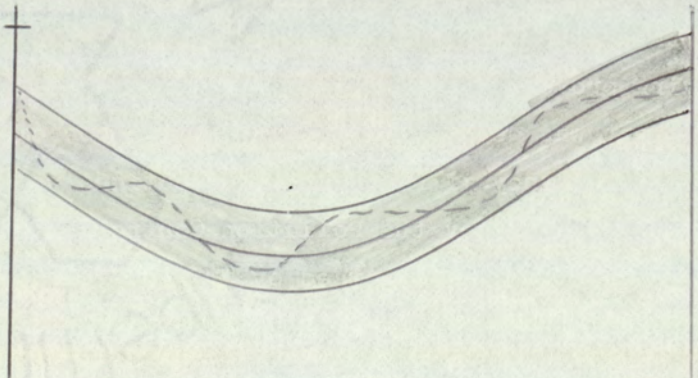
Mediocre fir over whole range Sea

II)



Good fit generally but divergent at area of great interest Sea

III)



Reasonable first fir but is it a valid model to give this type of prediction

6.1 TIDES at NEWPORT

6.1.0 Newport tides tend to have more harmonic distortion than many 'sea' ports because of the situation within the geometry of the Severn Estuary. Periodic Regression and Harmonic Analysis was carried out on the 1970 tidal records from the Newport Outer Cill Survey Station. Fitting one years data to a harmonic function required the use of 15 principal harmonics and so involved 31 terms. This gave a maximum deviation of about 0.3m (1').

6.1.1 A section of the data was examined in greater detail. The period started and ended with identical high spring tides of 14.85m (49'), with a lower spring tide intervening. Because of amplitude coefficients and sign variations, spring tides are less influenced by lower order harmonics, as shown by a comparison of the 4th and 9th order harmonic fits (fig 6.1.A and 6.1.B).

6.1.2 The purpose of the analysis was to assisst in considering what constituted a Steady State period, and also to enable tide profiles to be predicted. Given a starting condition, tide heights could be predicted and assumed that each tide was 12.4 hours after the previous high water. Together with tide-tables[1] (table 6.1.A) , tide height / time sequences of variable length and optional detail can be constructed for input to various models. Modification of such series allowed modelling of storm surges and tidal waves if required.

6.1.3 The analysis fitted the following equation to predicted or observed amplitudes :

$$T_a = T_0 + \sum (a_i \cos [\beta t] + b_i \sin [\beta t]) \qquad (6.1.3.A)$$

Where T_a is total amplitude , T_0 mean amplitude ,t is time and the factor β is = $2\pi i/k$ and in radians .

Analysis of a neap to neap cycle over 120 tides showed even limited series required up to 9 harmonic components. However, some coefficients could be neglected without loss of overall accuracy. To achieve a $\pm$ 2% maximum

Table 6.1.A - Tidal Reduction Tables , Newport

(by kind permission of the Harbour Commissioners)

NEWPORT DOCKS

TABLE SHOWING DEPTHS OF WATER ON SOUTH LOCK CILL EVERY HALF-HOUR FROM LOW WATER TO FOLLOWING LOW WATER.

Low Water	Hours Before High Water.										H.W.	Hours After High Water.										Low Water	Usual Dur'tion of Ebb
	5	4½	4	3½	3	2½	2	1½	1	½		½	1	1½	2	2½	3	3½	4	4½	5		
ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	hr. min.
5 6	—	5 6	12 0	18 0	24 0	29 9	35 3	39 10	45 1	48 0	49	47 9	44 9	40 3	36 0	31 4	27 0	22 6	19 3	16 0	12 9	5 6	7 40
6 0	—	6 0	12 6	18 4	24 0	29 9	35 3	39 8	44 3	47 2	48	47 1	44 1	39 10	35 8	31 0	26 10	22 8	19 3	16 1	12 11	6 0	7 40
6 3	—	8 2	13 9	19 4	24 4	29 9	34 10	39 6	43 8	46 8	47	46 2	43 7	39 5	35 4	30 9	26 9	23 0	19 4	16 3	13 2	6 3	7 35
6 6	—	9 6	15 1	19 9	24 8	29 9	34 7	39 0	42 9	45 4	46	45 3	42 7	38 11	34 10	30 6	26 7	22 10	19 4	16 4	13 5	6 6	7 30
7 0	—	11 4	16 6	20 9	25 0	30 0	34 5	38 6	42 1	44 5	45	44 4	41 7	38 3	34 4	30 3	26 6	22 10	19 5	16 5	13 6	7 0	7 25
7 6	9 9	14 0	18 0	21 9	25 3	30 0	34 3	38 1	41 4	43 5	44	43 4	40 10	37 8	33 10	30 1	26 6	22 10	19 5	16 6	13 11	7 6	7 15
8 0	10 7	14 7	18 5	22 0	25 6	29 9	33 9	37 7	40 7	42 5	43	42 5	40 1	36 6	33 4	29 10	26 3	22 8	19 7	16 8	14 1	8 0	7 00
9 0	11 7	15 3	18 9	22 2	25 9	29 9	33 6	36 10	39 7	41 5	42	41 5	39 5	36 0	32 10	29 5	26 0	22 7	19 8	16 10	14 4	9 0	6 50
10 0	12 3	15 9	19 0	22 5	25 9	29 7	33 3	36 3	38 5	40 5	41	40 5	38 7	35 8	32 4	29 1	25 9	22 7	19 9	17 0	14 8	10 0	6 40
11 0	13 5	16 6	19 6	23 0	26 0	29 6	33 0	35 9	37 10	39 6	40	39 6	37 9	35 2	31 10	28 9	25 6	22 6	19 10	17 4	15 0	11 0	6 35
12 0	14 5	17 2	20 0	23 0	26 2	29 3	32 6	34 11	36 11	38 6	39	38 6	36 9	34 6	31 4	28 5	25 4	22 6	19 11	17 7	15 4	12 0	6 30
13 0	15 5	17 11	20 6	23 3	26 6	29 2	31 11	34 4	36 3	37 6	38	37 6	35 11	33 9	31 0	28 2	25 3	22 5	20 0	17 9	15 8	13 0	6 25
14 0	16 6	18 8	21 0	23 7	26 9	29 0	31 9	33 9	35 5	36 6	37	36 6	35 2	33 3	30 8	28 0	25 3	22 5	20 0	18 0	16 0	14 0	6 20
15 0	17 9	19 9	21 10	24 2	26 9	29 0	31 5	33 4	34 7	35 6	36	35 7	34 4	32 7	30 2	27 9	25 3	22 5	20 1	18 2	16 4	15 0	6 20
16 0	18 6	20 4	22 3	24 6	26 9	29 0	31 0	32 8	33 9	34 8	35	34 8	33 8	32 0	29 8	27 5	25 0	22 5	20 2	18 5	16 9	16 0	6 25
17 0	19 11	21 6	23 3	25 0	27 0	29 0	31 0	32 3	33 1	33 9	34	33 9	33 0	31 6	29 4	27 3	25 0	22 6	20 6	18 11	17 6	17 0	6 30

deviation, the following coefficients and constant is used :

$$T_k = 12.371 + \sum (a_i\cos[\beta t] + b_i\sin[\beta t]) \qquad (6.1.3.B)$$

where :

i	a_i	b_i
1	-0.168	neg.
2	-0.164	0.512
3	-0.363	-.302
4	-1.472	neg.
5	0.393	0.144
6	0.147	-.131
7	neg.	neg.
8	neg.	neg.
9	0.147	neg.

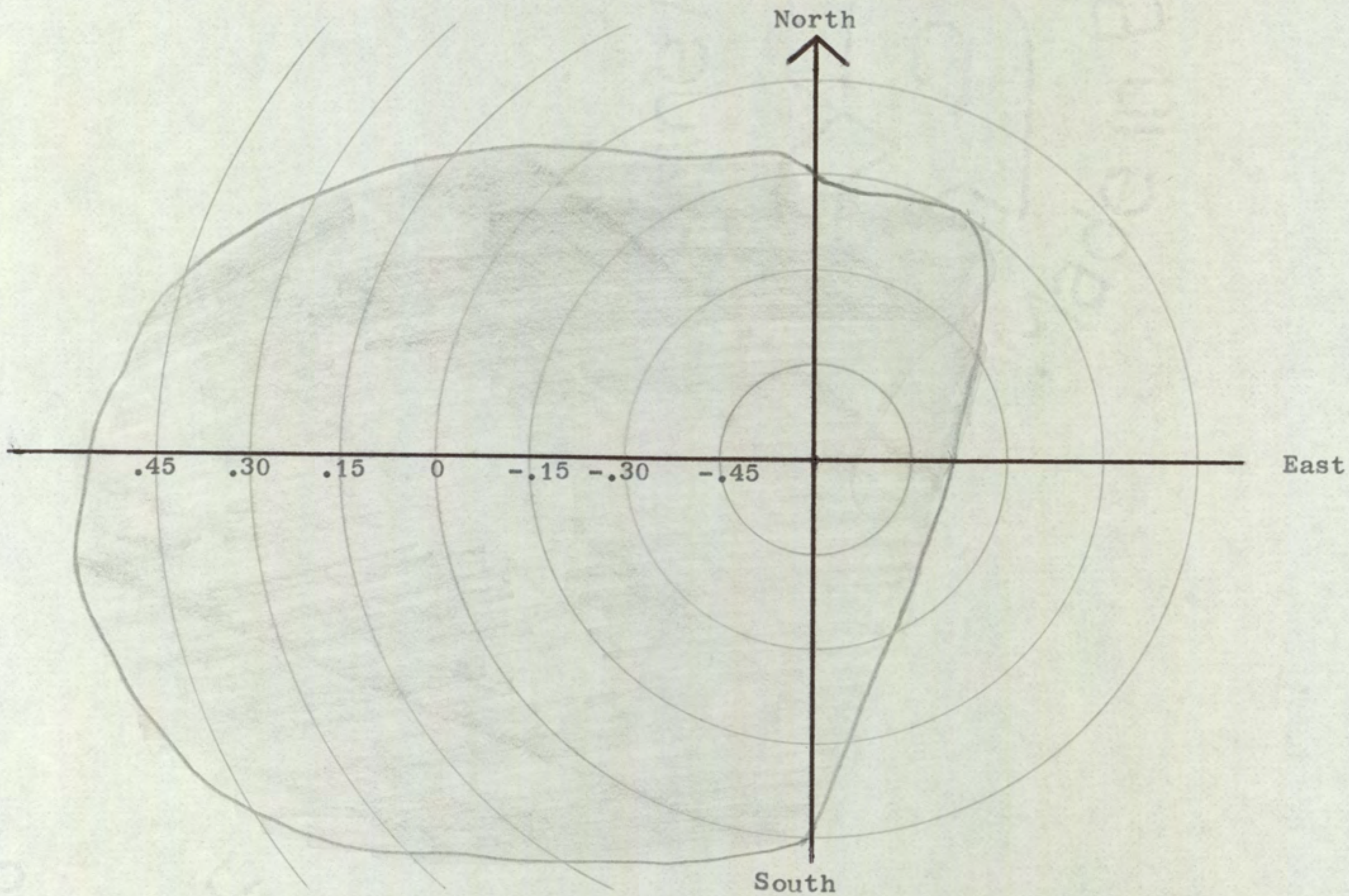
neg. is where coefficient is <0.05

6.1.4 Tides at Barry Port[2] were used mainly for the F2 Model. Recorded data from Avonmouth was supplied by the Docks Engineer of the Bristol Port Harbourmasters Office[3]. The tide gauge at Newport Dock Outer Cill was not well maintained . This collection point could be very useful additional data for any investigation as it strategically placed with respect to the whole Estuary, although its prime function is for shipping. The site is near the dimension interface for models F1/F2.

6.1.5 Wind Effects on tides are significant on high spring tides from the flood prevention point of view.A westerly of strength <5 on the Beaufort scale adds 0.2 to 0.5m to a tide. Winds >5 add 0.7 to 1.0m to a tide. Northely winds tend to reduce by about 0.4m and delay the time also. Fig. 6.1.C shows average tidal effects for varying wind directions for strength 3-4.

6.1.6 Barometric pressure deviations also influence tide heights. Every 5mm over 760mm Hg adds about 0.1m to a spring tide. Every 10mm below the norm reduces a high tide by about 0.1m. Although only approximate guides they are useful in calculating statistically extreme tidal heights.

Figure 6.1.C The Effect of Wind Direction on Predicted Tide Heights at Newport



Levels added or subtracted are in metres on high water slack. Any effect on low water levels in obscured by fresh water inflow levels.

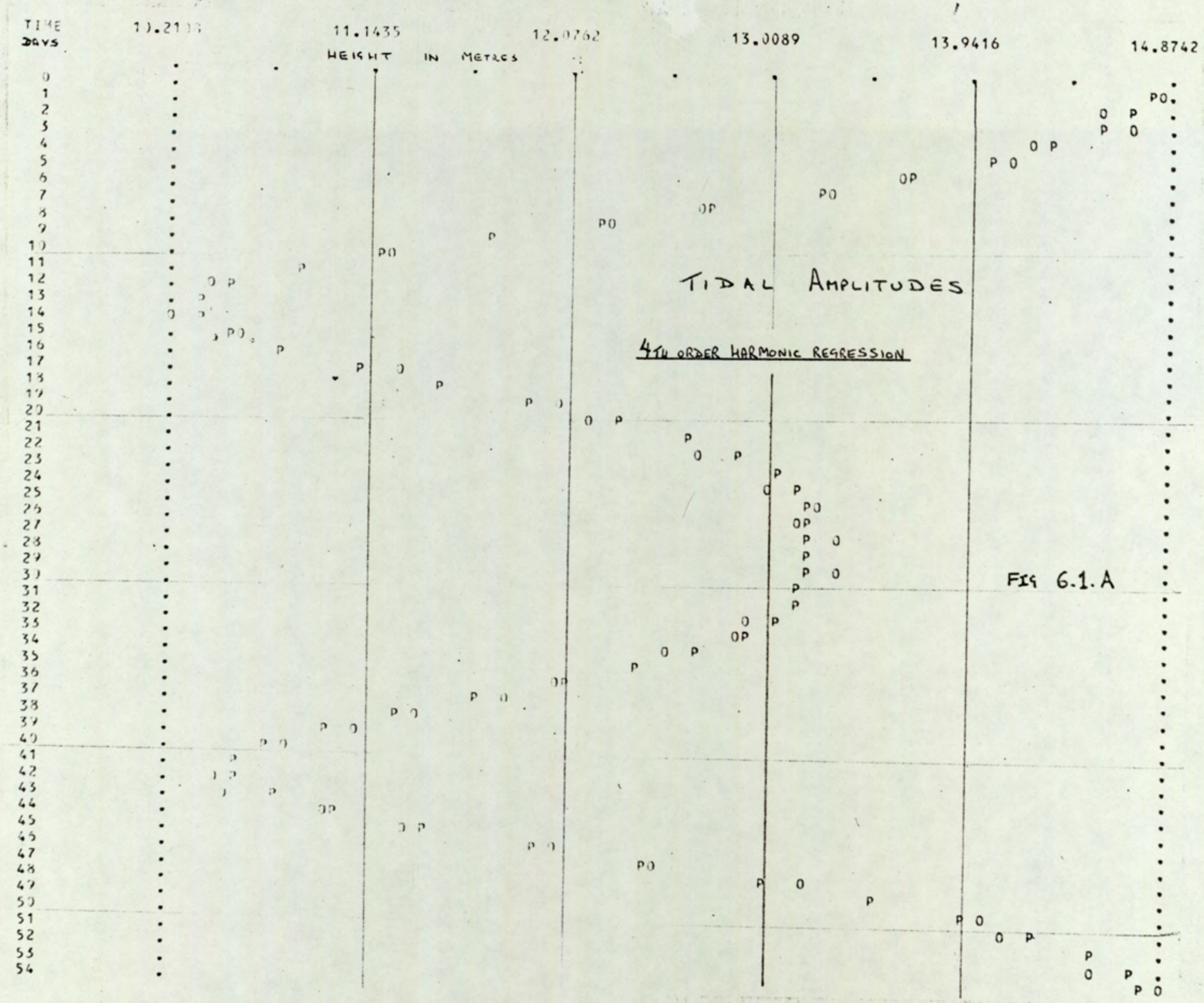
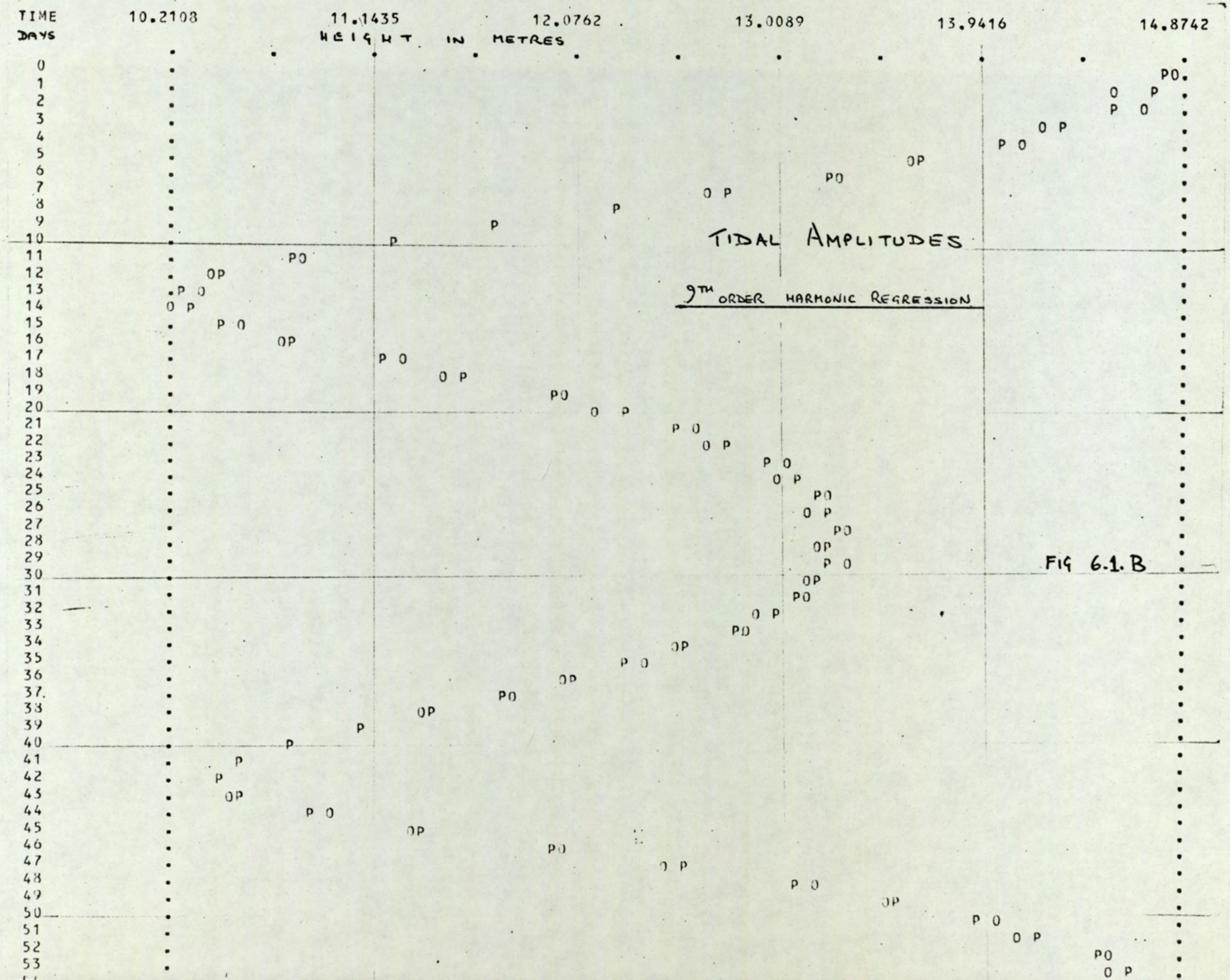

TIME DAYS
10.2108
11.1435
12.0762
13.0089
13.9416
14.8742
HEIGHT IN METRES
TIDAL AMPLITUDES
4TH ORDER HARMONIC REGRESSION

FIG 6.1.A



TIME
DAYS
10.2108
11.1435
12.0762
13.0089
13.9416
14.8742
HEIGHT IN METRES
TIDAL AMPLITUDES
9TH ORDER HARMONIC REGRESSION

FIG 6.1. B

6.2 Hydrological Data.

The Usk River Authority had commissioned an aerial survey of the tidal Usk up to a level of 30 ft. O.D.[5]. The survey was to enclose the river between Newbridge-on-Usk Bridge and the line joining the East and West Usk Lighthouses. The resultant report merely contained cut and fill type volume as heights tables for 5 benchmarked sections. The accompanying contour maps were claimed to be accurate ± 0.5' in 5' contours. Huntings kindly made the original photographic plates available and in attempting to duplicate the accuracy using stereo photogrammetry an accuracy of ± 2' was found to be more realistic because of overshadowed banks on insides of bends and general dull weather. The aerial survey data was recomputed in raw form and reprocessed to make typical cross-sections available across the main channel every 200 metres. Furthermore, as the survey was flown at low water spring, there was a critical void in the data. The sub water surface geometry was required for the full model. Above the step at Newport Road Bridge the regular geometry enables a triangular approximation with a maximum depth of 0.3m to be made. Below the Road Bridge data was available from a centre line survey conducted by the Engineers Department of the URA using echo sounding, and occasional cross-sections from Newport Borough Council Surveyors Department [5] [7] [8].

Close investigation of the data obtained revealed an exponential trend (figure 6.2.A), so the bulky aerial survey was condensed to about 25 cross-sections more scattered in the upper regions and more frequent in the lower sections of the estuary system.

Table 6.2.A.

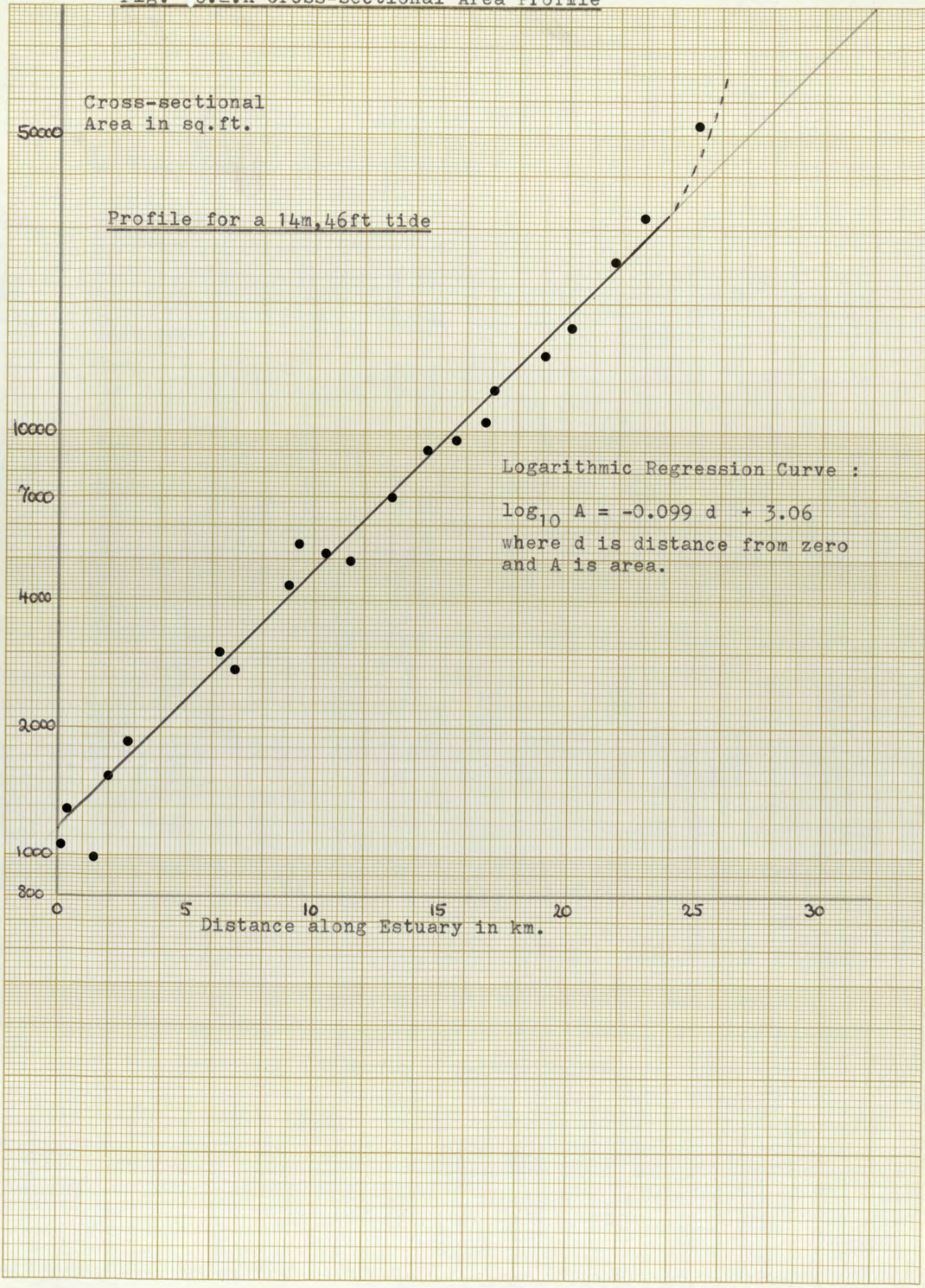
Relative Widths for Extremes of a Spring Tide.

Point	HW(ft)	LW(ft)	Banks	Bank % of HW width
Docks Entrance, just below	2500	520	1900	79%
Newport Road Bridge	360	265	195	54%
Oerlean Road Bridge	280	155	125	45%
Newbridge Bridge	140	75	65	46%

The high percentage of drying area emphasised the need for accurate estimates of volumes and surface areas (Table 6.2.A).

The Steady State Model required time averaged data the Fischer Model tables of height vs depth/width and the Stochastic Model point cross-sectional data.

Fig. 6.2.A Cross-sectional Area Profile



6.3 Discharges to the System.

The discharges to the system arise from four principal sources:

a) Newport Borough District Council.

b) Eastern Valley Sewage Works.

c) Caerleon District Council.

d) Tributaries.

a) Newport has an old sewage system which is very scattered and discharges via tidelocked doors. This results in a modification of the diurnal variations in loadings (see fig 6.3.A, double length tide cycle). As the tide rises above the door level the flow is reduced dramatically to a background leakage rate. Then the inflow is retained in the pipe network leading to the outfall. All the outfalls are gravity feed and as the reservoir builds up the pressure on the doors increases. The discharge rapidly builds up to a peak usually before the door is above the waterline. In the initial opening all the shaded area is discharged, together with the partially shaded, which is delayed only slightly. Once the reservoir capacity has been discharged, the normal and tidelocked discharge curve coincide.

Because there is no discharge around the high water area, the area of maximum tidal excursion is not utilized.

A shock load admission can create concentration effects, poor mixing and overemphasise the errors in diffusion. In areas of sub-critical flow very localised oxygen sags can result.

Table 6.3.A itemised the principal discharges to the main estuary and associated population-served.

Figure 6.3.A - Discharges Modified for Tidelocking

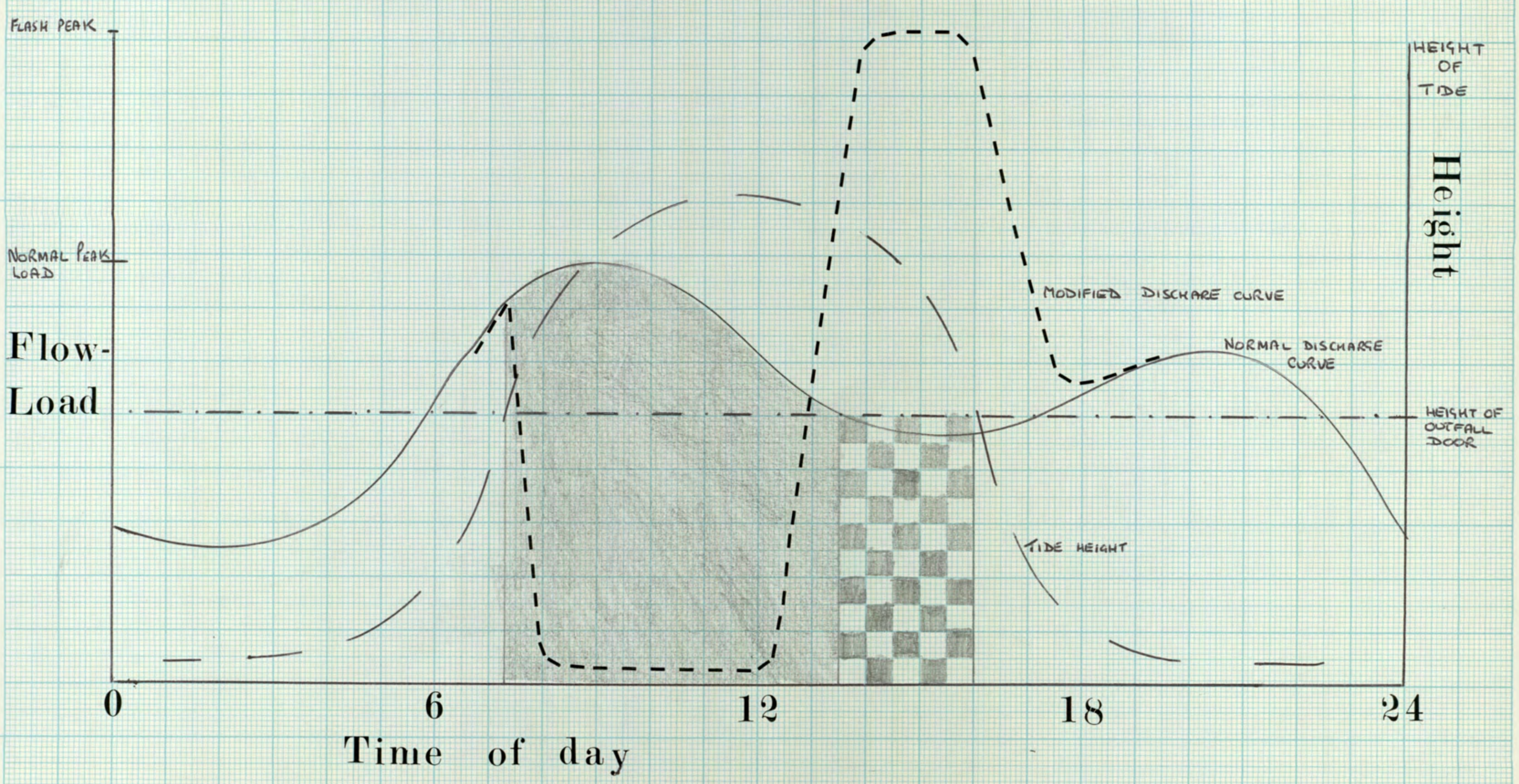


Fig. 6.3.A Tidelocking

Table 6.3.A Discharges to the Usk Estuary from NEWPORT B.D.C.[10]

Name of Sewer	Feed Pop. in '000	Flow per day in '000 gals/ day, mean 1973	B.O.D. in lbs load per day, mean 1973
Beaufort	1.5	75	180
St. Julians	4.25	213	510
Orchard	1.7	86	205
Cenotaph N.	2.84	142	342
Riverside	.35	18	50
Cenotaph S.	5.67	334	680
Maintee	10	540	1200
County	19.7	1115	2364
Brynglas/Bettws	22.9	746	2748
Barrack Hill	25.0	105	300
Civic	5.2	270	624
Town	4.5	225	540
Pill North	3.3	165	396
Tredegar Dry Dock	11.3	573	1350
Pill South	8.35	428	1002
Coronation	Industrial	500	-
Ringland	12.75	560	1528

The Borough Council plans a rationalisation of this system to remove all the major inputs and re-route them via a Sewage Treatment Plant at Nash, near the mouth of the estuary [11].

b) The Eastern Valley Trunk Sewer carries effluent from a hinterland of some 150,000 inhabitants and a wide variety of industry. The sewer is lead to Ponthir near Caerleon where it undergoes partial treatment. Up to 1967 the final effluent was discharged to the Afon Llwyd. However, as loadings rose this small tributary declined in quality and a pipeline was built to discharge direct to the Usk estuary just above the village of Caerleon. A further significant increase in load in the early 1970's was a principal reason to initiate this project. Pending the outcome of the investigation, the Secretary of State (Wales) granted permission for the increase to be permitted subject to review in the light of concerted scientific research programme.

c) Caerleon District Council has one sewage works with a reasonable level of treatment to serve a town of 10,000 and very little industry. The point of discharge is a creek on the North bank between the St. Julian and Beaufort outfalls. A 20% increase in loading occurred in the sixties due to residential development. In 1972 the town was declared a conservation area and so no major increases are imminent.

d) The principal tributaries are: R. Ebbw, R.Afon Lloyd, Sor Brook. The Ebbw is the largest of the three, with a DWF of 1.25 cummecs (or 25 mgd) but comes in at such an advanced stage that it's load nor it's dilution are significant.
The R.Afon Llwyd is a useful source of dilution together with the smaller Sor Brook, for the Eastern Valleys Effluent.

6.4 Survey Data.

6.4.0 Little specific survey work was initially planned for the project. The principal source of data input was to be a network of water quality monitoring stations (5 to 8 stations with 3 - 7 parameters) maintained by the Pollution Control Department and a series of 8 depth recorders maintained by the Authority Engineers Department. This was initiated to satisfy the constraints prescribed by the Secretary of State for Wales during a public enquiry on the extension of the Eastern Valleys Sewage Treatment plant at Ponthir [13].

6.4.1 Two stations were initially to be used for trouble shooting. These were to be sited at the B.A.C. Jetty (now Uskmouth Power Station) and St. Julians (near the small Beaufort outfall). The St. Julian monitor would sense water quality at the commencement of input from Newport's discharges. The water at this point would be well mixed from the major Eastern Valley outfall (3 km. and several large meanders away). The B.A.C. monitor would enable water leaving the system with the Newport Discharges loads. It was thought the net deterioration would show the effect of Newports loadings, but in fact the sensitive D.O. area arose in an almost invariant position. By the time the St. Julian monitor senses Eastern Valleys effluent it may well be mixed laterally and vertically and diffused longitudinally, but as it has only been resident for a short time, the decay process will not have been initiated for long enough to depelete oxygen sufficiently.

6.4.2 The B.A.C. Monitor operated over a span of four years from late 1969. Difficulties were encountered on all aspects (fig 4.6.B):

a) External Power supply was unstable as it was a feeder line from the British Aluminium Co. Smelters and when loads were taken at the works, mains voltage could drop by 25-30%.

b) Water Supply: As the station was bankside, water had to be pumped up to 80m via $2\frac{1}{2}$" plastic semi-rigid tubing to the monitor using a submersible Hoffmann Pump (which required mains supply).

c) Because of high suspended solids, the intake was liable to blockage and the pump could run dry.

d) Because of the size of the units, permanent concrete huts were built in a relatively isolated place. This was subjected to frequent vandalism.

e) Inadequacies of the system itself. Staff shortages meant that visits to the site were not as regular as required and error conditions were allowed to develop for some while.

f) Servicing difficulties. The pump could only be serviced on low water spring tides. Difficulties were encountered with the buoying system and shipping slicing the anchor cables.

g) The inside of the tubing tended to develop as a micro ecosystem and required regular flushing.

Dissolved oxygen, suspended solids, conductivity and temperature were to be monitored initially.

One major source of noise on the data acquired was the proximity of the Uskmouth Power Station (part of the CEGB). As this is a small station, with two substations, it was not always operational, so the noise could not be preprogrammed. At low water the thrust of the submerged outfall of the station's cooling water system was sufficient to create a fountain of considerable volume within the 2,000,000 galls/hour tidal prism, and 0.3m - 1.3m high. This gave rise to an area of high oxygen at a time when the general level of D.O. could be expected to drop rapidly as the tide recedes. When the tide turned, this water body passed the monitor and caused considerable deviation (fig 4.6.A). The dots on the D.O. curve are every 72 seconds and it is seen that the level rises from 40% saturation to 100% in about 8-10 minutes, and almost immediately

FIG, 6.4.A A TYPICAL CHART LOG SHOWING THE POWER STATION EFFECT (BAC JETTY)

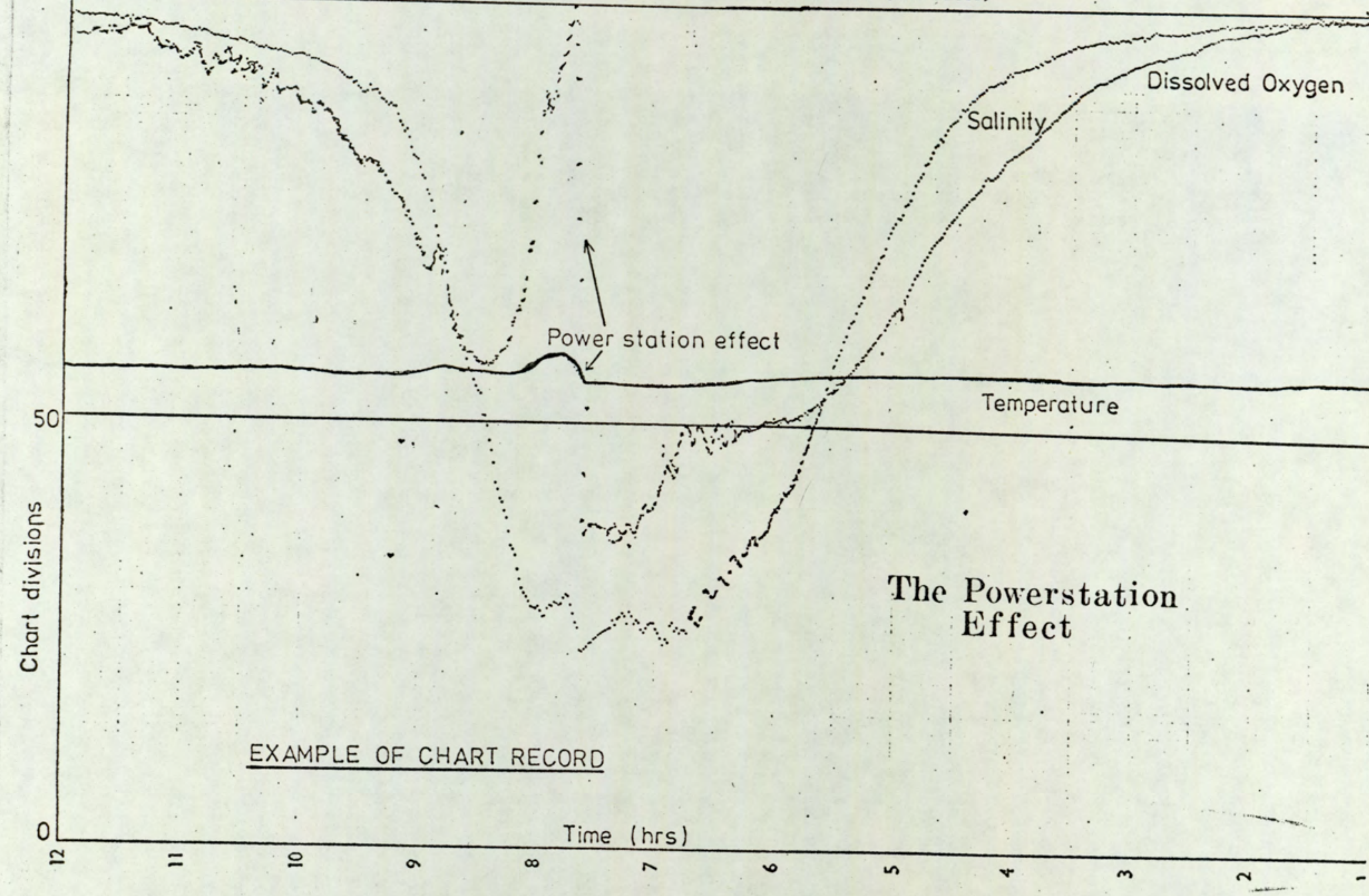
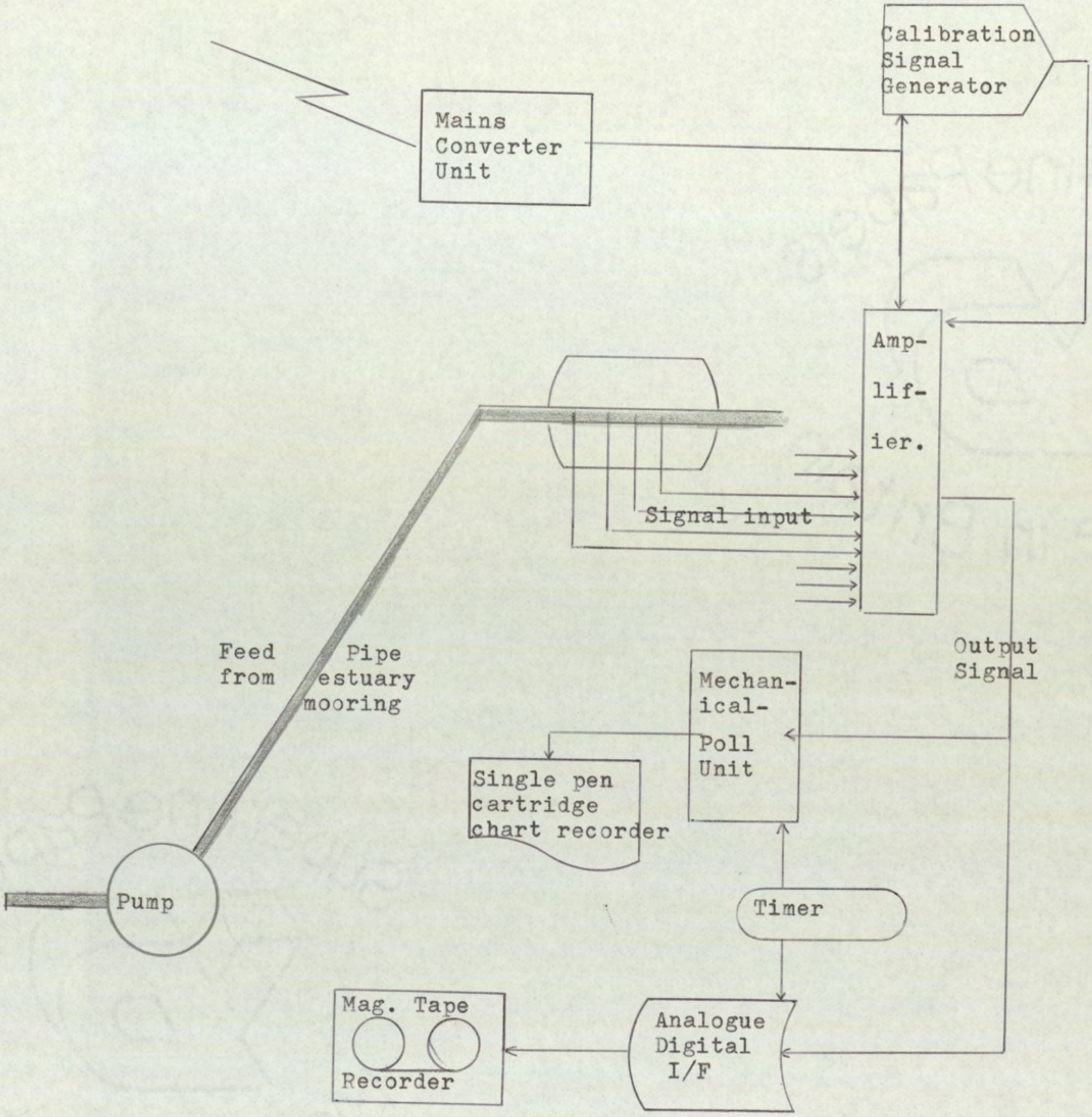


Fig. 6.4.B



General Layout of the EPSYLON INDUSTRIES Water Quality Monitor , Mark One decays so that within 25 minutes normal levels are achieved. It is interesting to note that this considerable perturbation cannot be traced on the outgoing tide. This indicates very good mixing within a tidal cycle. The temperature variation for this body of water accounts for about 80% of the variance of temperature within a tidal phase.

At present intervals (usually 5 minutes) the Analogue/Digital interface read off all channels and wrote levels to magnetic tape.

The recorder was 15 channel, of which no more than 7 were ever used including scan marker and calibration channels.

6.4.3 St. Julians experienced much the same problems although not of the magnitude of the disruption caused at B.A.C. The monitor was eleven channel only, of which no more than 6 were used. It was apparent that the U.R.A. lacked the manpower to maintain either monitor satisfactorily, and St. Julians was closed down early 1973 after just 12 months of operations.

6.4.4 Data Quality and Quantity.

It was hoped to collect 3.6 x 10^6 data values over the period of the Usk Estuary Investigation. The quality is such however that the quantity is heavily discounted.

Estimated accuracies are estimated from parallel boat surveys:

D.O. ± 1.5 mg/l	i.e. 10% - 15%	- unsatisfactory
Temperature ± 1 $^{\circ}$C		- satisfactory.
Conductivity ± 7,000		- not unreasonable.
Suspended Solids ?		- wide range deadens sensitivity

Each scan was to be prefaced by a scan marker (the digit 252). This was to be followed by an upper and lower calibration check and then either 8 or 12 data channels. The number of channels actually appearing could vary with some not registered or some recorded twice. Another common source was the loss of certain bits of a word. Losses of the 32 and upwards bits were very noticeable. Below that it is difficult to observe smaller weighted missing bits.

Typical error rates were (occurrences per 10^4 scans):

Scan marker wrong or missing	10 - 250
Calibration levels out of range	30 - 300
Incorrect number of scans	15 - 100
Observed 'bit' losses	5 - 1000

The processing problem was further compounded by switch on/switch off and channel changes on an ad hoc basis because of operational difficulties. The first 8 months of the project were devoted to trying to process the paper tapes that hold the data off the magnetic tape cassette. Unfortunately the manufacturer returned the tape in continuous, no parity, binary, non standard form. Considerable software was required to edit the output. A large part was in USERCODE (system 4 ICL machine code). No standard reader input can handle up to 400 m of such tape with no control and processing was delayed by a sporadic hardware failure (which was taken up with ICL, but not resolved in two years). Table 6.4.A estimates the advantages of processing tapes:

Table 6.4.A. Maximum Data Recovery Possible (%)

Year	D.O.	Conductivity	Temperature	Susp.Solids
1970	37%	38	39	15
1971	30%	27	31	28
1972	34%	37	35	24
1973	36%	33	30	30
Mean	35%	35%	35%	23%

By this time the volume of the software (program PTEDIT) and complexity of condition raised running costs beyond reasonable levels (1p - 2p per scan budgeted costs were now requiring 5p per scan processed), Combined with the variable quality, it was recommended that the system be closed from a continuous use method and be available for back up at times of normal estuaries.

It is worth noting that the manufacturers claimed a loss rate of one scan per 10^6. On the strength of this claim, no clock track was provided and that would have made the editing very much more efficient. Bearing in mind the quality and poor quantity, parameters are best used to describe relative relationships rather than absolute data values.

6.4.6 Instead of the bankside stations, a comprehensive boat and bridge survey schedule was planned and submitted. To compound the problem the Authority's own boat was unavailable for a year due to engine failure.

Data acquired and used is presented within the sections on specific models. All data is available as a matter of routine in the annual reports of the Authority.

6.4.7 Generally, BOD, Nitrates and Ammonias in highly saline conditions should be treated with caution due to interference in the analytical process. It was felt that modelling Total Organic Carbon (TOC) as opposed to BOD would be more beneficial but laboratory capabilities prevented this. It seems grossly illogical to proceed to model a component of dubious quantitative character with poor reproduceability solely because current legislative consent standards refer to this component. The BOD test is now nearly 100 years old and has not significantly changed in use or accuracy. In an economy orientated industrial enironment more doubt will be cast onto the BOD as alternative parameters could be easier to implement, analyse and perform in-situ measurements on.

6.5 The Re-aeration Rate.

6.5.0 Water with surface contact to a gaseous phase will dissolve an amount of that gas because of the phase rule [15], which indicates two degrees of freedom: Pressure and Temperature.

6.5.1 Henry's Law assumes ideal gases and states that "the mass of gas dissolved by a given volume of solvent, at constant temperature, is proportional to the pressure of the gas in equilibrium with the solution". This effect is independant of any other inert soluble constituent within the solvent. Le Chatelier's Principle [16] indicates qualitative response to pressure: "An increase in pressure would increase mass of dissolved gas in order to reduce the pressure constraint on the gaseous phase". (This is an abstraction of the van't Hoff (Clapeyron Clausius equation, derived from the thermodynamic relationship $\Delta F = -RT\ln K$ [15]).

6.5.2 The influence of temperature can be established with the same qualitative results from the above principals. Increasing temperature reduces solubility to make more molecules available in the gaseous phase.

A situation where an oxygen consuming pollutant with non zero reaction rate is present will result in a less than saturated solution of oxygen in the water body. The rate of restoration of the saturated level via the air/water interface is of primary interest when assessing natural assimilation capability.

6.5.3 If sufficient phytoplankton are present, additional sources of oxygen need to be considered [22]. In this project the effect was negligable. As this is the source of "supersaturation", this was not considered.

6.5.4. The recovery rate is proportional to the forcing function, i.e. the dissolved oxygen deficit in relation to saturation levels.
Carlson et. al.[17,18] formulated that

$$\frac{dC}{dt} = K(C_s - C) \qquad (6.5.4.A)$$

where C_s = Saturation level, K is the overall absorbtion coefficient (the reaeration coefficient).
Integrating between t_1 and t_2 as boundary conditions gives

$$\frac{C_s - C_2}{C_s - C_1} = C^{-K(t_2 - t_1)} \quad \text{or} \quad \log \frac{D_1}{D_2} = K\Delta t \qquad (6.5.4.B)$$

To allow for a more realistic situation where processes may be present that consume oxygen at an additional rate b(t), 6.5.4.A is written as

$$\frac{dC}{dt} = K(C_s - C) - b(t) \qquad (6.5.4.C)$$

Usually there is unsufficient data to satisfactorily estimate b as a time dependant function. The time averaged value is usually considered

$$\bar{b} = \frac{1}{T} \int_0^T b(t)dt \quad \text{where } T = \Delta t = t_2 - t_1$$

Integrating

$$\log \frac{K(C_s - C_1) - \bar{b}}{K(C_s - C_2) - \bar{b}} = K(t_2 - t_1)$$

To obtain a coefficient independent of area and volume,

f is defined as $\dfrac{K.\ (\text{Volume of water body})}{(\text{Surface Area})}$

$$= \frac{K.\ (\text{Cross-sectional Area})}{(\text{width})} \qquad (6.5.4.D)$$

Where f is the Exchange Coefficient[19]

6.5.5 The determination of this coefficient for each estuary and preferably for multiple sites and under widely varying conditions is an essential part of the survey programme. Values for one estuary have been measured varying from 1.0 ft./hour to 58 ft/hour [19]. Turbulence further increases the exchange and values up to 200 have been recorded[23].

Excluding extreme conditions, an average value for a large estuary is in the range 1.5 - 8.5 ft/hour [24] [25]. However a literature review will only highlight the lack of agreement and concensus on this topic. Several papers conclude that using published formulations can lead to using values 10 to 100 times smaller than actual values[27] [28] [29]. This is because published predictive relationships tend to be for constant geometry, unidirectional steady flow or empirical results based on characteristic types of streams.

Kramer[29] lists and reviews spheres of applicability of 17 such predictive methods.

6.5.6 Some purely theoretical expressions have been derived[30].

$$K = \sqrt{Du}/H^{3/2} \qquad (6.5.6.A)$$

where D is the oxygen diffusivity at 20°C (0.001944 sq. ft./day), u is the average stream velocity and H the average depth (or volume/surface area - cross-sectional area/width ratio). If D is in sq. ft/day, H in ft. and n in ft/sec.,

$$K = 0.538 \frac{U^{\frac{1}{2}}}{H^{3/2}} \qquad (6.5.6.B)$$

This was derived from surface renewal of a liquid film through internal turbulence. Verification was reasonably successful for a range of H : 1 ft. to 30 ft. u from 0.5 ft/sec to 1.5 ft/sec. Value of K itself rose from a minimum of 0.05 to 12.2 per day.

6.5.7 For faster water bodies, the formula

$$K = 11.6\ u/H^{1.67} \qquad (6.5.7.A)$$

is suggested[31], being basically empirically derived in flows up to 5 ft/sec, but in shallow regions (up to 11 ft).

Another expression often used combines the previous empirical work with some additional stream studies to give[32] [33]

$$K = \frac{21.6u^{0.67}}{H^{1.85}} \qquad (6.5.7.B)$$

for a velocity range 0.1 to 5 ft/sec and depths to 11 ft.

6.5.8 <u>Temperature</u> has been assumed constant for the previous expressions at 20°C. However, experimental results show that the temperature dependance[34] is well defined by the expression

$$K_t = K_{20}\ 1.0241^{(t-20)} \qquad (6.5.8.A)$$

where t is in degrees C. This represents a geometric growth of 2.41% per °C above 20°C. So

K_{10} = 78.8% of K_{20} , K_{15} = 88.7% of K_{20} , K_{25} = 112.6% of K_{20}, K_{30} = 126.9% of K_{20}

6.5.9 The saturation value has to be established in order to compute the oxygen deficit. In reaches of low salinity, the following expression is suitable[35] [36].

$$C_s(T) = 14.652 - 0.41022t + 0.0079910t^2 - 0.000077774t^3 \qquad (6.5.9.A)$$

Where salinity is appreciable, it may be required to correct for salinity. The following expression is used:

$$C_s = \alpha_t - \beta_t\ S \qquad (6.5.9.B)$$

where α_t, β_t are coefficients at temperature t (see table 6.5.A) and S is the salinity in parts per th.

Table 6.5.A - Coefficients for Calculation of C_s under Saline Conditions.

t^oC	t	βt	t^oC	t	βt
1	14.63	.0925	18	9.65	.0527
2	14.23	.0890	19	9.46	.0511
3	13.84	.0857	20	9.27	.0496
4	13.46	.0827	21	9.08	.0481
5	13.11	.0798	22	8.91	.0467
6	12.77	.0771	23	8.74	.0453
7	12.45	.0745	24	8.57	.0440
8	12.13	.0720	25	8.42	.0427
9	11.04	.0697	26	8.26	.0415
10	11.55	.0653	27	8.16	.0404
11	11.28	.0633	26	7.97	.0393
12	11.02	.0614	29	7.84	.0382
13	10.77	.0595	30	7.70	.0372
14	10.53	.0585	31	7.57	.0362
15	10.29	.0577	31	7.44	.0352
16	10.07	.0559	33	7.31	.0342
17	9.86	.0543	>33	7.18	.0332

Note : For temperature correction , use column t and t^oC. For the Salinity Correction , read ppth for temperature in deg. C and use column βt as the correction factor for use in section 6.5.9

6.5.10 In the absence of practical studies, equation 6.5.6.B is recommended for larger systems. For smaller estuaries, 6.5.7.B is probably more suitable. In badly polluted situations, the actual pollutants could be investigated. Presence of certain organic groups would tend to form surface films and radically reduce re-aeration. However, very little quantitative work has been done in this field, but the indications are that effects are marked. 0.5 ppm of some surface active matter (measured as 'Manoxol OT') reduces the reaeration rate by a factor of 0.48, and 0.1 ppm still gave a 0.42 reduction [19] [29] [40]. The reduction also seems to be moderated by improved flow rates.

The exchange coefficient is a more fundamental term to use as it is a measure of rate per unit surface area (where the interchange occurs) than the re-aeration rate, which is a more composite reflection of rate per unit volume.

The two are related by the expression 6.5.4.D rewritten as

$$f = K\,H_m \qquad (6.5.10.A)$$

where H_m is the mean hydraulic radius of a stretch or point.
A useful intuitive concept is to consider f the depth of a surface water 'slice' that is fully saturated in one unit of elapsed time if absorbtion were constant and there were no molecular or diffusive net fluxes out of the slice.

Some strong correlations between K and the longidudinal dispersion coefficients have also been established [92] although wind effects are inseparable[28].

6.6 Field Measurement of the Re-aeration Coefficient.

6.6.0 Section 6.5 illustrated the diversity of opinion on actual rates of reaeration. The Usk Estuary is a predominantly North to South flow in a narrow flood plain, between appreciable hills on either side. Because of the severity of tide level variations, the water level is well below the banks for large proportion of the tide cycle time, As the winds are predominantly westerly, it seemed feasible that the actual surface could remain reasonably sheltered and so result in a relatively low value of K or f.

6.6.1 The experiments previously used to gather field data for the reaeration coefficient were examined.

The use of simultaneous tracers[37] seemed to be an effective method but complex to operate without extensive man power and mobility/communications. The oxygen tent method[19] was practically difficult to operate in a small estuary without expert assistance. Because of an extensive natural D.O. sag curve deoxygenation is impractical because of the risk of causing irreversible damager.

6.6.2 An experiment was required that could be easily handled under ardous field conditions[38].

An approximately cubic polythene bag was manufactured of approximate dimensions 1m x 1m x 0.5m. 7 faces were closed and one side left open. Each corner had anchor eyelets anchored in the double seam. A frame of tubular steel or (preferred) aluminium is built slightly larger than the bag and the bag anchored within the rigid framework to give semi-rigidity.

The assembly is then transported to the estuary point where the experiment is to be carried out. When on station, the box is floated and filled with estuary water (figure 6.6.A). About 2-3" of the top of the plastic should remain clear so that the normal wave action will not ride into the water in the bag. The water in the bag is now deoxygenated using Sodium Sulphite and possibly a catalyst (0.2 ppm$_{Co}$[23]) to about 20% saturation. Agitation is required at this stage. The D.O. probe/recorder within the bag will show a drop to a minimum. If the level drops to 5% or less, too much sulphite has been added and the bag contents must be diluted and remixed.

6.6.3 When a minimum of D.O. has been reached, the external and internal salinities are matched, either by dilution or preferably moving station. The bag is allowed to drift with the mainstream current, while anchored to a monitoring boat. The two salinometers are used to control the positioning of the box within the selected segment of water. They should remain matched. Meanwhile the internal D.O. probe monitors the recovery of the contained water. Occasionally samples should be taken for Winkler titration as an instrument double check. If a D.O. probe is also outside the box, and the experiment commenced in an area of low oxygen, a parallel recovery may be observed if no further pollutants enter the system other than the original source.

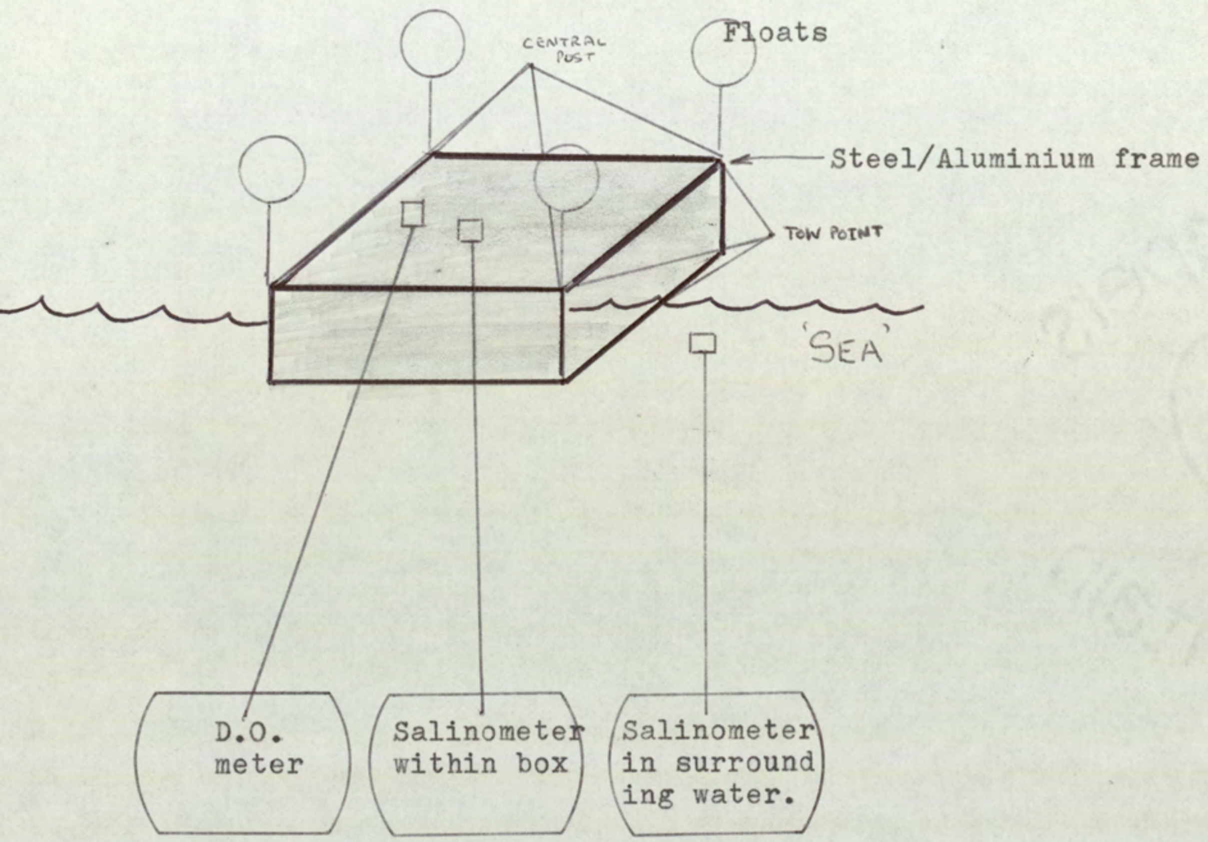
6.6.4 The concept is that this method

a) eliminates the 'diluting' effects of diffusion and dispersion (section 6.7) by using an artifically contained water mass.

b) yet the water surface experiences all the perturbations of the natural water surface (wind/wave action).

c) and, because of the flexible walls, a large degree of the turbulent mixing is transmitted to the contained water to maintain isotropy within the box.

Fig. 6.6.A. A Simple Experiment to Estimate Re-aeration in a water body



The floats are so that maximally only 2-3" of the top of the bag are above the water surface.Care should be taken not to allow many waves to break over the bag contents. The central post serves as a mounting for the two probes required.An additional D.O. instrument in the surrounding water yields useful additional data if the experiment is in an area of naturally caused D.O. depletion.

6.6.5 Some practical points are the safety considerations. In the second experiment the tidal velocity generated sufficient force on the rectangular side of the bag to put the monitoring launch into a hazardous position and the towing line had to be cut and the equipment retrieved later. For this reason it is advisable to only clip the two probes on the central post, for ease of detaching in case of urgent need to jettison. It is accepted that the Usk has a particularly violent tide surge compared to the vast majority of U.K. estuaries.

6.6.6 Both runs were in 'quiet' atmospheric conditions dry, 3-6/8th cloud and in the early morning to minimize any possible source of interference.

The run on an average tide resulted in 1.3 ft/day (1.7 cms/hr) with accuracy of ±15%. The spring tide gave 1.7 ft/day (2.1 cms/hr) with an accuracy of ±9%. The proximity of these results is surprising for widely differing tides and suggests that metereological conditions are probably the major perturbing forces. 95% confidence limits on the resultant traces (fig 6.6.B) were:

-5% = 0.9671, for a mean of 1.1139, + 5% = 1.2608

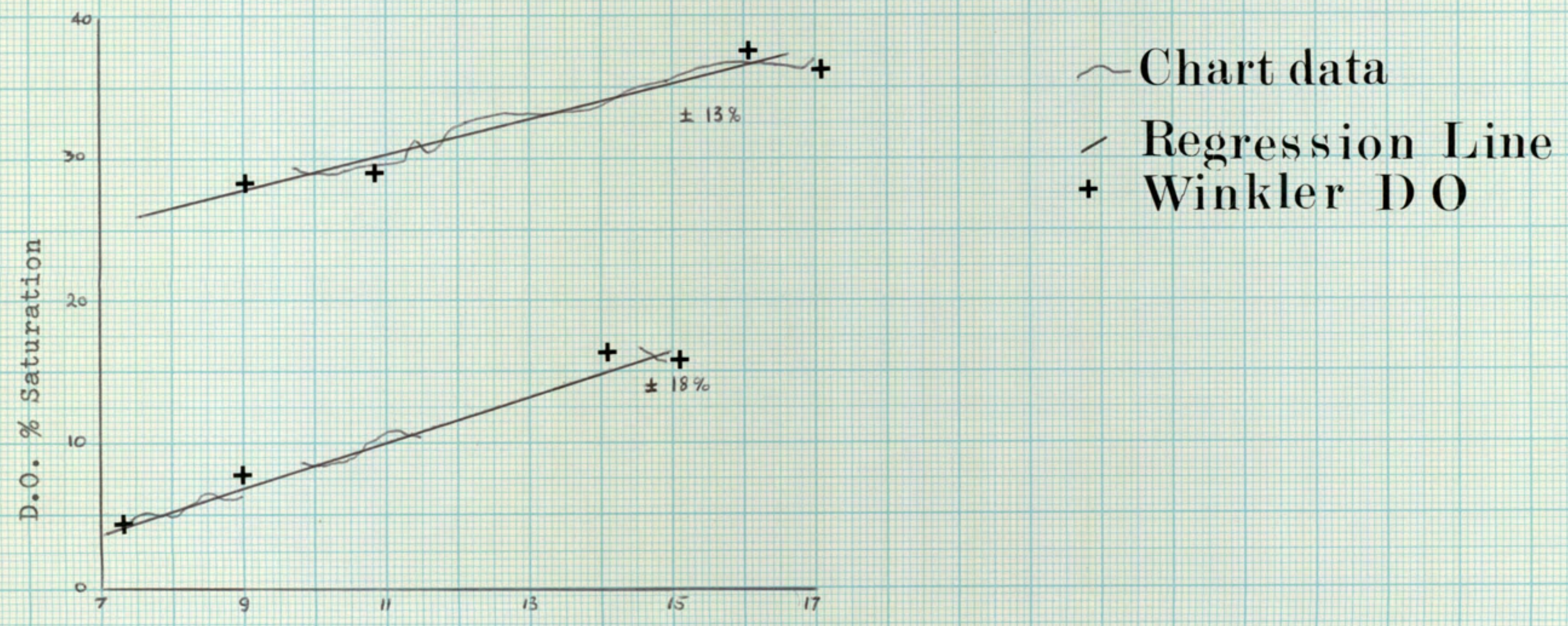
-5% = 1.4952, for a mean of 1.7173, +5% = 1.9394

(linear regression after transgeneration with all points).

Because of the multiplicity of discharges in the area of the survey, no external D.O. meter was used. Chlorophyll 'A' levels were below levels where significant amounts of oxygen could be produced (assuming that 1000 mgms/l generate at most 1 mg of oxygen per hour [22] [39].

6.6.7 The experiment is simple and easily reproduceable. For this reason it is ideal for use in quantitative investigation of the effect of organic surface pollutants. If any discharge is known to contain such surface active pollutants then they should be investigated as 6.5.10 demonstrates the radical effect on re-aeration until adequate mixing reduces concentrations to insignificant levels.

Fig. 6.6.B Re-aeration



6.7 Dispersion and Diffusion

6.7.0 Dispersion and Diffusion phenomena are a valuable source of mechanism of dispersal and therefore dilution within the receiving water body. Fluid turbulence and non-uniform velocity profiles give rise to these addit onal mechanisms. Dispersion is the longitudinal spreading of a pollutant within the water body in the direction of local instantaneous flow. Diffusion is the spreading of a pollutant within the advancing front as a 'back-up' mechanism to satisfy concentration gradients created by the dispersion front. Although the nature of the processes are not finally defined in terms of an all-enveloping theory, understanding of them has advanced in recent years to a point where numerous predictive methods are available seemingly to confirm Taylors early work in broad outline[65].

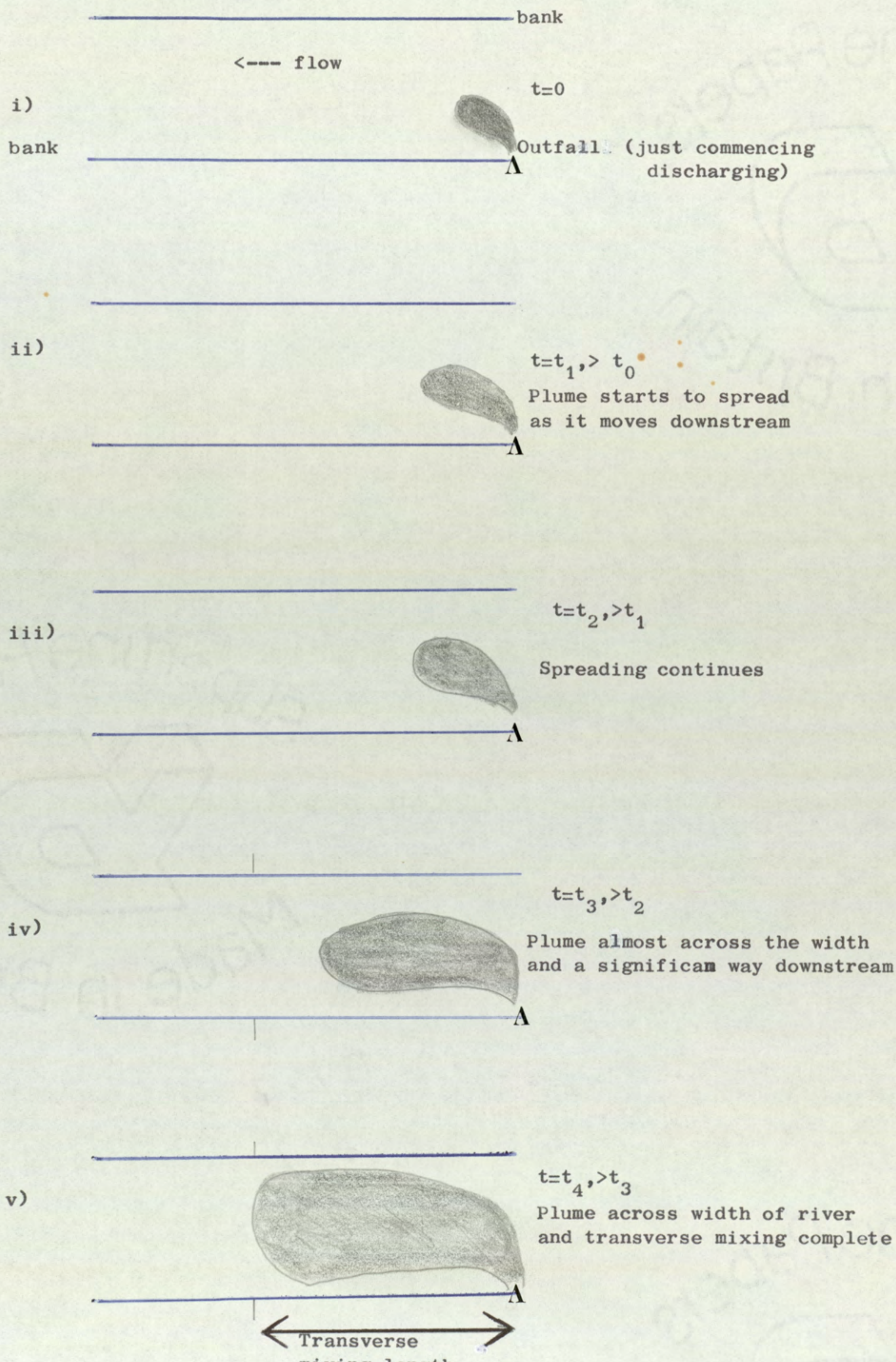
The turbulent diffusion coefficients arise in the conservation of Mass in turbulent flow equation[41] from the classical theory of Transport Equations:(See section 4.9.2)

$$\frac{\partial c}{\partial t} + u_x\frac{\partial c}{\partial x} + u_y\frac{\partial c}{\partial y} + u_z\frac{\partial c}{\partial z} = \frac{\partial}{\partial x}\left[D_x\frac{\partial c}{\partial x}\right] + \frac{\partial}{\partial y}\left[D_y\frac{\partial c}{\partial y}\right] + \frac{\partial}{\partial z}\left[D_z\frac{\partial c}{\partial z}\right] \qquad (6.7.0.A)$$

Knowledge of the longitudinal (D_x) , lateral (D_y) and vertical (D_z) diffusion/dispersion coefficients are important in a model of a real world system , and for hydraulic models because of scale effects.

6.7.1 Transverse Diffusion is important in situations where the outfall jet has a low momentum compared to the receiving water body , or because of geometry the jetting action is ineffective in the transverse direction. In this situation the dilution experienced by the discharge is less than that available from the water body. Fig. 6.2.A shows the stages in transverse diffusion for a unidirectional flow. The magnitude of the diffusion will determine the time required to proceed from state i) to v) ie t_4-t_0 . If the discharge continues the initial transport across is dispersion but the subsequent spreading diffusion within the definitions

Fig. 6.7.A Transverse Diffusion from an outfall



of 6.7.0. Many models implicitly assume complete lateral mixing , especially all one dimensional models. In the presence of oxygen consuming processes this can have serious consequences and result in severe local dissolved oxygen deficits[42].

A survey of previous studies showed that D_y can be expressed in the form [43][44][45]:

$$D_y = \lambda . H . U \qquad (6.7.1.A)$$

H and U are vaerage depths and water velocity for the point cross-section to be considered , and λ is a coefficient representative of the type of estuary being investigated. For well defined flow channel,regular geometry estuaries , the range is 0.02 to 0.04. For the Usk this value is 0.06 due to irregular flow patterns.

Setting an arbitrary definition that when 2% of the outfall concentration reaches the opposing bank, the transverse diffusion process is considered to have reached the opposing bank at a distance B away, then the

$$\text{Transverse Mixing Length } L_t = \frac{0.0543\ B^2}{0.06\ H} \qquad (6.7.1.B)$$

Applying this expression to the various main discharges on the Usk gave L_t values ranging from 300m to 500m . Eastern Valleys outfall mixing length was estimated at 400m.

Therefore it is apparent that transverse mixing in the Usk can be regarded as instantaneous , being generated primarily by tidal and bottom roughness eddies.

6.7.2 Vertical Diffusion is also considered usually as instantaneous (fig. 6.7.B) . There has been some work on this coupled with transverse mixing. The anisotropic nature of this mechanism is reflected in the reported ratios of D_y to D_z of 3[47] to 500[48]. The actual ratio is highly dependant on any stratification and is not necessarily a constant throughout the depth. Appreciable stratification usually implies ratios in excess of 15[49].

The Usk Estuary has little stratification , perhaps marginally on neap tides. The ratio therefore is likely to be in the range of 3 to 20 .

An absolute estimate can be obtained using the following expression[53]

$$D_z = \frac{2.86 * 10^{-4} * \bar{U} * H}{(1 + 0.276 N_R)^2} \qquad (6.7.2.A)$$

where N_R is the Richardson Number (stability measure), $\bar{U}$ the mean mid depth velocity of tide and H the depth.

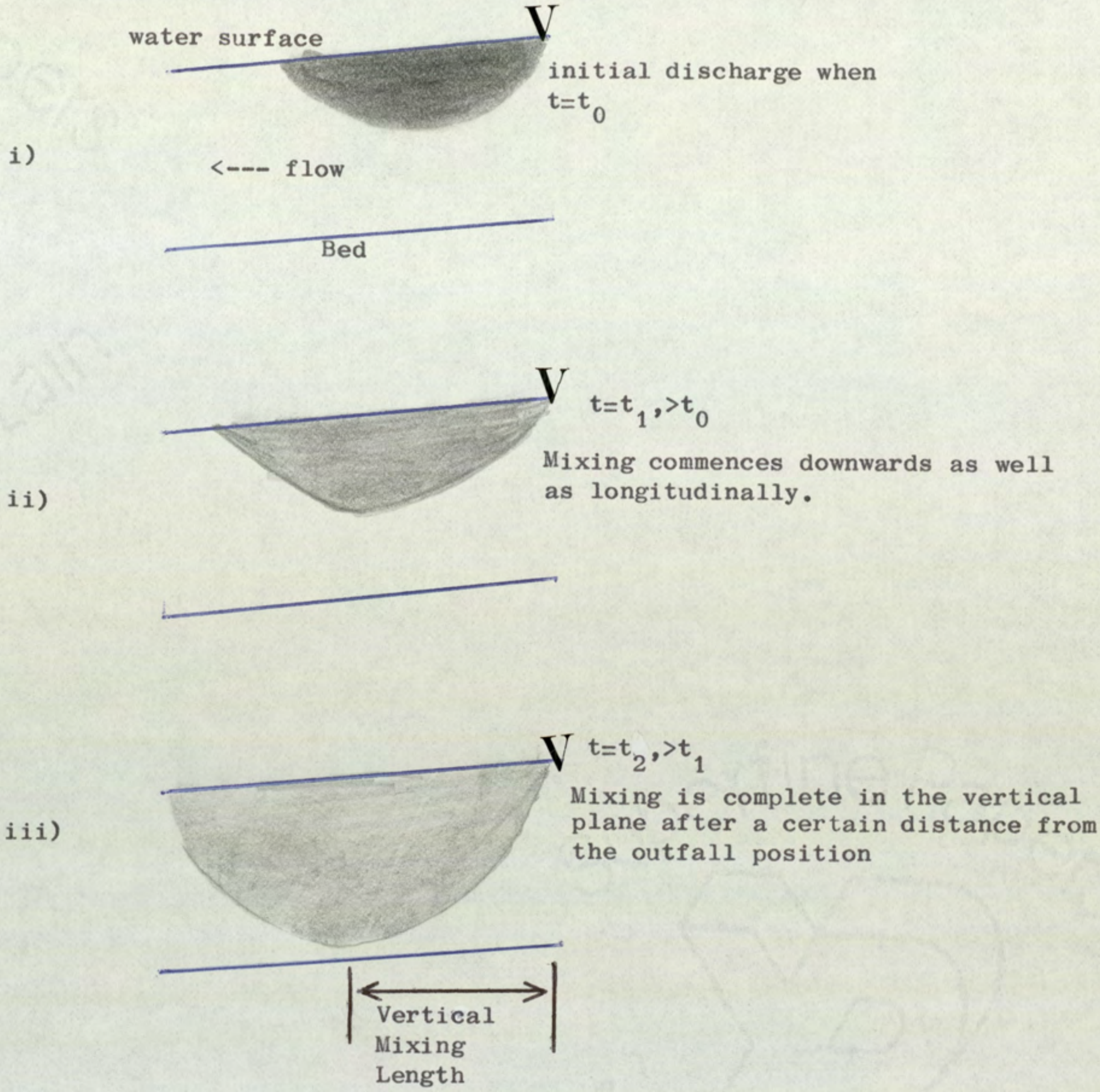
However, as the vertical plane is of less interest and for the Usk the typical mixing length values obtained are

$$L_v = 600\text{m to } 1500\text{m}$$

which can be considered instantaneous in the frame of the whole system. Practically these values are very liberal as most of the discharges for a large part of the discharge cycle have a strong vertical velocity component which is not considered in the above expression derivation.

6.7.3 At best , both transverse and vertical diffusion coefficients are considered constants over a reach if included in a model formulation. Considerable thought should be given to including these components in a model due to the wide reaching implications with limited theoretical base. The alternative method to allow these effects is to use a distance downstream weighting function for each outfall so that the main part of the load is not sensed at the outfall for the number manipulation , but some distance away from that point of discharge.

Fig. 6.7.B Vertical Diffusion from an outfall



Strength of shading is proportional to conentration, the left hand front in iii) continues to advance downstream

An alternative method of estimating these effects is to consider D_y as a function of D_z, width B, depth H , mean reach velocity V_r and mean point velocity V_p . Dimensional analysis shows that[51]

$$D_y = \left[\frac{V_p . B}{V_r . H}\right]^2 . D_z \qquad (6.7.3.A)$$

Another method used to assess field data is of the form[55][56]

$$D_{z,y} = \frac{1}{2} . \frac{d\sigma^2_{z,y}}{dt} \qquad (6.7.3.B)$$

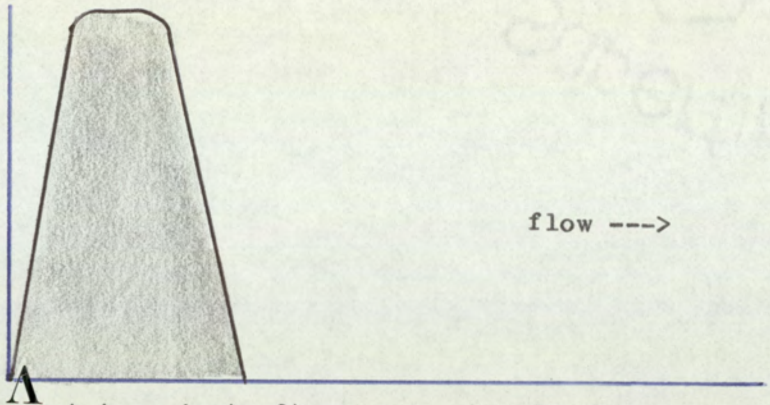
where σ^2 is the lateral or vertical variance in a measured curve and the first full derivative is used. The form used in calculation is :

$$D_{z,y} = \frac{\bar{U}}{2} . \frac{\sigma_{t_2}^2 - \sigma_{t_1}^2}{\bar{t}_2 - \bar{t}_1} \qquad (6.7.3.C)$$

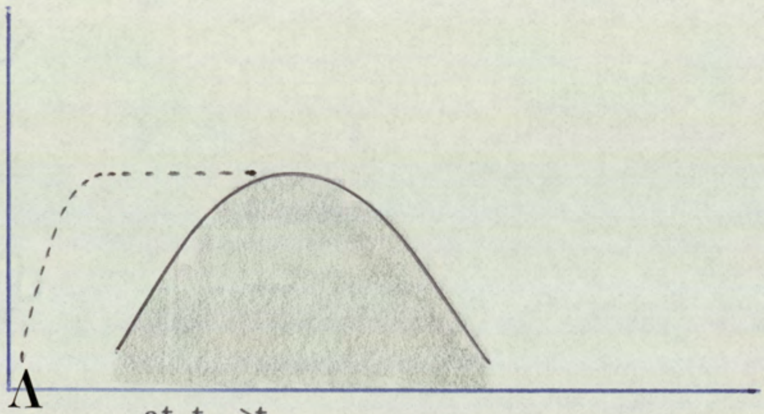
where t_1, t_2 are two times of measurement of the concentration curves and the time bar indicates the mean time of passage of tracer for the stations 1 and 2 .

6.7.4 Longitudinal Dispersion differs from transverse and lateral effects in that it has (effectively) no boundaries and so it is a process that will significantly affect a system given sufficient residence time (fig6.7.C) Few useful models ignore this effect completely , but due to practical difficulties and the dependance on many variables, this coefficient is often used as a tuning parameter in place of the re-aeration rate(sec. 6.5) However, the mainfestation of this phenomena is an additional dilution for a discharge and so it is useful to establish its magnitude with a view to its exploitation in an optimal disposal strategy.
Laboratory experiments with oscillating flows show that for steady state variables,in the sense of external forcing, D_x can be considered as time independant [50].

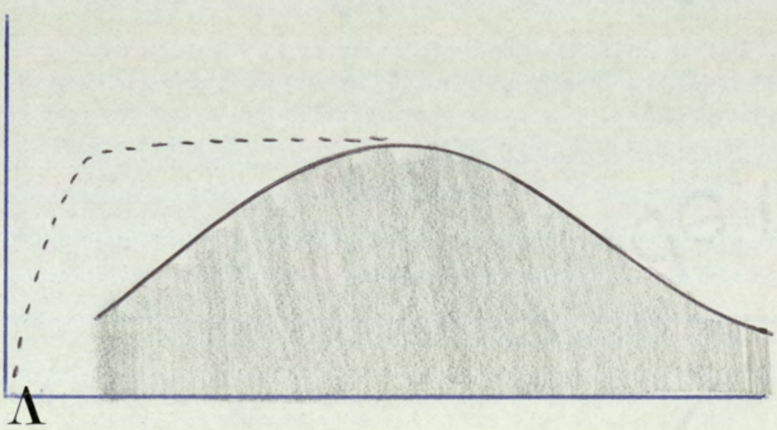
Fig. 6.7.C Longitudinal Dispersion from an outfall



at t_0, just after commencing discharge



at t_1, $>t_0$



at t_2, $>t_1$

The dotted line is the distribution for a continuous discharge from the outfall

Longitudinal diffusion forms a small effect within dispersion, in the region of contributing 4 to 15 % of the total[65][77] and both effects are considered in one coefficient due to their inherent similarity.

There are many predictive methods available to estimate D_x (table 6.7.A), but these are often for specific circumstances or a particular system. For similar system parameters the predicted values vary widely according to the predictive method used, no more than field variation though (fig. 6.7.D).

As for re-aeration, some field work is required for the estimation of the order of D_x and guide the selection of predictive methods to be employed. Table 6. 7.B gives some experimental values. Fig. 6.7.E considers only estuaries similar in size to the Usk/Severn to make comaprisons more valid. Even similar systems show considerable scatter.

From a review of literature, the set of expressions

$$D_x = \alpha \cdot u_* \cdot \delta \qquad (6.7.4.A)$$

(where δ is the mean hydraulic depth, α is a constant, u_* is the shear velocity $=\sqrt{g.R.S}$ where R is the hydraulic radius, S the surface energy slope = $-dH/dx$) look to have a wide range of applicability and are most commonly reported [57]-->[62][64].

In a system where there are multiple discharges in proximity, the dependance as a distance function becomes more important as a means of estimating whether adjacend discharges have overlapping effective perceived loads (fig. 6.7.F). If so then it must be established that the net compound loads are acceptable if a design strategy is being planned. Usually though the position of the advancing compound front is of great interest rather than its precise composition (unless the discharges have radically differing constituents)

Table 6.7.A Some Predictive Equations for D_x

Investigator	Ref	Equation Reported.
Mac Donald and Weissman	[52]	$D_x = 63.3\ n.U_{max}.R^{5/6}$
McQuivey, Keefer	[56]	$D_x = \frac{\bar{U}}{2} \cdot \frac{(\sigma_2^2 - \sigma_1^2)}{\bar{t}_2 - \bar{t}_1}$
Elder	[57]	$D_x = \left(\frac{0.404}{k_v^3} + \frac{k_v}{6} \right) . H.\ u_*$
Various , in :		D_x = constant . u_* . P where
		constant | P
Pipe Flow	[65]	10.11 | R
Open Channel	[58]	9.1 | H
Streams	[59]	500 | H
Open Channel	[57]	5.93 | H
(Aris)	[60]	const | R
(Saffman)	[61]	const | H
Smooth Channel	[62]	13.0 | H
Open Sea	[64]	3.6 | H
Parker	[63]	$D_x = 14.28\ R.(2.R.S.g)^{1/2}$
Thackston	[66]	$D_x = 7.25\ .\ u_*.H.\ (\bar{u}/u_*)^{1/4}$
Fischer	[67] [68] [69] [70]	$D_x = \frac{1}{A}\left[\int_0^z q'(z)\ .\ dz \right]^2$ where $q'(z) = \int_0^{d(z)} u'(y,z)dy$, $u' = \bar{u}_z - \bar{u}_{z,y}$

Key : n - Mannings n . U_{max} -Max tidal velocity, R - Hydraulic Rad.
H - depth, $\bar{u}$ - mean velocity from t_1 to t_2, $\bar{t}$ mean time of passage of pollutant, σ^2 - variance of distribution curve, k_v - von Karman coeff., u_* - shear velocity, S - slope of Energy line, g - accel. grav. $\bar{u}_z, \bar{u}_{z,y}$ - vertical and cross-sectional averaged velocities

Figure 6.7.D Bands of Reported Longitudinal Dispersion Coefficients

Refer Table 6.7.B for data source

Units in sq.km / day

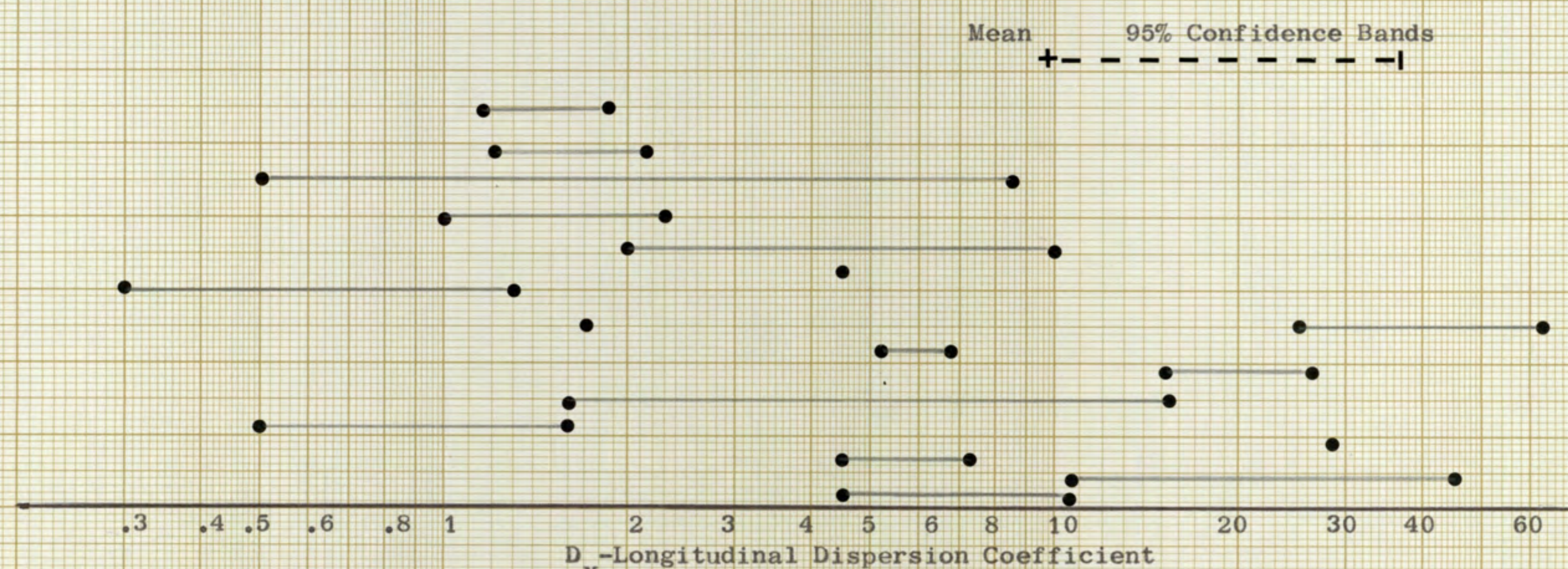


Table 6.7.B Some Values for D_x obtained Experimentally in Tidal Waters .

Site of Exp.	Comments	Ref	Value range (sq. km/day)
Severn (UK)	Summer	[71]	4.6 - 10.3
Severn (UK)	Winter	[71]	10.5 - 45.3
Thames (UK)	Low Flow	[71]	4.6 - 7.2
Thames (UK)	High Flow	[71]	28.4
Potomac (USA)	Upper 40 km	[72]	0.5 - 1.6
Potomac (USA)	Mid 40km(to 80)	[73]	1.6 - 15.4
Potomac (USA)	80km-160km	[73]	15.4 - 25.6
Waccasassa (USA)	Small Florids Estuary	[74]	5.1 - 6.9
New York Hbr. (USA)	Large System	[75]	25.6 - 61.5
Whitby (UK)	Yorks.,sea	[76]	1.7
Southern Bight (UK)	Coastal	[76]	0.3 - 1.3
Irish Sea	Sea	[76]	4.4
Lowestoft (UK)	Coastal	[76]	2.0 - 10.4
Blackwater (UK)	Essex Estuary	[76]	1.0 - 2.3
Solent (UK)	Southampton, split system.	[76]	0.5 - 8.4

Summary : Number of values - 31 Mean value 9.9 , Std. Dev. 13.8

to compare, a selection of non tidal rivers 0.019 - 1.05 , mean 0.08 per day [68] .

Figure 6.7.E Dispersion Coefficients for Small and Medium Estuaries

such as the Usk and Severn

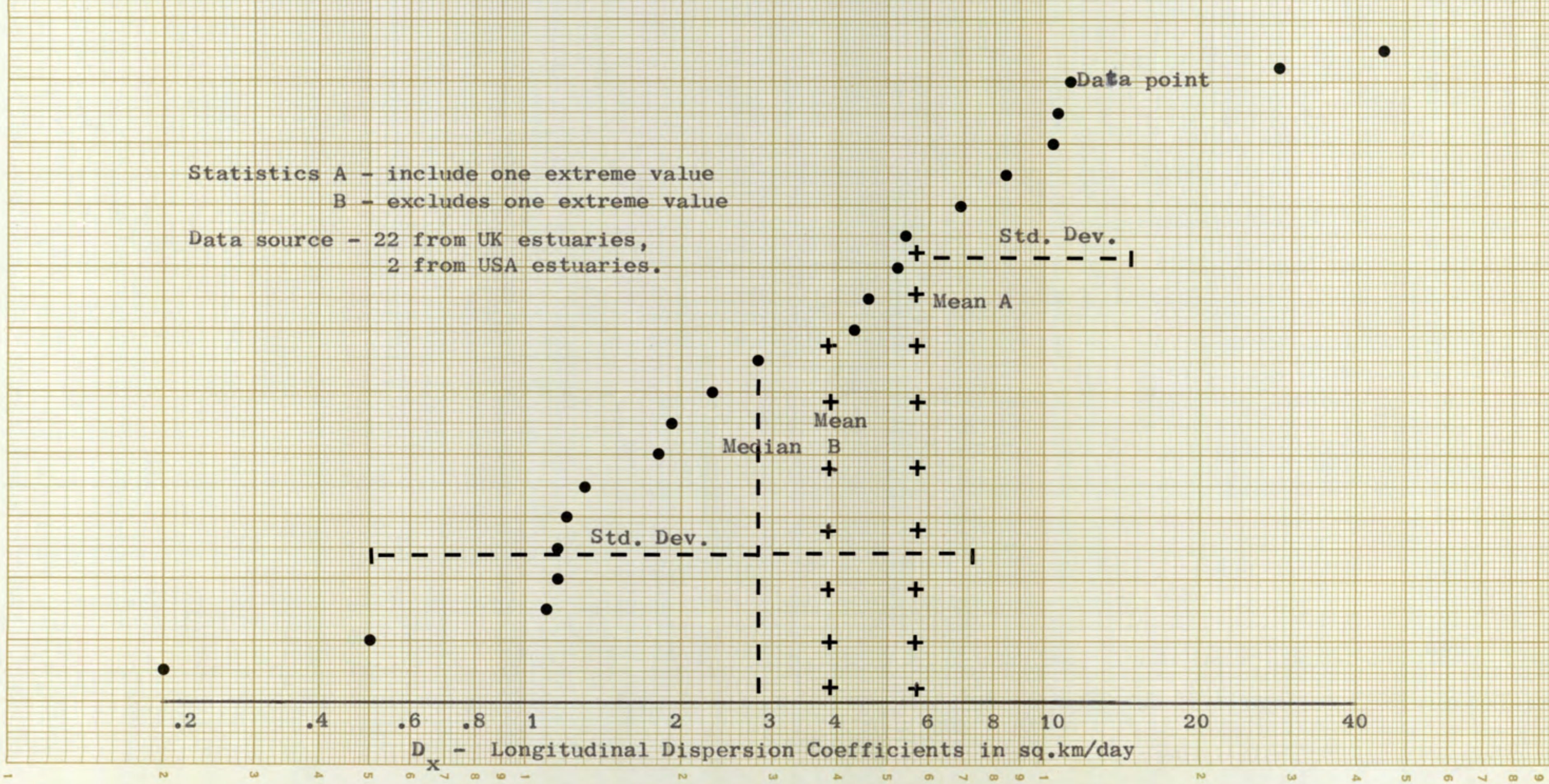
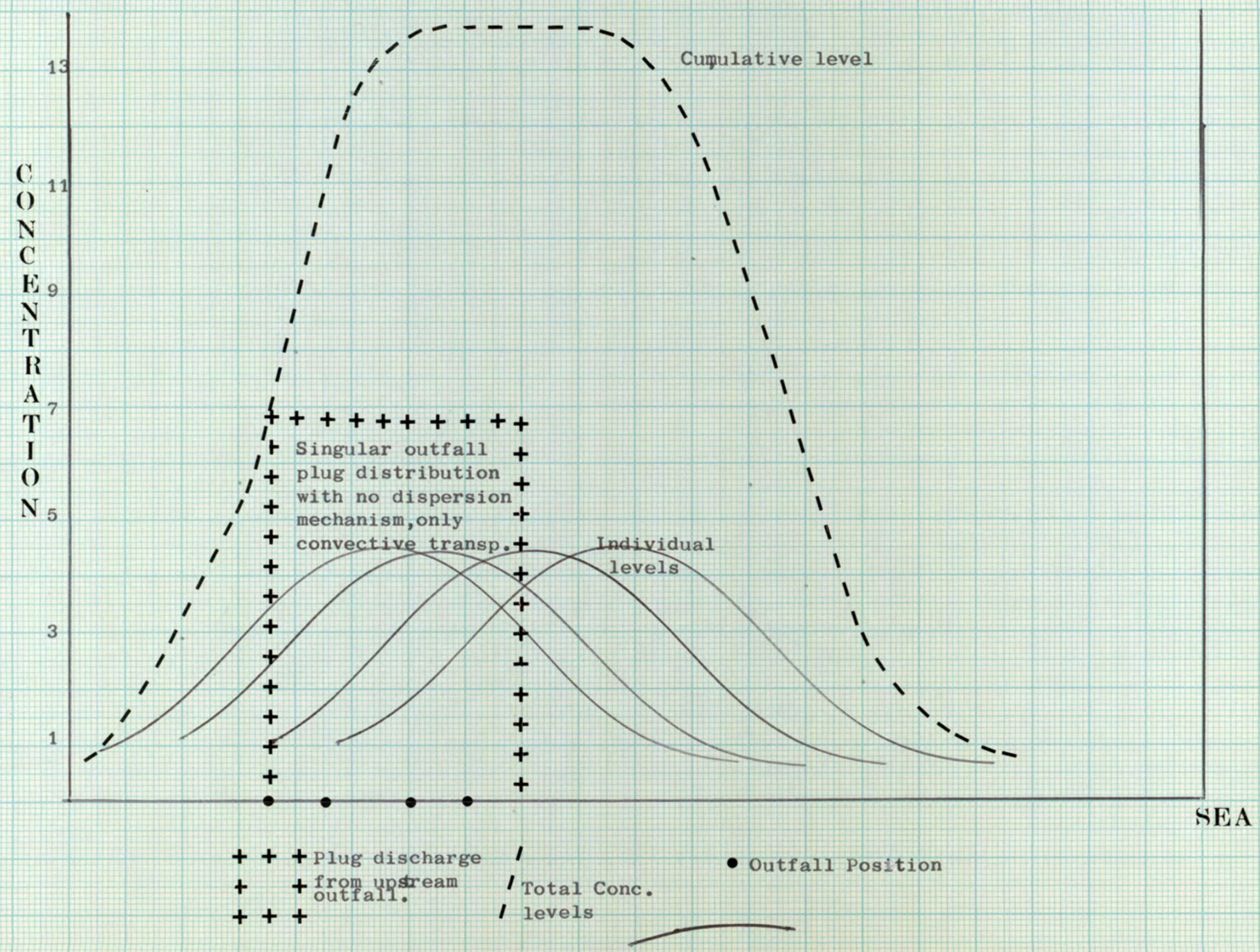


Fig. 6.7.F Cumulative Distribution for Multiple Outfalls in Proximity



6.7.5 Relative Magnitudes of Effects

Simultaneous measurements of all three parameters are restricted to hydraulic laboratories due to the complexity of the required monitoring process. However, the respective magnitudes are a useful guide in assessing the relative effects [77][87][88]

D_x	54.9	51.6	70.8	73.5	100
D_y	0.6	-	0.3	0.3	1

6.7.6 Methods of Data Collection for Measurement of the Dispersion Coefficient are all the process of fitting the distribution of a preferably consevative solute to a derived expression for D_x from a development of equation 6.7.0.A or one of its alternative forms. Tracers are widely used for this experiment.

Tracers fall in four main categories [76][84]:

a) Natural in-situ Tracers. Usually chlorides or heat in the estuary context. Tritium also has applications.

b) Chemical Tracers and Dyes. Easily detectable compounds not significantly occurring naturally in the system. Ideally completely conserved and perfectly soluble with no absobtion onto solids, organic or othetwise. A vast selection available satisfy many of criteria specified[85], although there is no one ideal tracer. Chemical tracers commonly used are chloride, sulphate, iodide. All are easily assayed and pose little problems in handling.
Among dyes, Rhodamine and Flourescein are commonly used.Rhodamine is absorbed by organic solids , but its DuPont derivative,Rhodamine WT less so. Photochemical decay and safety hazards are the main disadvantages with these dyes.

c) Radioactive Tracers.

A suitable isotope with a convenient half life is used in small quantities to label large volumes of water[84]. The obvious main disadvantages are safety hazards, legislative procedures and general public reaction. Yet quantities are very small in the absolute sense, the amount of γ activity being in the range 2-500 Curies. The great advantages are the high sensitivity and instantaneous data feedback. The final processing involves only re-computing counts back to the time of injection for absolute value.

d) Biological Tracers - this possibly includes oranges and parsnips[78] but not pumice stone[79]. The bacterium Serratia has been used as it dies off at a rate comparable to coliform organisms from effluents[80][81][82]. For longer time spans the spore Bacillus Subtilis is useful. The main disadvantage is the delay in data feedback, at best 4 hours incubation time. Therefore a larger sample programme is required.

The basic process of the experiment is

1) Dosing and 2) Tracking 3) Data Processing

Dosing can be either instantaneous or continuous depending on the terms of reference of the experiment. For data to determine D_x the method is instantaneous.

2) Tracking is the complex task of following the dosed component with minimizing the perturbations upon it. For radioactive tracers, scintillation counters coupled to chart recorders or digital printers are used. For dyes with flourescent components, a field flourimeter and chart recorder offer immediate data evaluation methods. Even for a small study data collection and boat positioning/tracking needs good organisation.[84]

The net result should be a selection of distributions of the tracer at different points in the system through time or other combinations of axis.

Fig. 6.7.J Dispersion Coefficients from Half-Tide to Low Water

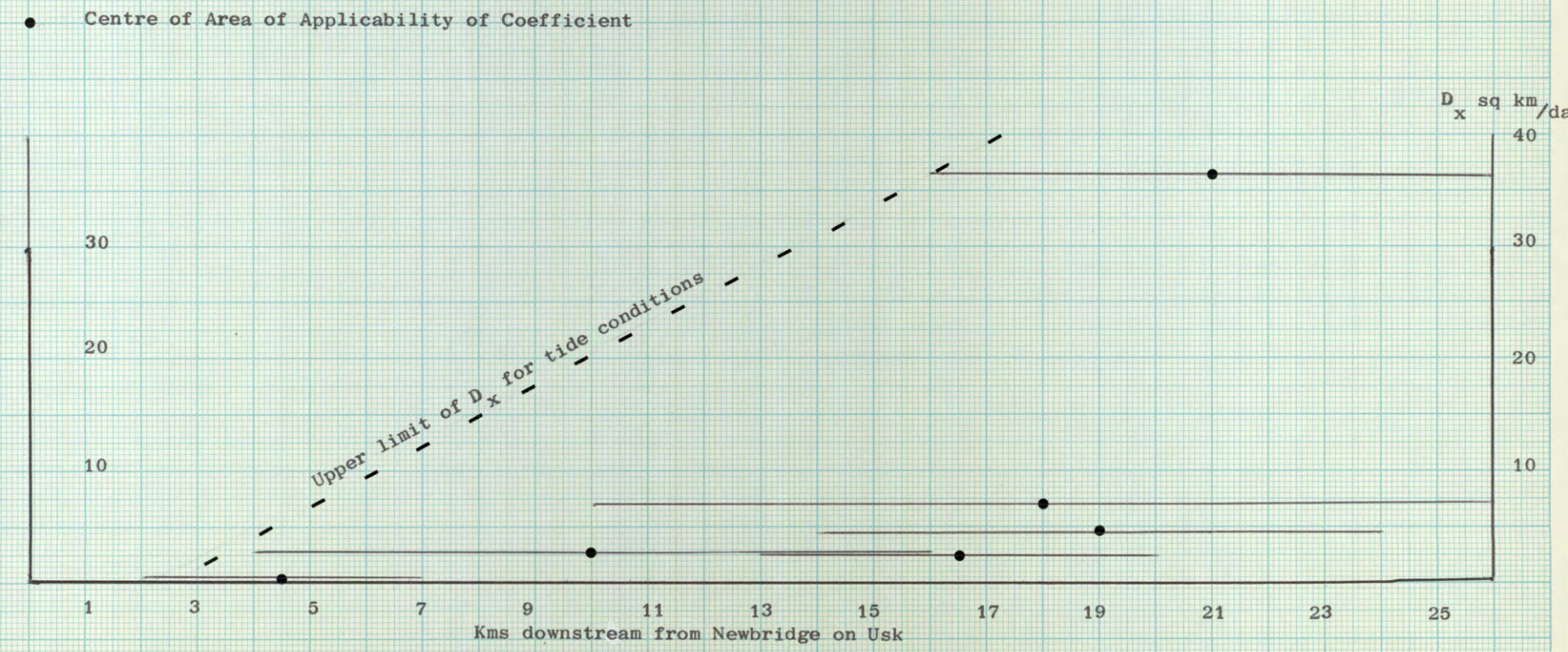


Fig. 6.7.1 Dispersion Coefficients for High Water

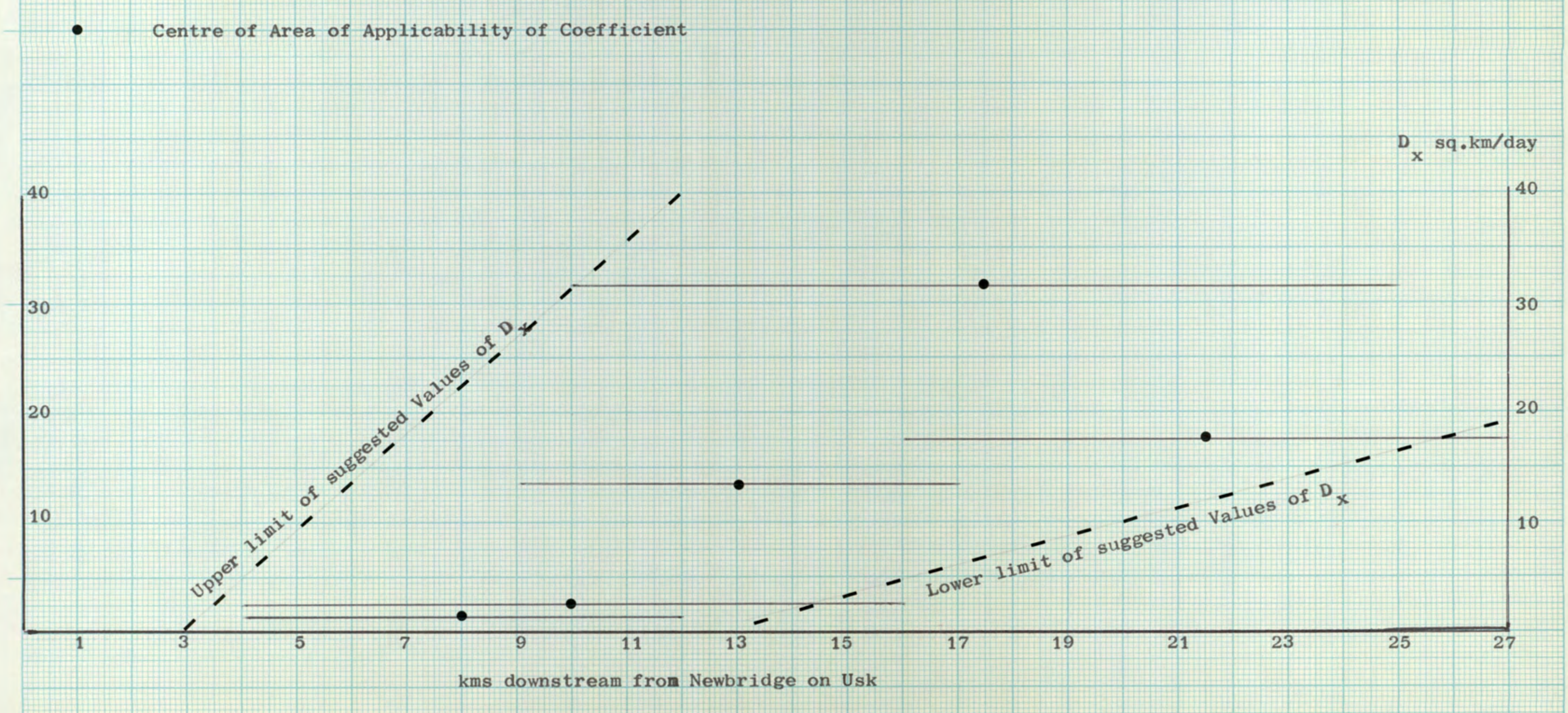


Fig. 6.7.H Salinity Distributions for Spring Tides in the Usk Estuary

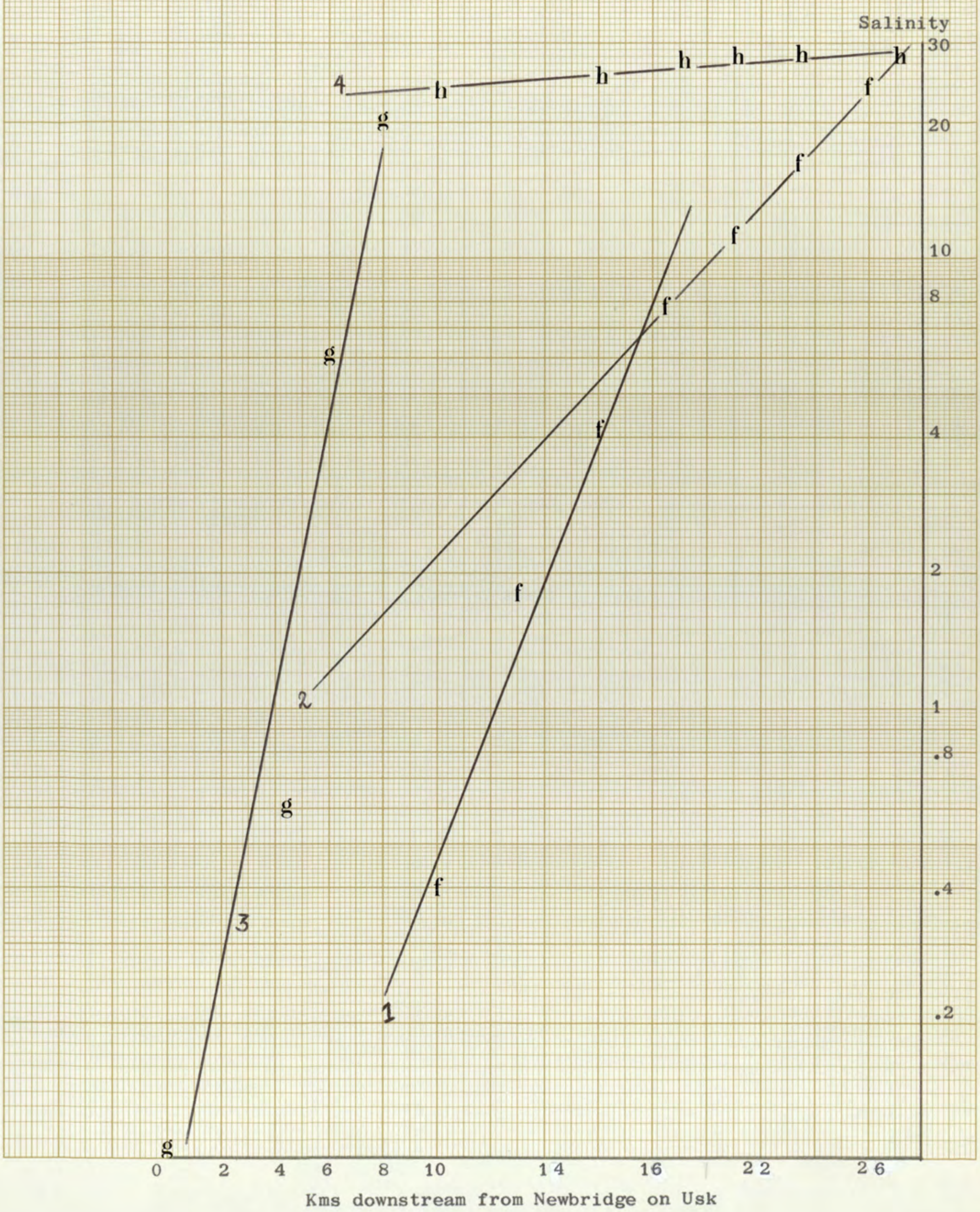
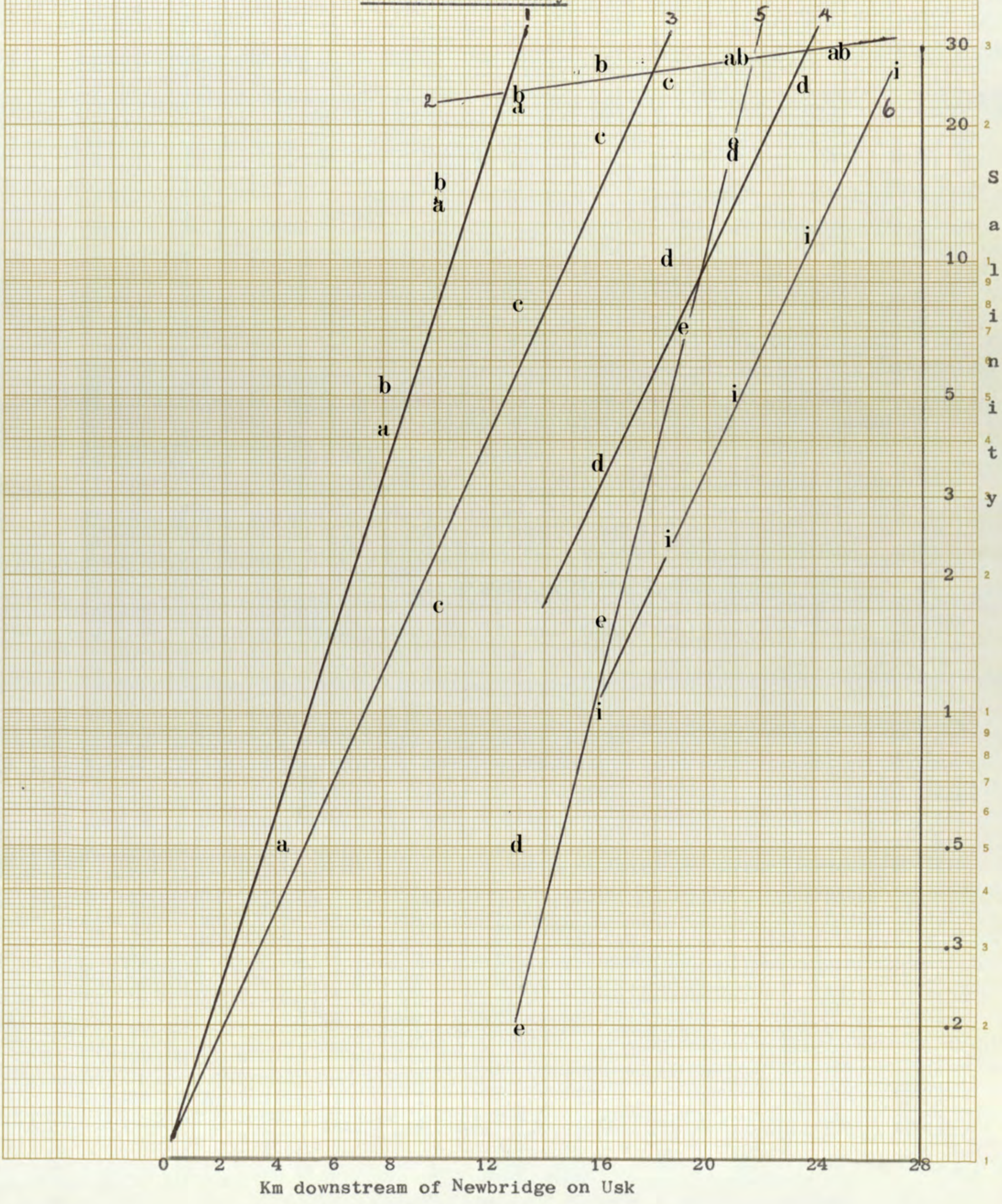


Fig. 6.7.G Salinity Distributions for Neap Tides in the Usk Estuary



3) Data Processing is the process of relating all the acquired data to a common base line because of decay, absorbtion or dilution by external forces and general interpretation of correlating data such as boat position, the developing shape of the slug of tracer. This task is much simplified with the availability of digital computer and graph-plotter.

Several texts giving details of such an experiment are available[72][76][84][86][91] for field and model tests. Work was proposed using radioactive tracer (^{85}Br, half-life 35 hours) for establishing D_x along the length of the Usk but not implemented.

However, it is possible to estimate D_x from available salinity distributions.

6.7.7 Method of Estimating D_x from Salinity Profiles

For the case of a conservative component under steady state conditions, the distributions can be described by

$$D_x \cdot \frac{d^2c}{dx^2} - u \cdot \frac{dc}{dx} + \frac{M}{A} = 0 \qquad (6.7.7.A)$$

where M is the mass of component discharged per unit time into the reach of interest , zero in the case of the Usk, A is avaerage cross-sectional area and u the average velocity. therefore

$$D_x \cdot \frac{d^2c}{dx^2} = u \cdot \frac{dc}{dx} \qquad (6.7.7.B)$$

For convenient boundary conditions, the solution for 6.7.7.B is

$$C = C_0 \cdot e^{ux/D_x} \qquad (6.7.7.C)$$

where u is the net downstream velocity, C_0 the consentration at the sea boundary (say, 30 ppth) and C the salinity longitudinal profile. Then a plot of c vs. distance on semi-log paper should be a straight line with slope u/D_x. u can be estimated by using Q/A=u for steady state inflow.

6.7.8 Calculation of D_x for the Usk Estuary

Tables 6.7.C and 6.7.D summarise the data of fig. 6.7.G and 6.7.H . The 10 typical values obtained demonstrate the inherent variability of the coefficient. Fig. 6.7.I suggests limits to be simulated for high water, fig. 6.7.J upper limits for other states of tide.

In general, the dispersion coefficient increases towards the seaward boundary of an Estuary[90]. This is due to the mixing via salinity gradients and greater turbulence[89]. This trend is pronounced in the Usk as may be expected due to its large relative tidal prism and tide turbulence .

Table 6.7.C Estimates of D_x for Neap Tides in different parts of the Usk

Line Ref.	Applicable from	to	$\Delta v\ 10^6$ cu.m	FW vel.	Grad.	D_x
1 (a)	4	12	2.0	.25	.176	1.42
2 (a)(b)	10	25	19.0	1.27	.040	31.75
3 (c)	4	16	2.7	0.23	.089	2.58
4 (d)	14	24	10.9	1.09	.277	4.01
5 (e)	13	20	5.0	0.71	.288	2.46
6 (i)	16	27	25.6	2.33	.063	36.98

Lines 1-3 for HW, 4-5 for half tide, 6 for LW

Table 6.7.D Estimates of D_x for Spring Tides in different parts of the Usk

Line Ref.	Applicable from	to	$\Delta v\ 10^6$ cu.m	FW vel.	Grad.	D_x
1 (f)	9	17	5.9	0.737	.055	13.4
2 (f)	16	27	22.6	2.05	.117	17.52
3 (g)	2	7	0.3	0.06	.186	0.3
4 (h)	10	27	2.6	0.153	.022	6.95

Lines 1-2 for HW , 3-4 for LW

Flow at 30 cumecs (just above daily mean flow of 28 cumecs)

6.8 Sources and Sinks of Oxygen

6.8.0 Apart from the process of re-aeration , other sources and sinks of oxygen may contribute to the dissolved oxygen budget. Some have already been briefly mentioned in section 6.5 .

6.8.1 Freshwater Inflow

Most Estuaries have some frshwater inflow at the head or heads of the system. This water is usually generously oxygenated and an important contribution . Unfortunaletly, the volume of this source is at a minimum at times of maximum requirement, during the summer low flow period. The Eastern Valley Outfall is sufficiently far up the system to use this as a main O_2 source. The contribution of main tributaries is also important.

6.8.2 Photosynthetic Production by Phytoplankton

As low levels of active chlorophyll and high suspended solids tend to inhibit contributions from this source[22] are small in the Usk. Contributions of 1 to 1800 tons of O_2 per day[93][94] have been recorded for estuaries. More data for the Usk will be necessary when some recovery becomes apparent. No effects have been considered in the three models.

6.8.3 Rain

Rain contributes appreciable amounts when falling as it is at least 90% saturated. For the Thames , a mean of 2 tons/day was estimated for this mechanism[19].

6.8.4 Tidal Inflow

Tidal water is usually saturated to a high degree, although having lower absolute levels due to the salinity content.In the Usk this is an important process and the models usually consider this by high boundary conditions which are prpogated through the system by the tidal prism.

6.8.5 Reduction of Nitrate

The use of nitrate as a terminal hydrogen acceptor in place of molecular nitrogen is a common property of many bacteria. Various paths can lead to different end products , dependant on the nature of the bacteria present and conditions favouring activity :

$$NO_3^- \longrightarrow NO_2^- \begin{cases} \longrightarrow NH_3 & (1) \\ \longrightarrow NO \text{ and } N_2O \quad \text{for pH<7} & (2) \\ \longrightarrow N & (3) \end{cases}$$

then (1) : NH_3

$$2NH_3 + 4O_2 + 2OH^- \longrightarrow 2NO_3^- + 4H_2O$$

If Path (1) is followed, there is no net O_2 gain as the oxidation of the generated NH_3 will require all the liberated oxidation from the preceeding reduction process. However, there can be a useful facility as more oxygen is available at a point where a large amount may be required, which can be repayed at a point downstream where the requirements may not be so pressing. The Eastern Valleys Outfall is a good example of this trade-off. Paths (2) and (3) liberate net amounts of oxygen.

Generally , once this process is established in the system, it will continue as D.O. levels continue to drop and will prevent the system from becoming completly de-oxygenated as long as possible[19][95].

6.8.6 Effluent Discharges

These usually contain appreciable amounts of oxygen unless they are very strong. Tentative estimates of up to 20% were used in the models from the little data available. As the sum of discharge volumes is less than 6% of the DWF, little effect was noticed overall.

6.8.7 Reduction of Sulphate

Sulphate reacts similarly to nitrate (6.8.5) but in a more easily reversible manner :

$$H_2 + SO_4^{--} \rightleftharpoons H_2S + [\, 4O^{\cdot} \rightarrow 2O_2 \,]$$

The H_2S is oxidised by sulphur oxidizng bacteria to produce elemental sulphur (which tends to sit in bottom muds) , sulphates and thiosulphates. As the hydrogen sulphide also tends tends to escape to the air, there is a net gain in avai lable oxygen. The pungent odour of low levels of hydrogen sulphide makes this reaction undesirable. The generation power of this process has been measured at about one fifth of the path (1) in 6.8.5 Few Estuaries have appreciable contributions from this source now, but its historical impact , particularly in the Thames Estuary, is great.

6.8.8 Artificial Means of Introduction

Structures may be built around outfalls or at critical points to encourage oxygen absorbtion. Diffusers and baffles are used to increase turbulence and so increase local exchange rates. In extreme cases air bubble guns are mounted on the estuary bed to be operated in event of low DO levels. Such sources are accomodated by either modifying the re-aeration rate distribution to include local maxima , or estimating the gross additional oxygen generated this way and adding this to the load characteristics. The advantages of such a system are obvious. However, if they are frequent in a system, their partial operation can cause severe practical difficulties in the assessment of data quality.

6.8.9 Benthic Plants

Although marine phytoplankton production data is quite extensive[99][100], very little is known about the source/sink effects of Benthic macrophytes occurring in the littoral zone. Yet two seaweed types, Chandrus and Fucus have been reported as producing 10^4 $mm^3 O_2$/gm of dry weight per hour[101]. The apparent suitability of the isotope ^{65}Zn will make the more detailed studies required easier[102][103]. No data for the Usk is available. However, as the potential is high, and the intermediate bio-assay of value anyway(via diversity[104]) , some data should be gathered.

6.8.10 Land RUNOFF

This term refers to all seepage to and from river plains through non-point sources. The effect can be net positive if there are bed springs from the aquifer supplying dilution water , or net negative if land run off is high in agricultural pollutants. This effect is difficult to estimate and is sometimes used to fit a model to measured baselines either in preference to or in conjuction with diffusion and re-aeration.

The model ST allows incorporation of a space variable steady state run-off load to the system as a source of pollutant.

6.9 The Biochemical Oxygen Demand

6.9.0 This is the principal measure of oxygen deamnd loading to water bodies. It has been adopted universally and in use now for almost 100 years, essentially unchanged[96]. A suitably diluted sample of water is incubated at 20°C for 5 days. The net difference in dissolved oxygen levels between commencement and termination of incubation is a measure of the Biochemical Oxygen Demand. The method used was the standard prescribed method[22][96] but is subject to a number of interferences. This is a measure of the oxygen uptake up to 5 days and then merely the net figure for that time. In some cases the residence times may either be shorter than 5 days, or as is more common , much longer. It is therefore essential to represent the whole course of oxidation until either the pollutant is wholly oxidized or until it is lost to the system through the downstream open boundary. If this is possible, instantaneous rates of demand can be computed for the whole of the retention period of the component under consideration.

6.9.1 It is also recognized that there are two principle oxygen deamnd areas, the oxidation of carbonaeceous constituents and the oxidation of nitrogenous constituents. There are no a priori reasons for assuming that both processes are parallel and similar in rates and character.Estuarine systems favour carbonaeceous oxidation in the first instance. Many models combine both into a singular lumped parameter representation.

6.9.2 Carbonaeceous Oxidation

Generally,

$$C + O_2 \dashrightarrow CO_2 \qquad (6.9.2.A)$$

is the predominant reaction in the chain process. Consequently, when considered net, each gram of Carbon requires 32/12 (ie 2.67) grams of oxygen for the reaction to be complete. The classical work of Phelps and Thierault[97][98] is still widely used. In this, the process summarised by 6.9.2.A is assumed to be

a) A 1st order (kinetic) reaction , ie the rate of uptake is proportional to the residual oxidzable material.

b) Implicit in a) that it proceeds independantly and parallel to other components.

c) It is independant of temperature . This will be discussed later in detail , but this is assumed initially as a base of $20^{o}C$ is taken.

Mathematically

$$\frac{dC}{dt} = - K_c . C \qquad (6.9.2.B)$$

where C is the remaining oxygen demand and K_c the first order rate constant. Integrating from time t_1 to a later time t_2 and $\Delta t = t_2 - t_1$ gives , with boundary conditions,

$$\log \left[C_2 - C_1\right] = -K_c \left[t_2 - t_1\right] = -K_c . \Delta t \qquad (6.9.2.C)$$

and where C_i is the instantaneous demand at time t_1. 6.9.2.C can be rewritten as

$$\frac{C_2}{C_1} = e^{-K_c . \Delta t} \qquad (6.9.2.D)$$

As $C_2 - C_1$ is the net difference in demand, and the execution of a unit of deamnd creates by stoichiometry, a unit of deficit but with an opposing sign , giving

$U = C_1 - C_2$, so $\log -U = -K_c.\Delta t = \log 1/U$ (6.9.2.E)

therefore $U = C_1\left[1 - e^{-K_c.\Delta t}\right]$ (6.9.2.F)

This last expression is identical for radioactive substances with a known half life. Using 6.9.2.F with $C_2 = C_1$ gives the expression

$$\delta_t = \frac{0.693}{K_c} \qquad (6.9.2.G)$$

(where δ_t is the half life)

The relationship of δ_t to the average retention time can be considered as the proportion of oxygen consuming processes excercised within the system. Consequently, longer retention times and high deacy rates are to be avoided.

The value for K_c normally used is 0.23 per day , being derived from work carried out in the Thames[19,p.213] and by Theriault[98] in preferance to the value obtained by Gotaas[108]. This is for 20°C constant.

Using 6.9.2.G it is seen that δ_t, the half life of domestic sewage, is of the order of 3 days. Table 6.9.A and Fig. 6.9.A and B show the percentage of the oxygen deamnd excercised and remaining after elapsed times from discharge.

Table 6.9.A Average Values of Percentage of Oxygen Demand Excercised and Remaining

Time (hrs)	% OD Excercised	% OD Remaining
1 hr.	1.0	99.0
3 hrs.	2.8	97.2
6	5.6	94.4
10	9.2	90.8
20	17.5	82.5
1 day (24)	20.5	79.5
2 days (48)	36.9	63.1
3 (72)	49.8	50.2
4 (96)	60.1	39.9
5 (120)	68.4	31.6
8 (192)	84.1	15.9
10 (240)	90.0	10.0
2 weeks (336)	96.0	4.0
3 (504)	99.2	0.8
4 (672)	99.84	0.16
30 days (720)	99.90	0.10
45 days (1080)	99.9997	0.0003
60 days (1440)	100	0

Table 6.9.B and Fig. 6.9.C shows the effect of varying basic rate of decay in terms of percentage deamnd excercised.

Other forms have been considered to represent the process on 6.9.2.A .

The two main alternatives are the retarded exponential decay and the multiple deçay rate representation.

Table 6.9.B Percentages of Oxygen Demand Excercised at different Elapsed Times since SDischarge at different values of K_c

Elapsed Time	Rates of decay in per day					
	0.15	0.20	0.23	0.27	0.3	0.4
1 hour	0.6	0.8	1.0	1.1	1.25	1.65
3 hours	1.85	2.5	2.8	3.3	3.7	4.4
6 hours	3.7	4.9	5.6	6.5	7.2	9.5
10 hours	6.1	8.0	9.2	10.6	11.8	15.0
20 hours	11.8	15.4	17.5	20.1	22.1	28.3
1 day (24)	13.9	18.1	20.5	23.7	25.9	33.0
2 days (48)	25.9	33.0	36.9	41.7	45.1	55.1
3 days (72)	36.2	45.1	49.8	55.5	59.3	69.9
4 days (96)	45.1	55.1	60.1	66.0	69.9	79.8
5 days (120)	52.8	63.2	68.4	74.1	77.7	86.5
8 days (192)	69.9	79.8	84.1	88.5	90.9	95.9
10days (240)	77.7	86.5	90.0	93.3	95.0	98.2

The retarded exponential decay rate assumes a time dependant K_c, of the form

$$K_c(t) = K_c(0) \;/\; (\; 1+C_r.t) \qquad (6.9.2.H)$$

where C_r is the coefficient of retardation and can be a function of the point of application but more usually a constant and $K_c(0)$ the initial rate of decay. Using this modifies 6.9.2.F to

$$U = C_1\left[\; 1 - (1+C_r.t)^{(-K_c(0)/C_r)}\right] \qquad (6.9.2.I)$$

There are difficulties in esablishing the initial rate as fast components may vary and thus distort the initial slope of the curve.A large volume of data is required to establish C_r with any degree of satisfaction.

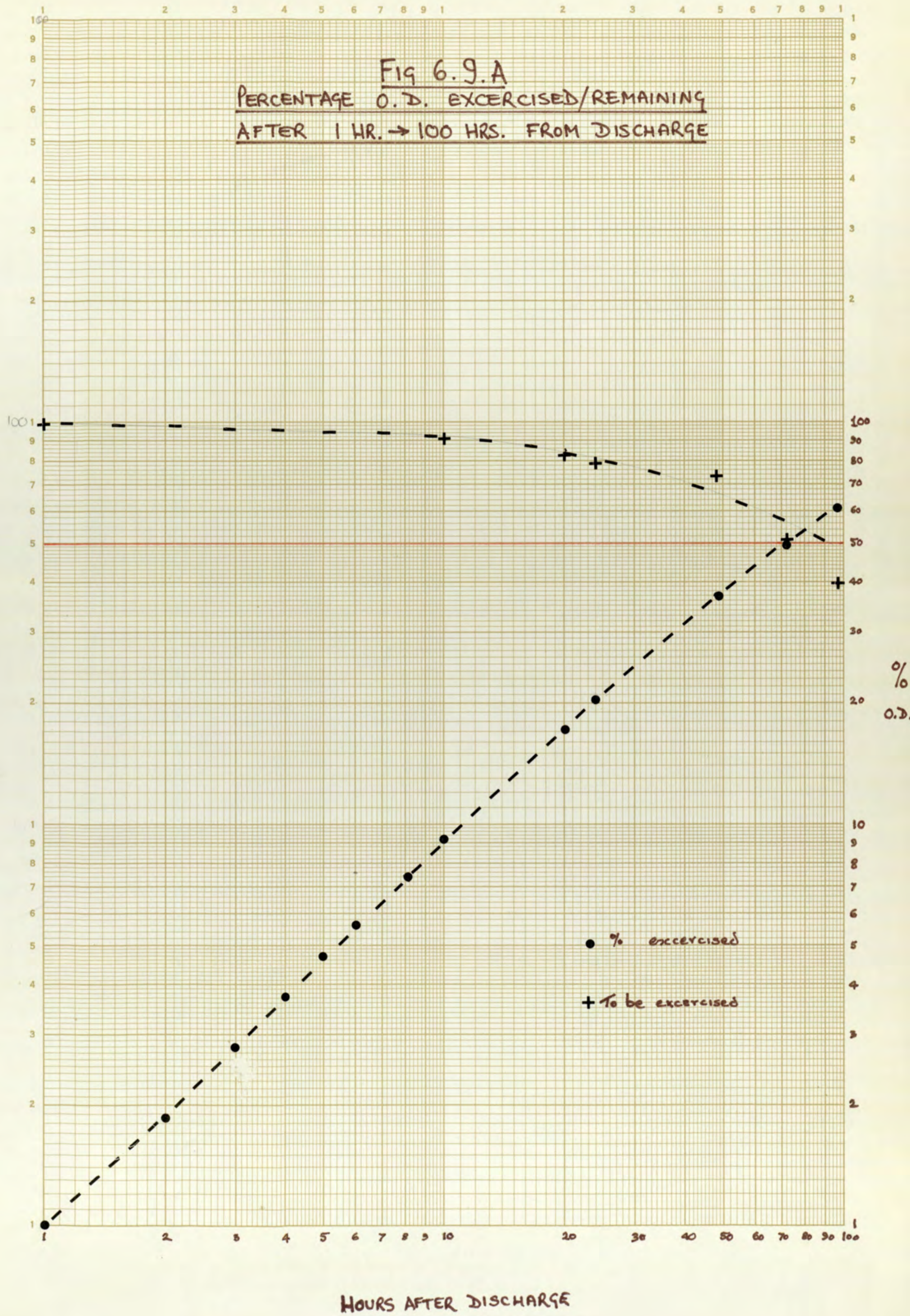


Fig 6.9.A
Percentage O.D. excercised/remaining
after 1 hr. → 100 hrs. from discharge
● % excercised
+ To be excercised
% O.D.
Hours after discharge

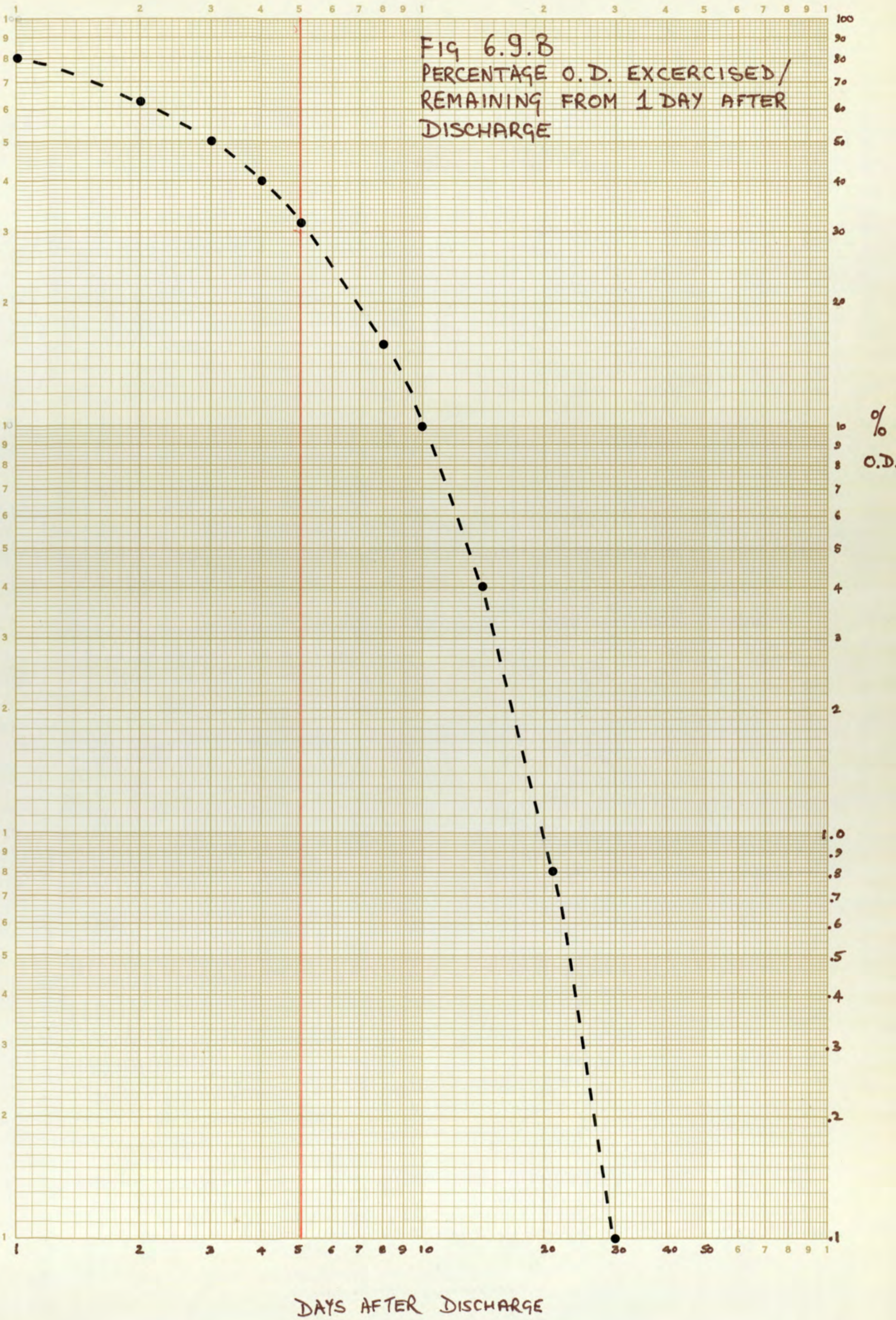


Fig 6.9.8
PERCENTAGE O.D. EXCERCISED/
REMAINING FROM 1 DAY AFTER
DISCHARGE

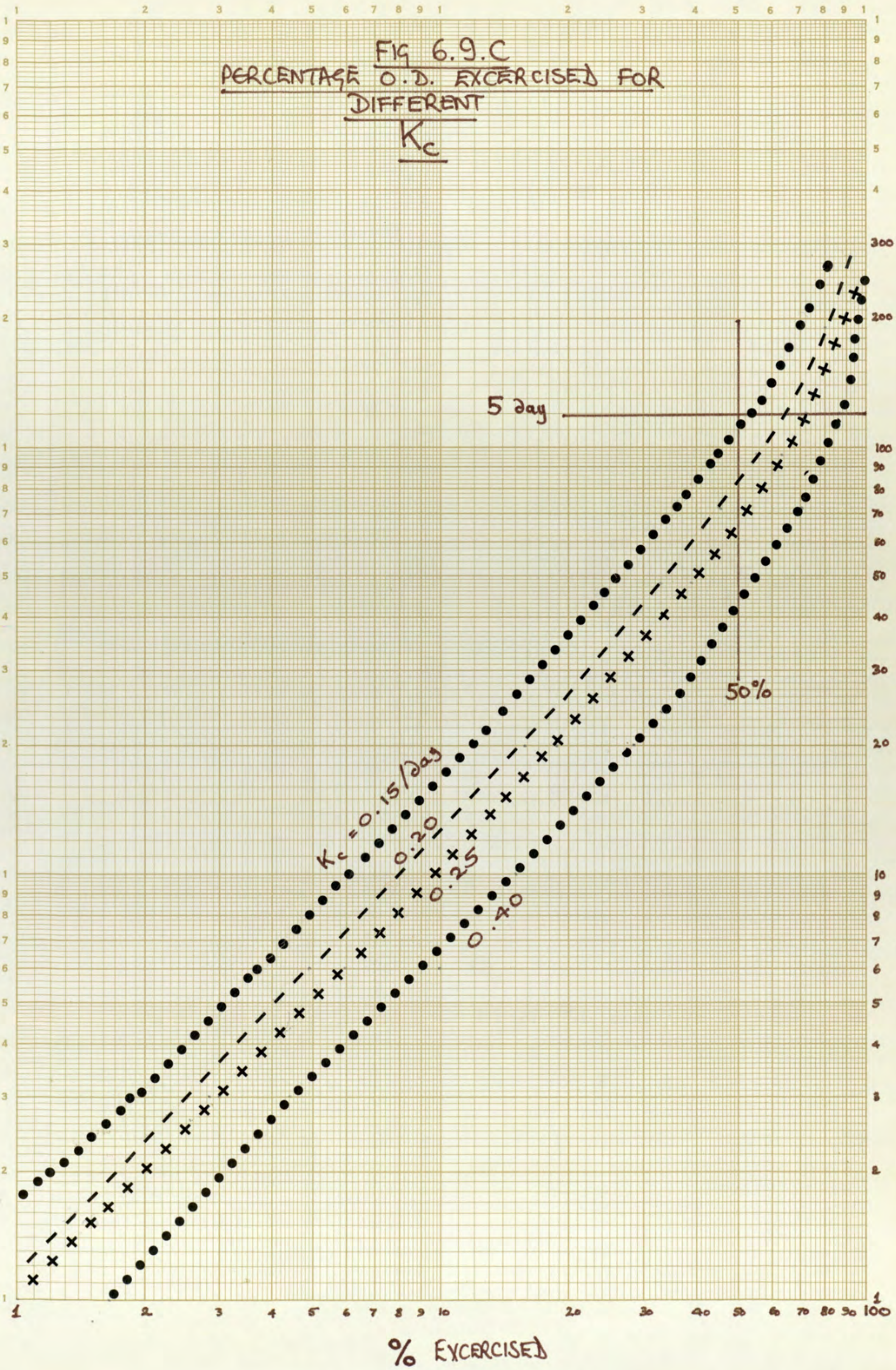


FIG 6.9.C
PERCENTAGE O.D. EXCERCISED FOR
DIFFERENT
K_c
5 day
50%
K_c = 0.15/day
0.20
0.25
0.40
ELAPSED TIME IN HOURS
% EXCERCISED

An alternative form of dealing with a non-first order is to assume that as the effluent is invariariably a mixture of conponents, the resulting decay characteristics are a summary of the individual independant components. Consider an effluent of n constituents, so that the probability that a unit of pollutant is of component i is p_i , and also

$$\sum_{i=1}^{n} p_i = 1 \qquad (6.9.2.J)$$

The rate constant associated with component i is K_{ci}.

Proceeding from 6.9.2.B to 6.9.2.E for each of the n components and then summing using the superposition principle gives a modified form of 6.9.2.F

$$U = C_1\left[1 - \sum_{i=1}^{n} p_i \cdot e^{(-K_{ci}\cdot\Delta t)}\right] \qquad (6.9.2.K)$$

Where constituents are highly specific, this representation is preferred. Up to three terms have been used to fit uptake curves to settled sewage[110]. Laboratory investigations into the broad category of carbonaeceous oxidation have shown[19] that the uptake characteristics can be duplicated very effectively by using 6.9.2.K with two terms. K_{c2} was generally found to be $K_{c1}/5$, and similar results were established for nitrogenous decay. So

$$U = C_1\left[\,1 - \left[(1-p)e^{-K_c\cdot\Delta t} - p \cdot e^{-K_c\cdot\Delta t/5}\right]\right] \qquad (6.9.2.L)$$

where p is the proportion of the 'slower' rate component in the carbonaeceous oxidation source'pool'. Δt is the elapsed time since discharge. Values of p were found by experiment[19][117] and summarised in fig. 6.9.E This representation is used in the steady state model SSM.

Eq. 6.9.2.K can also be used for components with delay times prior to commencing oxidation. This may arise where another process proceeds in preference because of prefeered conditions or a more favourably direct reaction path.

6.9.3 The Effect of Temperature on K_c

Pleissner's early work[111] was superceded by Streeter and Phelps[112] with an empirical relationship of the form

$$K_{ct} = K_{c,20} \cdot \beta^{(t-20)} \qquad (6.9.3.A)$$

where β was 1.047. Theriault confirmed 6.9.3.A independantly[98]. Various investigations have revealed broadly similar results with only some exceptions (fig. 6.9.D).

An alternative to the empirical relationship is to use the Classical Arrhenius Equation[15][19][116]. The values so obtained for K_c are markedly higher than those established experimentally[108].

6.9.4 The Effect of temperature on C_1 - the Oxygen Demand

Although generally accepted that the C_1 of eq. 6.9.2.C is not temperature dependant[108], some workers have reported differently and is a point of interest[98][113][115]. This could be due to the activation required for initiation of various active paths for the net reaction 6.9.2.A

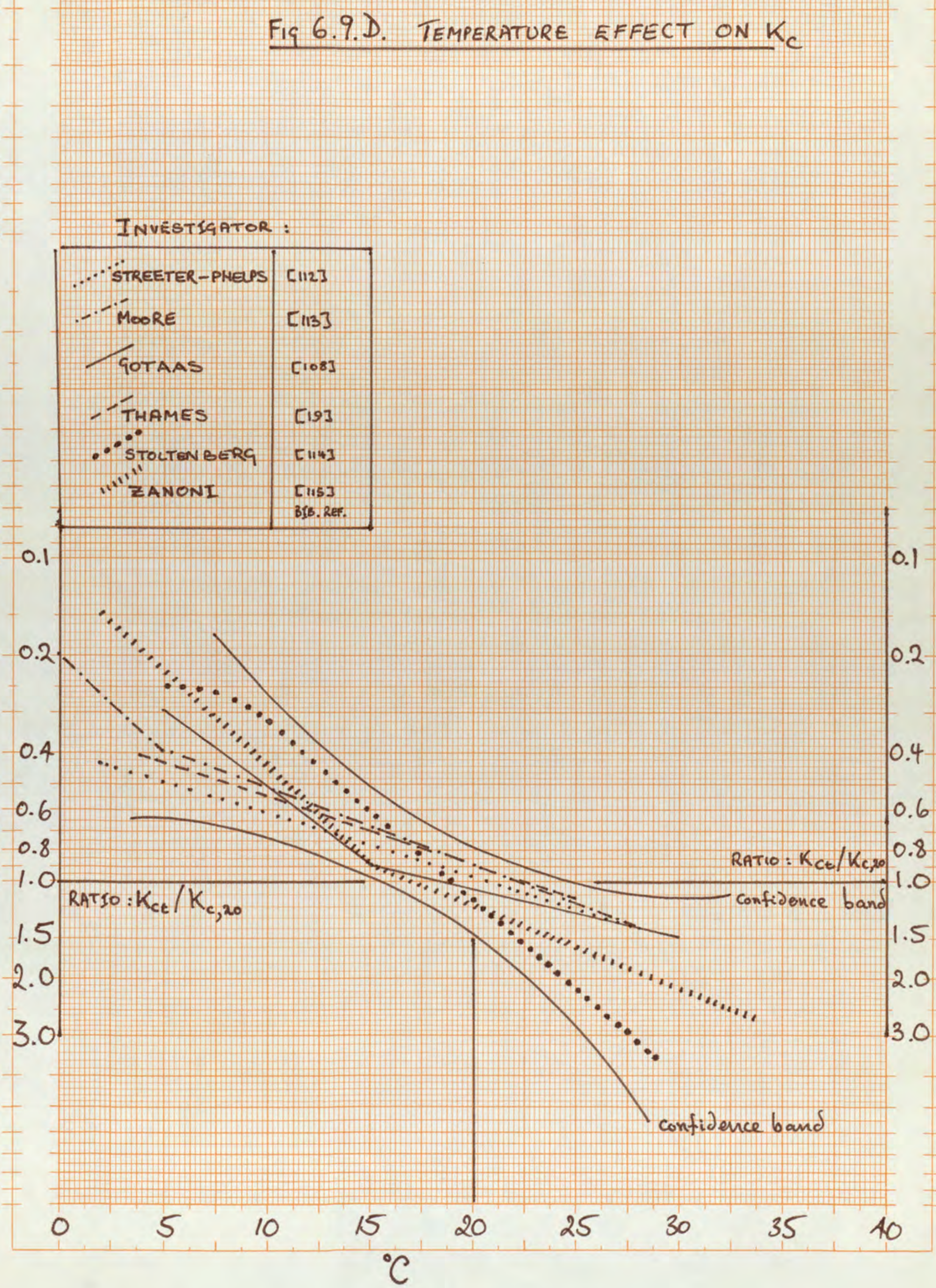
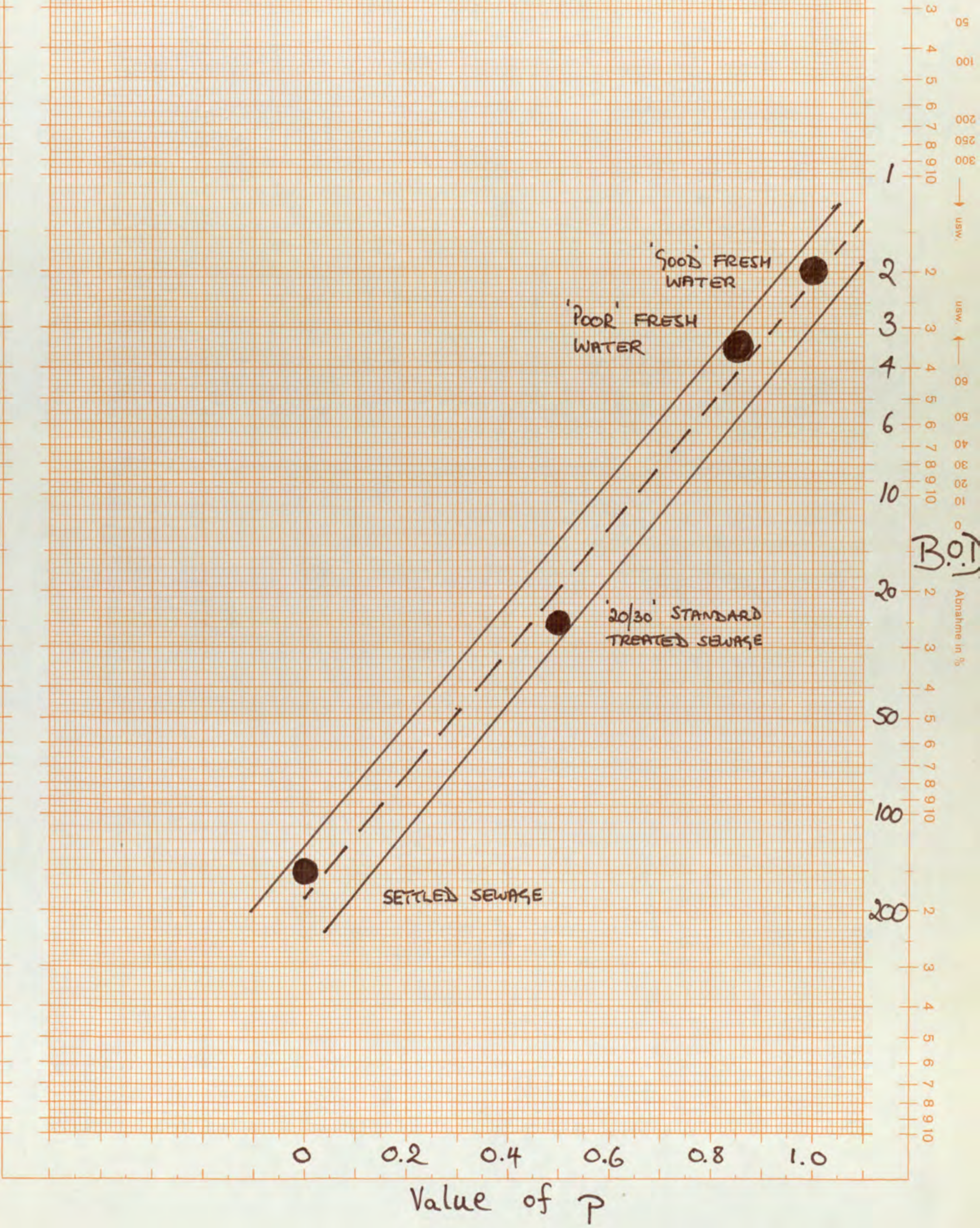


Fig 6.9.D. TEMPERATURE EFFECT ON K_c
INVESTIGATOR :
STREETER-PHELPS [112]
MOORE [113]
GOTAAS [108]
THAMES [19]
STOLTENBERG [114]
ZANONI [115]
BIB. REF.
0.1
0.2
0.4
0.6
0.8
1.0
1.5
2.0
3.0
RATIO : $K_{ct}/K_{c,20}$
confidence band
0
5
10
15
20
25
30
35
40
°C

FIG 6.9.E

SUGGESTED VALUE OF 'p'



6.9.5 Nitrogenous Oxidation

Less precisely established because of the complexity of options, it is nevertheless an important process. A review established uniformity in outline of reactions[118], but less uniformity in the infrastructure.

Generally , when ammonia is present in water containing reasonable levels of dissolved oxygen, it is oxidized via nitrite to nitrate . This nitrate can under low oxygen regimes be used in the oxidation of carbonaeceous components. Oxidation processes are usually predominant, but reduction processes are important under certain conditions[117].

Two very large scale studies have attempted to formulate the precise processes, in the Thames[19] and Delaware[119] Estuaries.In the Thames Study, nitrification was asumed to be a 1st order reaction :

$$\frac{dN}{dt} = -K_n.N \qquad (6.9.5.A)$$

In a segment slice δx of cross-section $A(x)$ and average concentration N. Let $S(\delta)$ be the set of all segments (sliced) of the estuary where nitrification is the dominant process. The net total rate of Utilization U_n is then

$$U_n = K_n \int_{\forall x \in S(\delta)} N.A(x).\delta x \qquad (6.9.5.B)$$

Calculation of K_n for the Thames over 40 quarters gave K_n at 0.1 per day with a standard deviation of 0.03. This is interference due to the phytoplankton growth/decay cycle as there are trends within seasonal quarters.

In the Delaware[119], averages of 0.3 per day were required to fit data to observed data, so it is likely that interferences were again significant. Generally the kinetics are similar to that for the carbonaeceous component and the steady state model uses the same mechanism for both.

6.9.6 Temperature Dependance of K_n

Only scarce data is available , but the relationship

$$K_{nt} = K_{n,20} \cdot \beta^{(t-20)} \qquad (6.9.6.A)$$

where β is 1.017 is postulated in the Thames Study[19,p.219] but is accepted as possibly too small[19,p.503]. Other work confirms the broad outline of 6.9.6.A although the noise level in data from various sources makes validation of the relationship difficult[113][120][121].

6.9.7 Restricted Oxygen Processes

The Steady State Model postulates the following processes at low oxygen levels :

a) oxidation of organic carbon proceeds independantly of the level of O_2

b) for DO > 0.4 mg/l , the rate of nitrification is proportional to the ammonia present.

c) At levels <0.4 mg/l nitrification ceases.

d) At levels <0.4 mg/l nitrate is reduced to N_2 to attempt to maintain the 0.4 mg/l threshold value.

Note : the 0.4 mg/l (or 5% saturation) is relatively flexible[19], but in any event the figure will be less than 10% or 1 mg/l.

6.10 Flow Data

6.10.0 Fresh Water inflow is a valuable source of oxygen and dilution.Its physical volume provides a mechanism for the gradual seaward displacement of pollutant inputs. Tidal Retention variations are largely determined by fresh water inflow levels. Protected flows at certain points are statutory minimum 'hands off' flows and it is important to simulate such extreme conditions. Should it prove possible to reduce , say, the Usk protected flow by only 10%, resource for 25,000 equivalent heads of population is created at no capital cost other than distribution.

The Usk protected flow level influences the operation of the Usk Reservoir -Llandegfedd Rservoir-Lanwern River Regulation System and the Usk-Wye Transfer.

The software used to analyses flow data was FWFANA main routines with the date-time package (Ref. Appendix F).

6.10.1 Definition of the Dry Weather Flow (DWF)

During the project, a reappraisal of the definition of the Dry Weather Flow was in progress. It was agreed that because of the lack of quantitative cohesive records available for the principal rivers and tributary brooks, the protected flow levels set for the tidal Usk were essentially arbitrary[105] Furthermore, there is no one definition of the term DWF and the '7 day mean minimum flow' was proposed[106]:

> "The lowest total discharge occurring over 7 consecutive days in any year expressed as a mean daily flow level ".

Previously , all flow levels were related to a nominal flow of 100 mgd at

Chainbridge on Usk. The above definition redefined the DWF for protection to 90 mgd (table 6.10.A). Thoms and Wain[107] suggested a modified definition of the above employing the median for the same period as opposed to the mean. This has the advantage of not overweighting extreme values within the period. It also offered a statistically useful value which the system can be expected to recede below for any one year with an even probability. This definition when applied to the Usk , would reduce the protected flow further to 83 mgd. Uniformity of definition is an urgent requirement on a national scale.

Table 6.10.A Various Definitions of DWF Applied to Various Rivers

River	DWF	%Ex	DWF	%Ex	DWF	%Ex
Afon L.	0.66 (12.5)	98	0.59 (11.2)	100	0.6 (11.4)	99
Ebbw	1.69 (32.1)	95	1.15 (21.9)	100	1.64 (31.2)	96
Usk	4.72 (89.7)	93	2.35 (44.7)	99	4.39 (83.5)	95

Notes DWF(lhs) - derived 7 day minimum flow
DWF(mid) - min. 7 day flow for duration of flow records
DWF(rhs) - Median flow of 7 day min. flow period
%Ex - Percentage of exceedance times of defined flows
Flows in cumecs and (mgd).
Data sequences vary from 6 to 15 years

6.10.2 Usk Flow Data

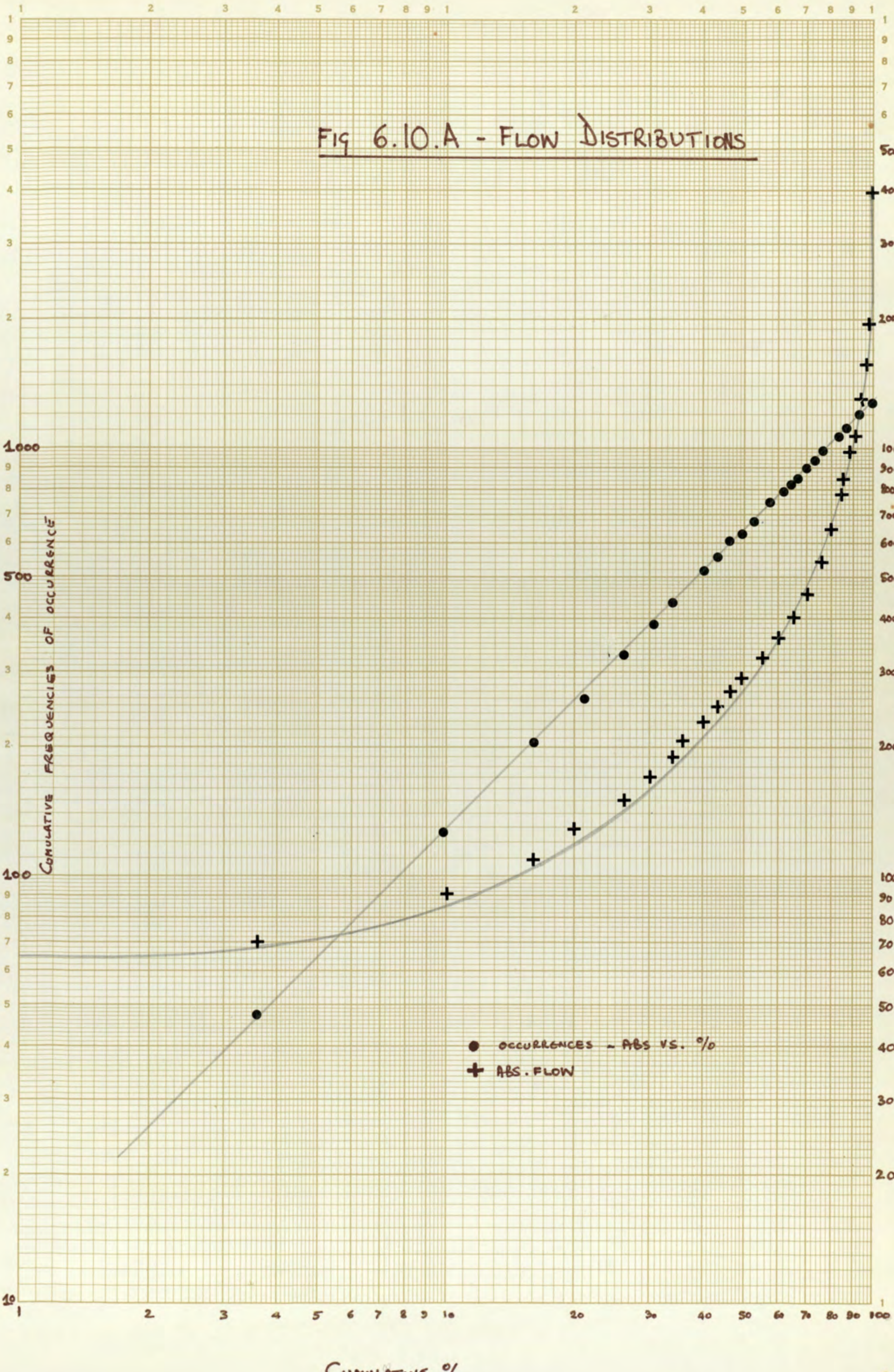
Over three years data was analysed to consider the nature of flow distributions for freshwater input to the Estuary as gauged at Chainbridge on Usk station. Fig. 6.10.A and Table 6.10.B summarise the data in percentile terms.

Table 6.10.B Percentage Daily Flow Levels for R. Usk at Chainbridge on Usk

% flow less than	m.g.d.	cumecs.
1	64.5	3.39
2	65	3.42
3	66	3.47
4	69	3.63
5	71	3.75
6	73.5	3.87
7	76	4.00
8	79.5	4.18
9	82	4.31
10	85	4.47
20	118	6.21
30	158	8.31
40	210	11.05
50	273	15.15
60	358	18.83
70	480	25.25
80	670	35.24
90	1025	53.92
100	4000	210.42

Flow (mgd)	%less than
50	0
75	6.5
100	15
125	22
150	28.3
200	38
250	47

FIG 6.10.A - FLOW DISTRIBUTIONS



Flows were generally unstable for anything like a ± 10 % day to day variation. Fig. 6.10.B and C show that for

± 2% max steady period is 5-6 days

± 4% max steady period is 6-7 days

± 6% max steady period is 8-10 days.

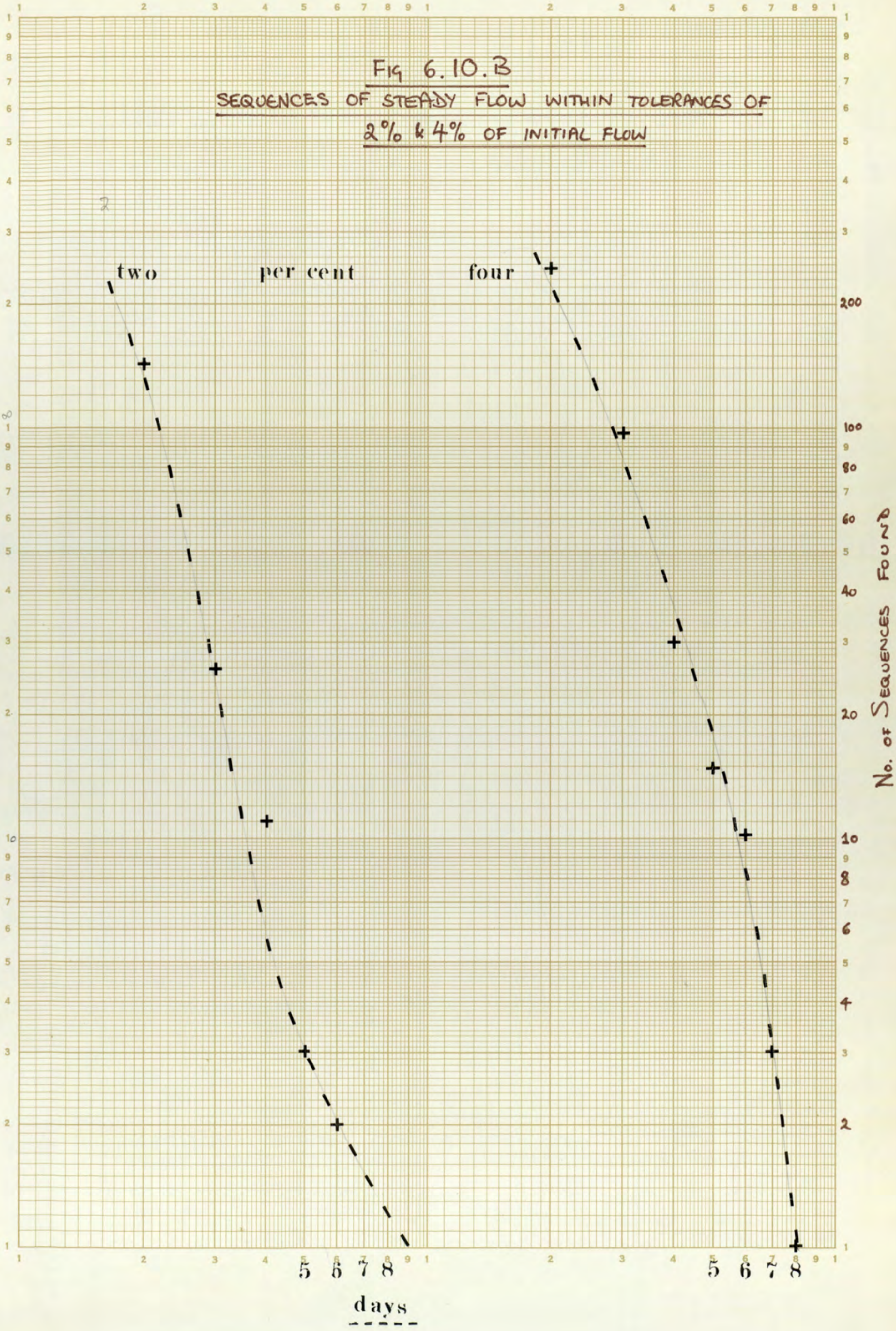
Therefore, fixing the typical time of the steady state model at 20 to 30 days mean, a flow tolerance of ± 20 % is required. The long term average flow appears to be about 470 mgd (25 cumecs).

6.10.3 Flows in Tributaries

There are closely correlated flows in tributaries to the Chainbridge levels. A two year correlative analysis between the Afon Llwyd and the Usk gave significant correlation at the 99% level. The long term average for the Afon Llwyd is around 57 mgd (3 cumecs).

Fig 6.10.B

SEQUENCES OF STEADY FLOW WITHIN TOLERANCES OF 2% & 4% OF INITIAL FLOW



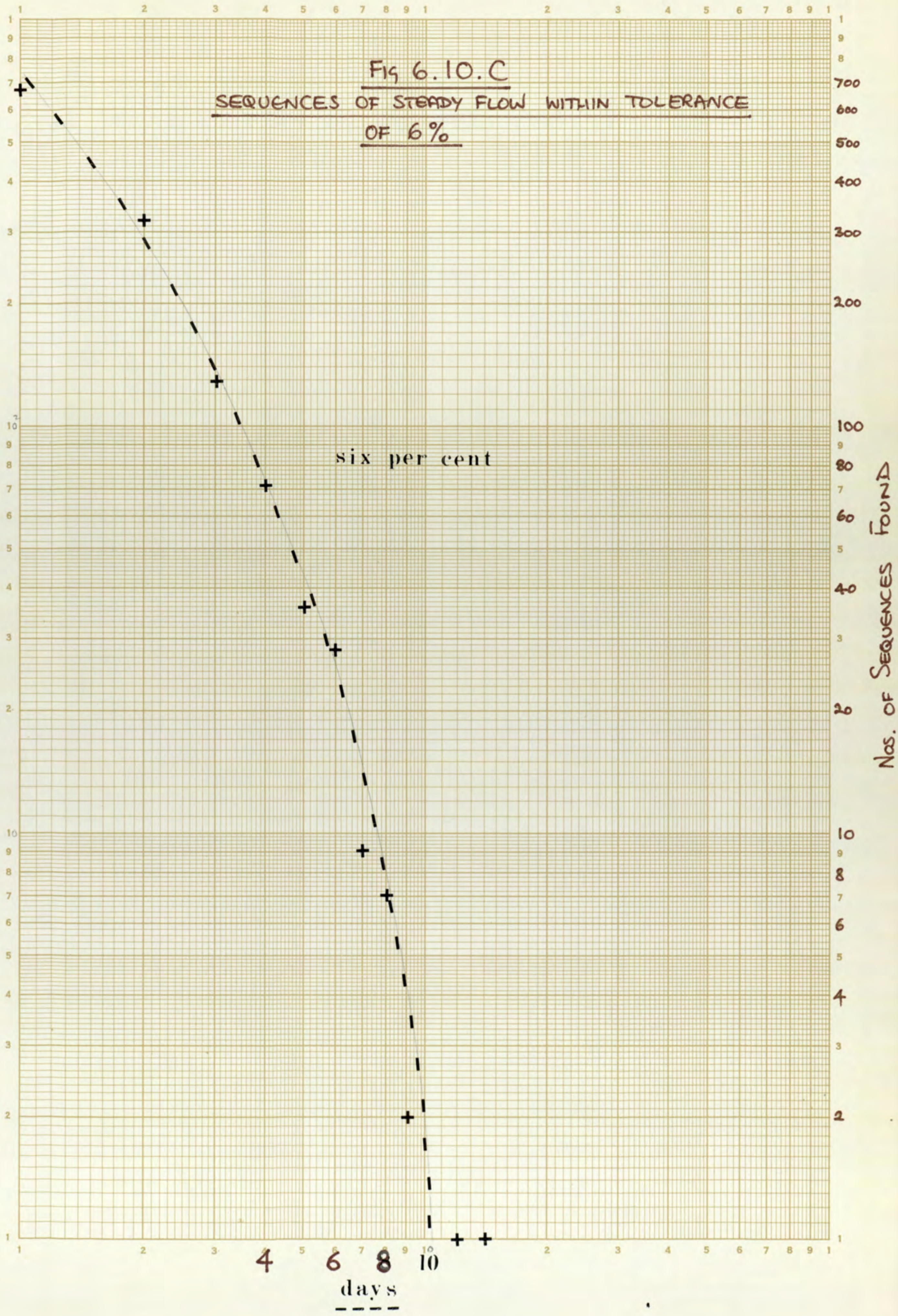


Fig 6.10.C
SEQUENCES OF STEADY FLOW WITHIN TOLERANCE OF 6%
six per cent
700
600
500
400
300
200
100
80
60
40
20
10
8
6
4
2
Nos. of Sequences Found
4
6
8
10
days

6.11 Estuary Fisheries

Table 6.11.A shows species recorded in the Usk Estuary in the period 1965- 1972[108].

Table 6.11.A Recorded Fisheries in the USK Estuary

Species	Name
Petromyzonidae	Marine Lamprey
Clupidae	Allis and Twaite Shad
Salmonidae	Atlantic Salmon (A) Sea Trout Brown Trout
Cottidae	Bullhead (A)
Pleuronectidae	Flounder

Note : (A) - especially abundant.

In order to maintain and improve fisheries available, oxygen levels must be maintained above certain limits (see 6.12). Total fishery value passing through or otherwise dependant on the Estuary is estimated at upwards of £2M[108].

Toxic effects are important also but are too diverse a subject to be discussed here . Obviously specific industrial effluents need to be considered closely at time of consent garnting with a view to toxic effects.

6.12 Minimum Dissolved Oxygen Requirements for Migratory Fish

The Pippard Report[122] quotes the minimum requirement for oxygen content at >30% saturation during the period April to May, nine years out of any 10 consequtive years.

Although much data is available for fresh water fish requirements,for a critical review see [123] , little actual work on estuaries is available. An experiment was planned for the Usk Estuary in conjunction with the Water Research Centre. The migratory fish were to be implanted with radio transmitters and then tracked.During periods of low oxygen, it was hoped to determine whether a fish would attempt to progress through the sag (and so probably die) or whether it would learn to await an improvement before negotiating a sag. The answer obtained would have obvious implications on the severity of future consent standards. Due to financial economies the experiment was cancelled, although similar work is scheduled for the R. Tyne.The answer is important in this estuary. On the whole, requirements vary widely depending on species and life-cycle stage[124][125][126]. Other conditions are assumed to be non-limiting factors in experiments.

If thermal limits are near, much higher levels are required to maintain most species. The R. Don suffered a heavy mortality with DO > 4 mg/l because average temperatures were in excess of 22^{o}C[127].

Fecundity and Embryonic Development is also affected by low levels of DO.

An analysis of past records of the Usk show that during the period 1951 to 1970 there has been a dramatic decline in the DO profile of the Estuary.In this period the mean level has fallen from 80% to 50% , and the minimum level from 55% to 5%. Yet there is no ancilliary trend in the salmon catches recorded[129][130][131][132]. However, minimums more recently proposed were not accepted by local management[132][133].

Alabaster [132] proposes the following percentile standards :

50 per centile levels to be > 9 mg/l
5 per centile levels to be > 5 mg/l

These would require major improvements in the Usk Estuary system.

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Chapter 7

Applying the Steady State Model

Chapter 7

Contents

7.0 Introduction

The Steady State Model was the first model made available to the Pollution Control Department Management. It satisfied their basic requirements and consequently those of the Welsh Office[2].Use consisted of series of ad hoc enquiries relating received consent requests or effects of outline plans. Specific industrial effluents are protected by law from disclosure to protect the plant method and products.

Most parameters used were reasonable well established. The whole model was geared towards eventual unsupported use and so running variants was a simple task that the manager was capable of. After a sensitivity analysis, some variables were extracted and a simple regressive type desk top equation was developed to give a 'real time' model facility.[13].

Section 6,7 highlights the diversity of opinion on dispersion and diffusion. Being a one dimensional model, only D_x need be considered.The model treats both as a lumped effect, being a continuous process of exchange between adjacent segments of equal and opposing flows[3][4]. This allows a definition of a mixing echange coefficient as :

$$F(x) = 2.\ D(x).A(x) \ / \ \Delta x \qquad (7.0.A)$$

Δx is the distance between adjacent segment boundaries, A(x) the cross-sectional area function and D(x) the dispersion coefficient function.

Alternatively, it was postulated that 7.0.A can be written as

$$F_i = Q_i.(S_i - S_0) \ / \ (S_{i+1} - S_i) \qquad (7.0.B)$$

where S_i is the mean salinity in segment i and Q_i the sum of fresh water inflows from the head of the system to the previous segment. This expression was validated against a physical model[5] and found to be satisfactory for time dependant and steady state versions of this model.

Data was obtained from previous sources itemised in chapter 6 as well as other internal sources [8][10].

7.1 The Steady State Definition

'Steady' state implies constancy. In an estuarine situation with a widely varying source of inputs and perturbations, a true steady state will never be reasonable established. However, generally it is acceptable to allow transients to pass through the system provided they do not skew the major state variables too much. Also, the term pseudo-steady state is permitted to mean that Steady State that would be established if the conditions currently constant were to remain so for a period exceeding the retention time of the system.

When inspecting data , some criteria of 'steadiness' has to be adopted for sake of consistency. If, for a set of data , a mean value x is the long term mean (ie the steady state mean), the values scattered about it will normally be distributed according to the normal distribution.The probability of any data value falling within the limits x_1 and x_2 are ($x_2 > x_1$)

$$P(x_1 \leq x \leq x_2) = \int_{x_1}^{x_2} \frac{1}{\sigma\sqrt{2\pi}} \cdot \exp\left[-(x-\mu)^2 / 2.\sigma^2\right] . dx \qquad (7.1.A)$$

where σ is the std. dev. and μ the mean .

Alternatively, specifying a mean probability P and an actual mean value x_m , limits can be calculated to satisfy the percentile requirements. The length of the steady state is then the length of time the data remains within the bands specified. Certain transients effects may be permitted by relaxation of the standard (eg the powerstation effect).

A third method is to calculate moving averages over a variable number of points. The definition would then be the time span for which the data sequence would remain within predetermined percentage fluctuations of the moving mean within the period. This method was adopted for this study and used in routine FWFANA.

There are some situations where the concept requires careful interpretation of is without meaning.

In biological systems, often a small change will trigger a whole series of events that may lead to wholly different effects. This would not be easily incorporated into a steady state. There is no way of accounting for irreversible steps as they are inescapably time dependant. In a system, a small change in flow may trigger a flow regulating system, or movement may be artifical through air-bubbles or jetting, sluices may alter or open or close. These would not normally be documented and it is also rare for this type of external agency to be active sufficiently long to set up its own steady state. The multiplicity of these systems now makes data analysis for the application of one of the foregoing definitions more difficult .

All the above difficulties are reflected in the task of data collection for the eventual validation of the model. Dealing with steady states implies a longer period of data collection than the maximum tidal excursion. The implied use of manpower and other resource is beyond many smaller units of the water industry and this is where the impending reorganisation should provide a benefit, with the creation of data collection teams.

7.2 Standard Data Input Set for the Steady State Model

Low water 3.5m, high water 12.4m. Half tide state at 5.4m (in terms of time through the tidal cycle). Horizontal Tide Profile, poor tidal excursion data and smoothed temperature distribution.

Segm -ent	From	to	Volumes	Surface Areas	Salinity
1	0	.5	.01	.032	0
2	.5	1	.011	.03	0
3	1	1.5	.012	.033	0
4	1.5	2	.009	.032	0
5	2	2.5	.01	.028	0
6	2.5	3	.007	.023	0
7	3	3.5	.009	.025	0
8	3.5	4	.009	.03	0
9	4	4.5	.009	.034	0
10	4.5	5	.012	.037	0
11	5	5.5	.012	.043	.025
12	5.5	6	.015	.04	.075
13	6	6.5	.015	.045	.125
14	6.5	7	.017	.04	.175
15	7	7.5	.02	.04	.3
16	7.5	8	.013	.045	.7
17	8	8.5	.015	.046	1.45
18	8.5	9	.013	.044	2.95
19	9	9.5	.025	.059	4.85
20	9.5	10	.032	.062	6.35
21	10	10.5	.041	.067	7.6
22	10.5	11	.079	.07	8.8
23	11	11.5	.087	.09	11.5
24	11.5	12	.133	.09	13.55
25	12	12.5	.16	.09	15.6
26	12.5	13	.3	.13	17.3
27	13	13.5	.38	.2	18.65
28	13.5	14	.42	.22	20.15
29	14	14.5	.56	.22	21.55
30	14.5	15	.86	.18	22.45
31	15	15.5	1.4	.2	23.1
32	15.5	16	1.6	.2	23.55
33	16	16.5	1.95	.22	23.85
34	16.5	17	2.3	.24	24.15

7.3 Tidal Excursion Data

Ordinate	0	5.	11.5	12.0	13.0	14.0	15.0	16.0
Downstream	11.5	11.5	4.75	4.5	4.3	3.7	3.45	3.0
Upstream	0	1.65	1.9	2.1	2.0	1.9	1.1	1.5

Note : All distances in miles. Zero is Newbridge on Usk.

7.4 Boundary Conditions

Parameter	Upstream Value	Downstream Value
Slow Carbon.	.45	.05
Fast Cardon.	.05	.01
Slow Nitrog.	.45	.05
Fast Nitrog.	.05	.01
Ammonia	.50	.04
Nitrate	.10	.05
D.O.	8.5	7.5
FWF	83mgd	-

Values obtained from existing URD sample programme. The flow is obtained by the use of the definition of Thoms & Wain.

7.5 Discharges to the System

Name	Dist.	Flow	Tidelocked ?
Sor Brook	6.85	1.0	Tributary
Eastern Valley	7.2	5.0	Major discharge, not tidelocked.
Beaufort	8.81	0.1	Yes
Caerleon	9.1	.2	No
St.Julian	9.61	.22	Yes
Orchard	10.81	.10	Yes
Brynglas & Bettws	11.08	.75	Yes
Cenotaph North	11.15	.15	Yes
Afon Llwyd	7.16	11.4	Tributary, dilution source for E. Valley
Riverside	11.22	.02	Yes, very small
Cenotaph South	11.41	.33	Yes
Barrack	11.45	.11	Yes
Civic	11.53	.27	Yes
Town	11.72	.23	Yes
Maindee	12.18	.55	Yes
Pill North 2	12.33	.405	Yes
County	12.70	1.115	Yes
Ringland	12.82	.6	Yes
Pill North 1	12.92	.405	Yes
Tredegar Dock	13.43	.58	Yes
Pill South	13.54	.45	Yes
Coronation Park	13.9	.5	Yes
R. Ebbw	16.05	31.	Major Tributary

7.6 Input Loadings to the Steady State Model

	Fast Carbon	Slow Carbon	Fast Nitrog.	Slow Nitrog.	Ammonia	Nitrate
Coronation Park	32	22	28	14	44	2.5
Pill South	572	0	70	0	243	0
Tredegar Dock	744	0	91	0	337	0
Pill North 1	522	0	63	0	48	0
Ringland	704	176	103	0	351	0
Pill North 2	522	0	65	48	48	0
Maindee	664	0	86	0	296	2
Town	300	0	36	0	130	0
Civic	444	0	42	0	152	0
Barrack	173	0	20	0	68	0
Cenotaph South	368	0	46	0	174	2
Riverside	26	0	4	0	11	0
Cenotaph North	196	0	24	0	82	1
Brynglas & Bettws	1508	0	161	0	580	5
Orchard	130	0	16	0	49	0
St Julian	288	0	35	0	123	0
Caerleon	82	82	12	12	44	1
Beaufort	130	0	16	0	43	1
Eastern Valleys	1304	1304	242	242	1760	50
Ebbw	1500	1500	610	610	1000	28
Afon Llwyd	650	650	240	240	400	24
Sor Brook	10	10	1	1	6	1
County	1302	0	165	0	575	8

```
00000100 5.4" TIDE HEIGHT ,H.W. & 12.4" . DYE BY VAINS METHOD,NO BGC DISCHARGE     TITLE1
00000200 .                                                                          TITLE2
00000300 .                                                                          TITLE3
00000400 .                                                                          TITLE4
00000500      34          NUMBER OF SEGMENTS                                        NSEG
00000600      20          HISTORY OF CONVERGENCE FLAG                               IPRINT
00000700     500          MAXIMUM NUMBER OF ITERATIONS FOR CONVERGENCE              MAXLOO
00000800       2          NUMBER OF INCREMENTS OF REAERATION COEFFICIENTS           NODELS
00000900      21          NUMBER OF OUTFALLS IN ESTUARY                             NOUTF
00001000      11          NUMBER OF READINGS FOR TIDAL EXCURSION                    NOL
00001100     1.2          ACCELERATION COEFFICIENT FOR NITROGENOUS PHASE            OMN
00001200     1.3          ACCELERATION COEFFICIENT FOR AMMONIA PHASE                OMAM
00001300     1.5          ACCELERATION COEFFICIENT FOR D.O. PHASE                   OMD
00001400     1.0          ACCELERATION COEFFICIENT FOR DE-NITRIFICATION             OMW
00001500      .4          DO LEVEL BELOW WHICH DENITRIFICATION SETS IN              DOXMIN
00001600    62.3          FINAL MULTIPLIER FOR VOLUMES                              FCTRL
00001700    35.3          MULTIPLIER FOR VOLUMES TO 10E6CU.FT.                      VACTOR
00001800    10.7          MULTIPLIER FOR SURFACE AREAS TO 10E6SQ.FT.                FACTSA
00001900     10.          MULTIPLIER FOR FRESH WATER FLOWS                          FACTOR
00002000     83.          INITIAL F.W.F. IN M.G.D.                                  FLOWS
00002100    183.          FINAL MAXIMUM F.W.F. IN M.G.D.                            FLWMAX
00002200     50.          INCREMENT IN F.W.F. IN M.G.D.                             FLWINC
00002300    1.02          FACTOR FOR RUN-OFF SUBROUTINE                             RUNFAC
00002400    .183          CARBONACEOUS RATE CONSTANT                                RKC
00002500    .183          NITROGENOUS RATE CONSTANT                                 RKN
00002600    .260          AMMONICAL RATE CONSTANT                                   RKAMM
00002700    .004          NITRATE RATE CONSTANT                                     RKNO3
00002800    2.86          OXYGEN EQUIVALENT OF NITRATE                              COXNO3
00002900    1.00          AMMONIA EQUIVALENT FOR NITROGEN                           AN
00003000    1.00          NITRATE EQUIVALENT FOR AMMONIA                            ANO3A
00003100    4.57          OXYGEN EQUIVALENT FOR AMMONIA                             OXA
00003200    .001          MAXIMUM ERROR ALLOWED IN ITERATIVE PROCESS                ERROR
00003300      2.          INCREMENT IN REAERATION RATES                             DELRE
00003400     0.4          UPSTREAM FAST CARBONACEOUS LOAD IN PPM                    FC(1)
00003500     0.08         DOWNSTREAM FAST CARBONACEOUS LOAD IN PPM                  FC END
00003600     0.6          UPSTREAM SLOW CARBONACEOUS LOAD IN PPM                    SC(1)
00003700     0.2          DOWNSTREAM SLOW CARBONACEOUS LOAD IN PPM                  SC END
00003800     0.4          UPSTREAM FAST NITROGENOUS LOAD IN PPM                     FN(1)
00003900     0.08         DOWNSTREAM FAST NITROGENOUS LOAD IN PPM                   FN END
00004000     0.5          UPSTREAM SLOW NITROGENOUS LOAD IN PPM                     SN(1)
00004100     0.2          DOWNSTREAM SLOW NITROGENOUS LOAD IN PPM                   SN END
00004200     1.0          UPSTREAM AMMONIA LOAD IN PPM                              AMM(1)
00004300     0.05         DOWNSTREAM AMMONIA LOAD IN PPM                            AMMEND
00004400     0.1          UPSTREAM NITRATE LOAD IN PPM                              ANO3(1)
00004500     0.1          DOWNSTREAM NITRATE LOAD IN PPM                            ANO3END
00004600    8.51          UPSTREAM DISSOLVED OXYGEN LEVEL IN PPM                    DOX(1)
00004700    7.46          DOWNSTREAM DISSOLVED OXYGEN LEVEL IN PPM                  DOXEND
00004800       FFFFFFFF     FF      FFFFFFF FFFFF        + SMOOTH ARRAY (F OR T)
00004900 TTTTTTTTTTTTTTTTTFFTTTTTTTTTFTFTFTTFTFTTTTFFFTTT  + PRINTING ARRAY(F OR T)T
00005000 0  .5  1.   1.5   2   2.5  3   3.5  4  4.5  5       +SEGMENT BOUNDARIES A
00005100 5.5 6  6.5  7    7.5  8   8.5  9   9.5 10.  10.5    +SEGMENT BOUNDARIES B
00005200 11 11.5  12.  12.5  13  13.5  14  14.5  15  15.5    +SEGMENT BOUNDARIES C
00005300 16  16.5  17  17.5                                  +SEGMENT BOUNDARIES D
00005400 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5            +REAERATION PER SEGMENT FT/DAY
00005500 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5            +REAERATION B
00005600 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5            +REAERATION C
00005700 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5            +REAERATION D
00005800 .010  .011   .012   .009   .010   .007   .009   .009  +VOLUME PER SEGMENT
00005900 .012  .011   .012   .015   .015   .017   .020   .013  +VOLUME B
00006000 .015  .013   .025   .032   .041   .079   .087   .133  +VOLUME C
```

FIG 7.6A (i)

COMPUTER FILE OF MODEL INPUT DATA

```
00006100 .16  .3   .33  .42  .56  .86  1.4  1.6    *VOLUME D
00006200 1.75 2.3                                   *VOLUME E
00006300 .032 .030 .033 .032 .028 .023 .025 .030 .034   *SURFACE AREAS PER SEGMENT  A
00006400 .037 .043 .040 .045 .04 .04 .045 .046 .044     *SURFACE AREAS PER SEGMENT  B
00006500 .059 .062 .067 .07 .09 .09 .09 .13 .2 .22 .22 *SURFACE AREAS PER SEGMENT  C
00006600 .18 .2 .2 .22 .24                              *SURFACE AREAS PER SEGMENT  D
00006700 0 0 0 0 0 0 0 0 0 0 .02 .07 .12 .17 .3 .7 1.45 *SALINITIES PER SEG IN PPT  A
00006800 2.95 4.85 6.35 7.6 8.8 11.5 13.55 15.6 17.3 18.65 *SALINITIES PER SEG IN PPT  B
00006900 20.15 21.55 22.45 23.1 23.55 23.85 24.15 24.45 *SALINITIES PER SEG IN PPT  C
00007000 24.75                                          *SALINITIES PER SEG IN PPT  D
00007100 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. *TEMPERATURE PER SEG IN C  A
00007200 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. *TEMPERATURE PER SEG IN C  B
00007300 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. 20. *TEMPERATURE PER SEG IN C  C
00007400 0. 5. 11.45 12. 12.5 13. 14. 14.5 15. 16. 18.  *TIDAL EXCURSION ORDINATES  A
00007500 11.5 11.5 4.75 4.5 4.3 3.7 3.45 3.2 3 3 3     *DOWNSTRM EXCURSIONS  A
00007600 0. 0. 1.65 1.9 1.85 2.1 2.0 1.9 1.7 1.5 1.5   *UPSTREAM EXCURSIONS  A
00007700 TTTTTTTTTTTTTTTTTTFFFFF                        *TIDELOCKED? LOGICAL ARRAY T/F
00007800  2                                             * MODE OF INPUT (USUALLY #2)
00007900 CORC      #A 13.90 0.50 32.0 22 28 14 44 2.5 20 0/FALL1
00008000 PILL S    #B 13.54 .45 570. 0 70 0 240 0 20     0/DALL2
00008100 TRED DD   #C 13.43 .58 740 0 100 340 0 20       0/FALL3
00008200 PILL N 1#D 12.92 .4 300 0 60 0 50 0 20          0/FALL4
00008300 RING      #E 12.82 .6 700 200. 100 0 1350 0 20  0/FALL5
00008400 PILL N 2#F 12.33 .4 530 0 100 0 50 0 20         0/FALL6
00008500 MAINDE    #G 12.18 .55 660 0 100 0 300 2 20     0/FALL7
00008600. TOWN     #H 11.72 .23 300 0 40 0 130 0 20      0/FALL8
00008700 CIVIC     #I 11.53 .27 450 0 40 0 150 0 20      0/FALL9
00008800 BRRCK     #J 11.45 .11 170 0 20 0 70 0 20       0/FALL10
00008900 CENO S    #K 11.41 .33 370 0 50 0 175 2 20      0/FALL11
00009000 RIVER     #L 11.22 .02 30 0 5 0 10 0 20         0/FALL12
00009100 CENO N    #M 11.15 .15 200 0 25 0 80 1 20       0/FALL13
00009200 BRYNBETT#N 11.08 .75 1510 0 160. 0 600 5 20     0/FALL14
00009300 ORCH      #O 10.81 .1 130 0 20 0 50 0 20       0/FALL15
00009400 ST JULIA#P 9.61 .22 300 0 40 0 120 0 20         0/FALL16
00009500 CAER      #Q 9.1 .2 80. 80. 15. 15. 40 1. 20.   0/FALL17
00009600 BEAU      #R 8.81 .1 130. 0 20. 0 40. 1. 20.    0/FALL18
00009700 E VALL    #S 7.2 5 1300 1300 250 250 1800 50 20 0/FALL19
00009800 EBBW      #T 16.1 .31 1000 1000 600 600 600 30 90 0/FALL20
00009900 COUNTY    #U 12.7 1.2 1300 0 170. 0 600 7.6 20. 0/FALL21
```

FIG 7.6.A (ii)

COMPUTER FILE OF MODEL INPUT DATA

7.7 The Desk Top Steady State Model[13]

Although the Steady State Model itself is not data intensive, it was thought useful if the whole model could be further reduced. The model sensitivity was analysed for condensation to a desk top form. No attempt has been made to relate any significant correlations in terms of physical interpretations , as these relationships are necessarily model induced.

The standard data set for simulating the estuary using the Steady State Model consists of 3 tributaries and some 20 pollutant discharges. For the purpose of this investigation , it was assumed that the effects of pollutant loads could be computed by superposition. Therefore for the generation of predictions, the estuary system was simplified to the initial tributaries (as these provide useful dilution and so could affect extreme levels) and one mobile discharge point.

From sensitivity tests on the whole model, the following were found to be primary influencing factors for the value Dissolved Oxygen (minimum) and its occurrent position;

a. The type of pollutant discharged and its total load

b. The point of discharge of the pollutant of a.

c. Ambient conditions that determine re-aeration rates.

d. Fresh Water Flows to the head of the system.

All other physical and chemical parameters were set to constants to represent average 'poor' conditions. Flow was set constant to DWF level This mean the following 8 parameters were independant variables :

a. Fast Carbon Load

b. Slow Carbon Load

c. Fast Nitrogen Load

d. Slow Nitrogen Load

e. Ammonia Load

f. Nitrate Load

g. Re-aeration rates

h. Point of Discharge from head of system.

For all runs, the volumes discharged were set to 1mgd, representing a consistent 1% dilution on DWF flow levels. About 1050 simulations were run with some systematic variation of all 8 parameters and some random data values. The output from these runs was analysed using a regression package with transgeneration facility(see App. F.7).

Standard Statistical Tables[9] gave the following percentage points for levels of rejection of the independance hypothesis for about 1000 degrees of freedom :

Percentage Point :	5%	1%	.1%
Critical Value of coefficient(t)	1.96	2.58	3.29

This meant that a correlation coefficient in excess of 0.09891 (say 0.1) indicates a slightly greater than 99% probability of being significant. The correlation matrix is given in Table 6.

The main point of interest is that where the D.O. min occurs if a significant load is entered is always in a reach from mile 11 to mile 13, within passing through the main town of Newport.No regression was attempted for this dependant variable in the light of this fact.

The three dependant variables mainly considered were D.O.min and its log transform , and the Sag Severity Index (SSI, Appendix A).

Table 7.7.A Significant Correlation Coefficients for Parameters in the Steady State Model

Variable Id.	a	b	c	d	e	f	g	h	i	j	k	l	m	n	o
Max DO def a*	1														
SSI b*	.97	1													
DO Present c*	-.98	-.99	1												
DO Absent d*	.74	.75	-.75	1											
Point DOm e*	-.1	-.13	.14	-	1										
FC Load f	.44	.35	-.36	.28	-	1									
SC Load g	.29	.26	-.27	.24	-	-	1								
FN Load h	.48	.41	-.43	.34	-	.19	.19	1							
SN Load i	.44	.40	-.40	.35	-	-	.56	.22	1						
NH3 Load j	.75	.68	-.69	.51	-	-	.28	.22	.46	1					
NO3 Load k	.22	.22	-.22	.15	-	-	-	-	.12	.34	1				
Re-Aeratn. l	-.2	-.25	.28	-.17	-	-	-	-	-	-	-	1			
Pt of Dischm	-.2	-.28	.29	-.2	.24	-	-	-	-	-	-	-	1		
NO3*Pt.Dis n	-	-	-	-	-	-	-	-	.11	.3	.83	-	.28	1	
log(DOmin) o*	.77	.70	-.76	.57	-	.27	.28	.44	.36	.62	.17	-.32	-.2	-	1

Note : - appears where computed correlation coefficients are not significant.

* signifies a dependant variable.

variables n and o were generated internally by the program.

7.7.1 D.O. Minimum as the Dependant Variable

Variable d being dependant, variables f to o are independant.

From statistical tables[9], a Student 't' value of 3.36 is very significant, at the 1% level. The regression gave :

Multiple Correlation Coefficient	0.9145
Students 't' value	7.9
Regression Constant	13.43

The individual components gave :

Variable	Coefficient	Error	't' value	Significant
FC Load	.3	.009	34.8	Yes,highly
SC Load	.1	.023	4.3	Just
FN LOad	.45	.019	23.0	Yes,highly
SN Load	.09	.033	2.7	No
NH3 Load	.03	.022	1.4	No
Re-aeratn	-2.56	.14	-18.2	Yes,highly
Position	-0.72	.029	-24.4	Yes,highly

The error in the above table is the standard error of the correlation regression equation. One point of interest is the ratio of FC/SC and FN/SN coefficients. Because of the model formulation these should be about 5 (as for the nitrogenous BOD load component) , but for the carbonaceous load this is nearer 3 . This indicates a lack of assumed linearity in the superposition principle of load addition. It must be emphasised that the variable D.O.min(%) is now the deficit in D.O. from 100% constant value in the system. This was found to be of more use. The above table allowed the following equation to summarise the predictions of the Steady State Model :

$$\begin{array}{l}\text{D.O.min(\%)}\\ \text{(deficit)}\end{array} = 13.4 + (3*FC + SC)/_{10} + (FN + {}_{6}*NH3)/10 - 2.6*REAER - 0.7 * DISCH \qquad (6.$$

where REAER= Re-aeration rate and DISCH- point of discharge

The equation dealt well with average variations , but , as could be expected, gave poor estimates if an isolated load for input was greatly in excess of the others (1.5 orders of magnitude or greater).

7.7.2 The Sag Severity Index as the Dependant Variable

Only a slightly lower multiple regression coefficient resulted from this run, but because of more degrees of freedom, the net regression was more significant. Abstracting from the output as for the previous section gave the following predictive equation :

SSI = 0.137 + 0.0013 * (FC + 9 * NH3 + 2 * NO3 - 14 * REAER - 3 * DISCH)
+ 0.002 * FN - 0.003 * (NO3 * DISCH)

A run was attempted to predict the natural log transform of the DOmin% and SSI but neither gave improved predictions. The effects of load increases can be seen in uniform signs on the coefficients , and the negative signs on re-aeration and position showing these as opposing moderating influences.

Because the tributaries did not contribute pure dilution water, but of a quality only slightly lower than river head waters, there remains a natural background DO deficiency. This is reflected by the equation coefficient in each case. there is a 13.4% natural sag , of overall SSI of 0.137 . If the cumulative effects of several discharges is to be evaluated, then the constants are ignored until the individual effects are summed. The natural deficiency is then added to give the estimated true deficiency for a multpile input situation. All loads are input in lbs per day , re-aeration in ft/day and position in miles (or any consistent set of units)

7.8 <u>The Convergence Acceleration Parameters</u>

These are required for the interacting phase of the solution. Three basic values are required , for ammonia, nitrate and dissolved oxygen. Initially, setting all values to unity will ensure convergence although up to 1000-1500 iterations may be required. Resetting of these parameters is best an acquired skill, values usually found to be acceptable range from 1.05 to 1.45 depending on the severity of the variant. There are advantages to selecting an approximate value which will handle most variants without setting. Too fine a tune on the parameter will cause divergence of the method for all but the data set for which the parameter has been calculated.

A fourth parameter is defined in the program, OMW. This is only used in anaerobic situations and is usually left at unity as the section of the program should only be involved occasionally.

An estimate of the parameter value can be calculated from the expression [6][7]

$$\alpha_{opt} \simeq \frac{2}{\left[\sqrt{1-\beta^2} + 1\right]} \qquad (7.8.A)$$

where α_{opt} is the optimal choice of the parameter, and β is the largest normalized eigenvalue of the iteration matrix (ie the spectral radius) This is calculated Using routine EIGEN (Appendix F). The effort to compute this is often far in excess of the cost of several testing runs to search for a reasonable set of values. Each component requires a different acceleration parameter to be developed.

7.9 Basic Output of the Steady State Model.

The data set calculated represents the situation in the estuary in 1973 with no discharge from the British Glue Co. Ltd. in Newport, which contributed a high percentage of the load prior to the factory closure. The tide is a 12.4m Newport tide and the Thoms-Wain Method is selected for defining the DWF[11]. The point of time selected in the tidal cycle is when the flooding tide is at 5.4m , roughly 40% into the tidal cycle. The essential data is outlined in previous sections. Fig. 7.9.A summarises routine parameters. The convergence coefficients used in this case are

$$\alpha_{NH_3} = 1.3$$

$$\alpha_{NO_3} = 1.2$$

$$\alpha_{DO} = 1.5$$

The routine has two possible internal loops, the inner loop automatically re-running part simulations for varying re-aeration, the outer repeating the whole simulation for incrementing freshwater inflow from the head of the system.

The data presented will simulate:

83, 133 and 183 mgd at re-aeration 1.5 and 2.5 per day.

Fig. 7.9.B and C itemise some of the output of basic input data. All the initial printing can be supressed if desired. It is seen that the tidal excursion data is sparse at a point of great interest. Fig. 7.9.D shows the significant effect of excursion on segment surface areas. This was a recent incorporation into the model[12]. The overall effect is to improve

the D.O. profile as the segments around the Eastern Valleys outfall benefit most.

Fig. 7.9.E (in 5 parts) show the breakdown of each outfall into loads per segment. Each segment is given a weight, depending on how much of the outfall the segment 'sees'. Non zero values are printed. Tributaries are considered as additional outfalls (e.g. OUT fall 20).

Fig. 7.9.F summarises the input loads of each outfall to the system. Normally this table is sufficient for output. The loads are then split according to the computed weights to yield a distribution as in Fig. 7.9.G. Loads are in lbs per day.

Fig. 7.9.H outputs the sum freshwater flow at each segment, and the computed mixing exchange coefficient as defined by section 7.0.

The initial dissolved oxygen distribution is printed. (D.O.INIT.) The nearness of this to the final solution greatly influences the iterations for convergence. Using a zero estimate other than for boundaries, 969 iterations were required. Using a linear slope between the boundaries reduced this to 103. Accuracy required is 0.001.

Fig.7.9.I lists the rate arrays and mixing arrays. These are influenced by temperature and freshwater inflow respectively.

Fig. 7.9.J and K plot the history of convergence. The accuracy defined as 0.001 is the sum total of the residuals over all segments and over all the interacting components. The difference between the two rates of convergence is due to the finer tuning of the a cceleration parameters. Fig. 7.9.K is

10 times more accurate for only 42% of iterations required by 7.9.J.

Fig. 7.9.L is the table prediction of the model proper. The run is for 83 mgd and re-aeration of 1.5 per day. The first six columns are FC, SC, FN, SN, NH3, NO3 and as they are not measured on estuarine samples, they are of no interest at the moment. However they combine to generate the dissolved oxygen predictions (col.7 and 8 in mg/l and % s at.). The Column 9 headed 'U' is non zero only when restricted oxidation of ammonia occurs. The 10th column headed 'W' is non zero when denitrification becomes a predominating process. Positive values in these columns occur when the D.O. level falls through 0.4 mg/l.

Fig. 7.9.M compares a predicted curve against observed data. The agreement in the upper reaches is to be expected as there are no discharges in the initial 7.2 miles. For the remaining section, observed and predicted remain within 8%, the observed lower. It should be noted that the simulation was prior to the data collection, and so temperature will not correspond ($20^{o}C$ was modelled, when $21^{o}C$ to $23.2^{o}C$ were recorded). Also flow levels were 16% lower than those simulated. There was no adjustment of any parameter to encourage improved correlation and the prediction is a mean of all such states over a lunar tidal cycle. There is no data available for calibration of the other distributions.

Little validation data is available in the form of tide-cycle duration. Monitor data is difficult to use because it has to be repositioned and in the absence of good velocity data is completely impractical.

The model then uses the prediction to initiate the next run (internally the same data but an incremented re-aeration). This means subsequent simulations become extremely cost-effective, with as few as 4 iterations required to generate the new predictions (depending on the severity of the change).

Figure 7.9.N summarises the predictions of the minimum D.O. and the total sag severity index for the six simulations. It is seen that flow is an excellent preventative for low mean D.O. levels, implying that high protected flow levels may be a cheap effective solution to the problem of pollution control in relative (concentration) terms.

```
*****************************  DATA SET NUMBER     1

                               5.4M TIDE HEIGHT ,H.W. @ 12.4M . DWF BY WAINS METHOD.NO BGC DISCHARGE      TITLE1
                               *                                                                          TITLE2
                               *                                                                          TITLE3
                               *                                                                          TITLE4

SEGMENTS  34    PRINT FLAG20     LOOP LIM 500   NO.OF REAR  2    OUTFALLS  21    TIDE EXC  11

OM NITRATE           1.2000   OM AMM             1.3000
OM D.O.              1.5000   OM W.              1.0000
D.O. MIN             0.4000   FCTRL             62.3000
VACTOR              35.3000   FACTSA            10.7000
FACTOR              10.0000   F.W.FLOW          83.0000
RATE CONSTANTS; C,N,NH3,NO3 =      0.18300   0.18300    0.26000    0.00400
EQUIVALENTS ; COXNO3,AN,ANO3,OXA   2.86000   1.00000    1.00000    4.57000
ITERATIVE ACCURACY               0.001000
INCREMENT IN REAERATION COEFF      2.0000

BOUNDARY VALUE TABLES

 UPSTREAM    *   DOWNSTREAM

     0.400           0.080   FAST CARBON
     0.600           0.200   SLOW CARBON
     0.400           0.080   FAST N
     0.300           0.200   SLOW N
     1.000           0.050   NH3
     0.100           0.100   NO3
     8.510           7.460   D.O.

SMOOTHING = F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F F   Smooth Input data?
 PRINTING = T T T T T T T T T T T T T T T T T T T T T T T T T T T T T T T F F T F T T T T T T F F F T T T F F F   Tracer levels
```

Run time parameters

(ppm)

FIG. 7.9.A

TITLE & RUN PARAMETERS

FLOW LIMITS

BOUNDARIES

FIG 7.9.8

PHYSICAL PARAMETERS

```
FLOWS MODELLED ARE FOR  83.000 INCR BY 50.0  TO MAX OF   183.00

ARRAY DISTANCE .ELEMENT    1  TO   35
( 1 )=  0.00000E 00  ( 2 )=  5.00000E-01  ( 3 )=  1.00000E 00  ( 4 )=  1.50000E 00  ( 5 )=  2.00000E 00  ( 6 )=  2.50000E 00
( 7 )=  3.00000E 00  ( 8 )=  3.50000E 00  ( 9 )=  4.00000E 00  ( 10 )=  4.50000E 00  ( 11 )=  5.00000E 00  ( 12 )=  5.50000E 00
( 13 )=  6.00000E 00  ( 14 )=  6.50000E 00  ( 15 )=  7.00000E 00  ( 16 )=  7.50000E 00  ( 17 )=  8.00000E 00  ( 18 )=  8.50000E 00
( 19 )=  9.00000E 00  ( 20 )=  9.50000E 00  ( 21 )=  1.00000E 01  ( 22 )=  1.05000E 01  ( 23 )=  1.10000E 01  ( 24 )=  1.15000E 01
( 25 )=  1.20000E 01  ( 26 )=  1.25000E 01  ( 27 )=  1.30000E 01  ( 28 )=  1.35000E 01  ( 29 )=  1.40000E 01  ( 30 )=  1.45000E 01
( 31 )=  1.50000E 01  ( 32 )=  1.55000E 01  ( 33 )=  1.60000E 01  ( 34 )=  1.65000E 01  ( 35 )=  1.70000E 01  (

ARRAY REAERTN. .ELEMENT    2  TO   35
ARRAY IS SINGLE VALUED AT   1.50000E 00

ARRAY VOLUMES. .ELEMENT    2  TO   35
( 2 )=  1.00000E-02  ( 3 )=  1.10000E-02  ( 4 )=  1.20000E-02  ( 5 )=  9.00000E-03  ( 6 )=  1.00000E-02  ( 7 )=  7.00000E-03
( 8 )=  9.00000E-03  ( 9 )=  9.00000E-03  ( 10 )=  1.20000E-02  ( 11 )=  1.10000E-02  ( 12 )=  1.20000E-02  ( 13 )=  1.50000E-02
( 14 )=  1.50000E-02  ( 15 )=  1.70000E-02  ( 16 )=  2.00000E-02  ( 17 )=  1.30000E-02  ( 18 )=  1.50000E-02  ( 19 )=  1.30000E-02
( 20 )=  2.50000E-02  ( 21 )=  3.20000E-02  ( 22 )=  4.10000E-02  ( 23 )=  7.90000E-02  ( 24 )=  8.70000E-02  ( 25 )=  1.33000E-01
( 26 )=  1.60000E-01  ( 27 )=  3.00000E-01  ( 28 )=  3.80000E-01  ( 29 )=  4.20000E-01  ( 30 )=  5.60000E-01  ( 31 )=  8.60000E-01
( 32 )=  1.40000E 00  ( 33 )=  1.60000E 00  ( 34 )=  1.95000E 00  ( 35 )=  2.30000E 00  (

ARRAY SURF.ARS .ELEMENT    2  TO   35
( 2 )=  3.20000E-02  ( 3 )=  3.00000E-02  ( 4 )=  3.30000E-02  ( 5 )=  3.20000E-02  ( 6 )=  2.80000E-02  ( 7 )=  2.30000E-02
( 8 )=  2.50000E-02  ( 9 )=  3.00000E-02  ( 10 )=  3.40000E-02  ( 11 )=  3.70000E-02  ( 12 )=  4.30000E-02  ( 13 )=  4.00000E-02
( 14 )=  4.50000E-02  ( 15 )=  4.00000E-02  ( 16 )=  4.00000E-02  ( 17 )=  4.50000E-02  ( 18 )=  4.60000E-02  ( 19 )=  4.40000E-02
( 20 )=  5.90000E-02  ( 21 )=  6.20000E-02  ( 22 )=  6.70000E-02  ( 23 )=  7.00000E-02  ( 24 )=  9.00000E-02  ( 25 )=  9.00000E-02
( 26 )=  9.00000E-02  ( 27 )=  1.30000E-01  ( 28 )=  2.00000E-01  ( 29 )=  2.20000E-01  ( 30 )=  2.20000E-01  ( 31 )=  1.80000E-01
( 32 )=  2.00000E-01  ( 33 )=  2.00000E-01  ( 34 )=  2.20000E-01  ( 35 )=  2.40000E-01  (

ARRAY CONV VOL .ELEMENT    2  TO   35
( 2 )=  2.19919E 01  ( 3 )=  2.41911E 01  ( 4 )=  2.63903E 01  ( 5 )=  1.97927E 01  ( 6 )=  2.19919E 01  ( 7 )=  1.53943E 01
( 8 )=  1.97927E 01  ( 9 )=  1.97927E 01  ( 10 )=  2.63903E 01  ( 11 )=  2.41911E 01  ( 12 )=  2.63903E 01  ( 13 )=  3.29878E 01
( 14 )=  3.29878E 01  ( 15 )=  3.73862E 01  ( 16 )=  4.39838E 01  ( 17 )=  2.85895E 01  ( 18 )=  3.29878E 01  ( 19 )=  2.85895E 01
( 20 )=  5.49798E 01  ( 21 )=  7.03741E 01  ( 22 )=  9.01668E 01  ( 23 )=  1.73736E 02  ( 24 )=  1.91330E 02  ( 25 )=  2.92492E 02
( 26 )=  3.51870E 02  ( 27 )=  6.59757E 02  ( 28 )=  8.35692E 02  ( 29 )=  9.23660E 02  ( 30 )=  1.23155E 03  ( 31 )=  1.89130E 03
( 32 )=  3.07886E 03  ( 33 )=  3.51870E 03  ( 34 )=  4.28842E 03  ( 35 )=  5.05814E 03  (

ARRAY CONV SA. .ELEMENT    2  TO   35
( 2 )=  3.42400E-01  ( 3 )=  3.21000E-01  ( 4 )=  3.53100E-01  ( 5 )=  3.42400E-01  ( 6 )=  2.99600E-01  ( 7 )=  2.46100E-01
( 8 )=  2.67500E-01  ( 9 )=  3.21000E-01  ( 10 )=  3.63800E-01  ( 11 )=  3.95900E-01  ( 12 )=  4.60100E-01  ( 13 )=  4.28000E-01
( 14 )=  4.81500E-01  ( 15 )=  4.28000E-01  ( 16 )=  4.28000E-01  ( 17 )=  4.81500E-01  ( 18 )=  4.92200E-01  ( 19 )=  4.70800E-01
( 20 )=  6.31300E-01  ( 21 )=  6.63400E-01  ( 22 )=  7.16900E-01  ( 23 )=  7.49000E-01  ( 24 )=  9.63000E-01  ( 25 )=  9.63000E-01
( 26 )=  9.63000E-01  ( 27 )=  1.39100E 00  ( 28 )=  2.14000E 00  ( 29 )=  2.35400E 00  ( 30 )=  2.35400E 00  ( 31 )=  1.92600E 00
( 32 )=  2.14000E 00  ( 33 )=  2.14000E 00  ( 34 )=  2.35400E 00  ( 35 )=  2.56800E 00  (
```

VOLUMES

SURFACE AREAS

CONVERTED VOLUMES

CONVERTED SURFACE AREAS

```
ARRAY DEPTHS.. .ELEMENT    2  TO   35
 (  2 )=  1.03096E 00   (  3 )=  1.20966E 00   (  4 )=  1.19966E 00   (  5 )=  9.27862E-01   (  6 )=  1.17824E 00   (  7 )=  1.00406E 00
 (  8 )=  1.18766E 00   (  9 )=  9.89720E-01   ( 10 )=  1.16438E 00   ( 11 )=  9.80803E-01   ( 12 )=  9.20669E-01   ( 13 )=  1.23715E 00
 ( 14 )=  1.09969E 00   ( 15 )=  1.40210E 00   ( 16 )=  1.64953E 00   ( 17 )=  9.53063E-01   ( 18 )=  1.07578E 00   ( 19 )=  9.74724E-01
 ( 20 )=  1.39791E 00   ( 21 )=  1.70274E 00   ( 22 )=  2.01683E 00   ( 23 )=  3.72323E 00   ( 24 )=  3.18910E 00   ( 25 )=  4.87529E 00
 ( 26 )=  5.86501E 00   ( 27 )=  7.61323E 00   ( 28 )=  6.26823E 00   ( 29 )=  6.29822E 00   ( 30 )=  8.39762E 00   ( 31 )=  1.57622E 01
 ( 32 )=  2.30934E 01   ( 33 )=  2.63925E 01   ( 34 )=  2.92417E 01   ( 35 )=  3.16160E 01   (

ARRAY SALINITY .ELEMENT    1  TO   36
 (  1 )=  0.00000E 00   (  2 )=  0.00000E 00   (  3 )=  0.00000E 00   (  4 )=  0.00000E 00   (  5 )=  0.00000E 00   (  6 )=  0.00000E 00
 (  7 )=  0.00000E 00   (  8 )=  0.00000E 00   (  9 )=  0.00000E 00   ( 10 )=  0.00000E 00   ( 11 )=  2.00000E-02   ( 12 )=  7.00000E-02
 ( 13 )=  1.20000E-01   ( 14 )=  1.70000E-01   ( 15 )=  3.00000E-01   ( 16 )=  7.00000E-01   ( 17 )=  1.45000E 00   ( 18 )=  2.95000E 00
 ( 19 )=  4.85000E 00   ( 20 )=  6.35000E 00   ( 21 )=  7.60000E 00   ( 22 )=  8.80000E 00   ( 23 )=  1.15000E 01   ( 24 )=  1.35500E 01
 ( 25 )=  1.56000E 01   ( 26 )=  1.73000E 01   ( 27 )=  1.86500E 01   ( 28 )=  2.01500E 01   ( 29 )=  2.15500E 01   ( 30 )=  2.24500E 01
 ( 31 )=  2.31000E 01   ( 32 )=  2.35500E 01   ( 33 )=  2.38500E 01   ( 34 )=  2.41500E 01   ( 35 )=  2.44500E 01   ( 36 )=  2.47500E 01

ARRAY TEMPERTR .ELEMENT    2  TO   35
ARRAY IS SINGLE VALUED AT  2.00000E 01

ARRAY T.EX ORD .ELEMENT    1  TO   11
 (  1 )=  0.00000E 00   (  2 )=  5.00000E 00   (  3 )=  1.14500E 01   (  4 )=  1.20000E 01   (  5 )=  1.25000E 01   (  6 )=  1.30000E 01
 (  7 )=  1.40000E 01   (  8 )=  1.45000E 01   (  9 )=  1.50000E 01   ( 10 )=  1.60000E 01   ( 11 )=  1.80000E 01   (

ARRAY DOWNSTRM .ELEMENT    1  TO   11
 (  1 )=  1.15000E 01   (  2 )=  1.15000E 01   (  3 )=  4.75000E 00   (  4 )=  4.50000E 00   (  5 )=  4.30000E 00   (  6 )=  3.70000E 00
 (  7 )=  3.45000E 00   (  8 )=  3.20000E 00   (  9 )=  3.00000E 00   ( 10 )=  3.00000E 00   ( 11 )=  3.00000E 00   (

ARRAY UPSTREAM .ELEMENT    1  TO   11
 (  1 )=  0.00000E 00   (  2 )=  0.00000E 00   (  3 )=  1.65000E 00   (  4 )=  1.90000E 00   (  5 )=  1.85000E 00   (  6 )=  2.10000E 00
 (  7 )=  2.00000E 00   (  8 )=  1.90000E 00   (  9 )=  1.70000E 00   ( 10 )=  1.50000E 00   ( 11 )=  1.50000E 00   (
```

DEPTHS

SALINITIES

TEMPERATURE

TIDAL EXCURSION

DOWNSTREAM

UPSTREAM

FIG 7.9.C

REMAINING PHYSICAL PARAMETERS

Fig 7.9.D Modification of the Surface Area through Tidal Excursion of the Segments

TIDAL EXCURSION OF SEGEMNTED SURFACE REDISTRIBUTED SURFACE AREAS.

I.SEG	DIST	UP	DOWN.DB	SEGMNT	OLD VAL	NEW VAL
2	0.250	0.250	11.750	2	0.3424	1.3603
3	0.750	0.750	12.250	3	0.3210	1.3459
4	1.250	1.250	12.750	4	0.3531	1.3219
5	1.750	1.750	13.250	5	0.3424	1.2905
6	2.250	2.250	13.750	6	0.2996	1.2719
7	2.750	2.750	14.250	7	0.2461	1.2600
8	3.250	3.250	14.750	8	0.2675	1.2582
9	3.750	3.750	15.250	9	0.3210	1.2407
10	4.250	4.250	15.750	10	0.3638	1.2258
11	4.750	4.750	16.250	11	0.3959	1.2047
12	5.250	5.225	16.511	12	0.4601	1.1504
13	5.750	5.666	16.495	13	0.4280	1.2957
14	6.250	6.096	16.440	14	0.4815	1.3256
15	6.750	6.514	16.356	15	0.4280	1.3528
16	7.250	6.920	16.254	16	0.4280	1.3821
17	7.750	7.314	16.146	17	0.4815	1.4126
18	8.250	7.694	16.041	18	0.4922	1.4445
19	8.750	8.062	15.952	19	0.4708	1.6294
20	9.250	8.415	15.889	20	0.6313	1.6789
21	9.750	8.755	15.863	21	0.6634	1.7347
22	10.250	9.080	15.885	22	0.7169	1.7945
23	10.750	9.391	15.966	23	0.7490	1.8538
24	11.250	9.686	16.117	24	0.9630	1.7366
25	11.750	9.928	16.360	25	0.9630	1.8003
26	12.250	10.373	16.674	26	0.9630	1.7080
27	12.750	10.789	16.766	27	1.3910	1.7878
28	13.250	11.123	16.825	28	2.1400	1.8521
29	13.750	11.689	17.237	29	2.3540	1.8860
30	14.250	12.291	17.573	30	2.3540	1.8706
31	14.750	12.945	17.840	31	1.9260	1.8519
32	15.250	13.622	18.250	32	2.1400	1.6488
33	15.750	14.221	18.750	33	2.1400	1.3736
34	16.250	14.750	19.250	34	2.3540	1.1294
35	16.750	15.250	19.750	35	2.5680	0.9036

```
OUTFALL   CORO      AT=13.90 MLS,FLOW= 0.500 M.G.D
LOADS=FC   32.00 SC=   22.00 FN=   28.00 SN=   14.00 NH3=   44.00 NO3=   2.50 D.O.=   8.04

SEGMENT 25  HAS WGHT =0.0214
SEGMENT 26  HAS WGHT =0.0909
SEGMENT 27  HAS WGHT =0.0909
SEGMENT 28  HAS WGHT =0.0909
SEGMENT 29  HAS WGHT =0.7060
UB + DB=   11.3821 + 17.3845  TOTAL EX=      5.502
OUFALL SEGMENT= 29  TIDELOCKED ?= T
OUTFALL  1  SUM OF WEIGHTS =1.000
```

OUT 1

```
OUTFALL   PILL S    AT=13.54 MLS,FLOW= 0.450 M.G.D
LOADS=FC  570.00 SC=    0.00 FN=   70.00 SN=    0.00 NH3=  240.00 NO3=   0.00 D.O.=   7.24
```

L O A D S

```
SEGMENT 24  HAS WGHT =0.0069
SEGMENT 25  HAS WGHT =0.0886
SEGMENT 26  HAS WGHT =0.0886
SEGMENT 27  HAS WGHT =0.0886
SEGMENT 28  HAS WGHT =0.0886
SEGMENT 29  HAS WGHT =0.6386
UB + DB=   11.4608 + 17.1039  TOTAL EX=      5.043
OUFALL SEGMENT= 29  TIDELOCKED ?= T
OUTFALL  2  SUM OF WEIGHTS =1.000
```

WEIGHTING FACTORS OF LOADS MET BY EACH SEGMENT

OUT2

TIDAL EXCURSION OF OUTFALL

```
OUTFALL   TRED DD   AT=13.43 MLS,FLOW= 0.580 M.G.D
LOADS=FC  740.00 SC=    0.00 FN=  100.00 SN=    0.00 NH3=  340.00 NO3=   0.00 D.O.=   9.41
```

→ (ESTIMATED FLOW)

```
SEGMENT 24  HAS WGHT =0.0289
SEGMENT 25  HAS WGHT =0.0881
SEGMENT 26  HAS WGHT =0.0881
SEGMENT 27  HAS WGHT =0.0881
SEGMENT 28  HAS WGHT =0.7067
UB + DB=   11.3357 + 17.0105  TOTAL EX=      5.675
OUFALL SEGMENT= 28  TIDELOCKED ?= T
OUTFALL  3  SUM OF WEIGHTS =1.000
```

OUT3

```
OUTFALL   PILL N 1  AT=12.92 MLS,FLOW= 0.400 M.G.D
LOADS=FC  500.00 SC=    0.00 FN=   60.00 SN=    0.00 NH3=   50.00 NO3=   0.00 D.O.=   6.55

SEGMENT 23  HAS WGHT =0.0246
SEGMENT 24  HAS WGHT =0.0855
SEGMENT 25  HAS WGHT =0.0855
SEGMENT 26  HAS WGHT =0.0855
SEGMENT 27  HAS WGHT =0.7189
UB + DB=   10.3558 + 16.7058  TOTAL EX=      5.850
OUFALL SEGMENT= 27  TIDELOCKED ?= T
OUTFALL  4  SUM OF WEIGHTS =1.000
```

OUT 4

FIG 7.9.E

DISCHARGES TO THE SYSTEM

(i) 1 → 4

FIG 7.9.E

(ii) 5 -> 8

```
OUTFALL   RING      AT=12.22 MLS,FLOW= 0.600 M.G.D
LOADS=FC  700.00 SC=  200.00 FN=  100.00 SN=    0.00 NH3= 1350.00 NO3=    0.00 D.O.=    9.82

SEGMENT 23  HAS WGHT =0.0328
SEGMENT 24  HAS WGHT =0.0845
SEGMENT 25  HAS WGHT =0.0845
SEGMENT 26  HAS WGHT =0.0845
SEGMENT 27  HAS WGHT =0.7137
UB + DB=   10.8061 + 16.7233  TOTAL EX=      5.917
OUFALL SEGMENT= 27  TIDELOCKED ?= T
OUTFALL  5  SUM OF WEIGHTS =1.000
```

OUT5

```
OUTFALL   PILL N 2  AT=12.33 MLS,FLOW= 0.400 M.G.D
LOADS=FC  530.00 SC=    0.00 FN=  100.00 SN=    0.00 NH3=   50.00 NO3=    0.00 D.O.=    6.60

SEGMENT 22  HAS WGHT =0.0024
SEGMENT 23  HAS WGHT =0.0800
SEGMENT 24  HAS WGHT =0.0800
SEGMENT 25  HAS WGHT =0.0800
SEGMENT 26  HAS WGHT =0.7573
UB + DB=   10.4852 + 16.7386  TOTAL EX=      6.253
OUFALL SEGMENT= 26  TIDELOCKED ?= T
OUTFALL  6  SUM OF WEIGHTS =1.000
```

OUT6

```
OUTFALL   MAINDE    AT=12.18 MLS,FLOW= 0.550 M.G.D
LOADS=FC  660.00 SC=    0.00 FN=  100.00 SN=    0.00 NH3=  300.00 NO3=    2.00 D.O.=    9.07

SEGMENT 22  HAS WGHT =0.0293
SEGMENT 23  HAS WGHT =0.0790
SEGMENT 24  HAS WGHT =0.0790
SEGMENT 25  HAS WGHT =0.0790
SEGMENT 26  HAS WGHT =0.7338
UB + DB=   10.3148 + 16.6452  TOTAL EX=      6.330
OUFALL SEGMENT= 26  TIDELOCKED ?= T
OUTFALL  7  SUM OF WEIGHTS =1.000
```

OUT 7

```
OUTFALL   TOWN      AT=11.72 MLS,FLOW= 0.230 M.G.D
LOADS=FC  300.00 SC=    0.00 FN=   40.00 SN=    0.00 NH3=  130.00 NO3=    0.00 D.O.=    3.83

SEGMENT 21  HAS WGHT =0.0198
SEGMENT 22  HAS WGHT =0.0773
SEGMENT 23  HAS WGHT =0.0773
SEGMENT 24  HAS WGHT =0.0773
SEGMENT 25  HAS WGHT =0.7484
UB + DB=    9.8720 + 16.3433  TOTAL EX=      6.471
OUFALL SEGMENT= 25  TIDELOCKED ?= T
OUTFALL  8  SUM OF WEIGHTS =1.000
```

OUT 8

FIG 7.9.E

(iii) 9 ⇒ 12

```
OUTFALL   CIVIC      AT=11.33 MLS,FLOW= 0.270 M.G.D
LOADS=FC  450.00 SC=     0.00 FN=   40.00 SN=     0.00 NH3=  150.00 NO3=    0.00 D.O.=    4.50

SEGMENT 21  HAS WGHT =0.0307
SEGMENT 22  HAS WGHT =0.0777
SEGMENT 23  HAS WGHT =0.0777
SEGMENT 24  HAS WGHT =0.0777
SEGMENT 25  HAS WGHT =0.7364
UB + DB=     9.8026 + 16.2417   TOTAL EX=       6.439
OUFALL SEGMENT= 25  TIDELOCKED ?= T
OUTFALL  9  SUM OF WEIGHTS =1.000

OUTFALL   BRRCK      AT=11.45 MLS,FLOW= 0.110 M.G.D
LOADS=FC  170.00 SC=     0.00 FN=   20.00 SN=     0.00 NH3=   70.00 NO3=    0.00 D.O.=    1.85

SEGMENT 21  HAS WGHT =0.0312
SEGMENT 22  HAS WGHT =0.0781
SEGMENT 23  HAS WGHT =0.0781
SEGMENT 24  HAS WGHT =0.8125
UB + DB=     9.8000 + 16.2000   TOTAL EX=       6.400
OUFALL SEGMENT= 24  TIDELOCKED ?= T
OUTFALL 10  SUM OF WEIGHTS =1.000

OUTFALL   CENO S     AT=11.41 MLS,FLOW= 0.330 M.G.D
LOADS=FC  370.00 SC=     0.00 FN=   50.00 SN=     0.00 NH3=  175.00 NO3=    2.00 D.O.=    5.56

SEGMENT 21  HAS WGHT =0.0347
SEGMENT 22  HAS WGHT =0.0781
SEGMENT 23  HAS WGHT =0.0781
SEGMENT 24  HAS WGHT =0.8091
UB + DB=     9.7774 + 16.1824   TOTAL EX=       6.405
OUFALL SEGMENT= 24  TIDELOCKED ?= T
OUTFALL 11  SUM OF WEIGHTS =1.000

OUTFALL   RIVER      AT=11.22 MLS,FLOW= 0.020 M.G.D
LOADS=FC   30.00 SC=     0.00 FN=    5.00 SN=     0.00 NH3=   10.00 NO3=    0.00 D.O.=    0.34

SEGMENT 21  HAS WGHT =0.0516
SEGMENT 22  HAS WGHT =0.0779
SEGMENT 23  HAS WGHT =0.0779
SEGMENT 24  HAS WGHT =0.7926
UB + DB=     9.6639 + 16.0870   TOTAL EX=       6.418
OUFALL SEGMENT= 24  TIDELOCKED ?= T
OUTFALL 12  SUM OF WEIGHTS =1.000
```

OUT 9

OUT 10

OUT 11

OUT 12

FIG 7.9.E

(iv) 13→18

```
OUTFALL   CEMO N     AT=11.15 MLS,FLOW= 1.150 M.G.D
LOADS=FC  200.00 SC=    0.00 FN=   25.00 SN=    0.00 NH3=   30.00 NO3=    1.00 D.O.=    2.53

SEGMENT 21  HAS WGHT =0.0578
SEGMENT 22  HAS WGHT =0.0778
SEGMENT 23  HAS WGHT =0.0778
SEGMENT 24  HAS WGHT =0.7865
UB + DB=     9.6283 + 16.0543  TOTAL EX=       6.426
OUFALL SEGMENT= 24  TIDELOCKED ?= T
OUTFALL 13  SUM OF WEIGHTS =1.000
```

OUT 13

```
OUTFALL   BRYNBETT  AT=11.03 MLS,FLOW= 0.750 M.G.D
LOADS=FC 1510.00 SC=    0.00 FN=  160.00 SN=    0.00 NH3=  600.00 NO3=    5.00 D.O.=   12.64

SEGMENT 21  HAS WGHT =0.0641
SEGMENT 22  HAS WGHT =0.0777
SEGMENT 23  HAS WGHT =0.0777
SEGMENT 24  HAS WGHT =0.7805
UB + DB=     9.5375 + 16.0224  TOTAL EX=       6.435
OUFALL SEGMENT= 24  TIDELOCKED ?= T
OUTFALL 14  SUM OF WEIGHTS =1.000
```

OUT 14

```
OUTFALL   ORCH      AT=10.81 MLS,FLOW= 0.100 M.G.D
LOADS=FC  130.00 SC=    0.00 FN=   20.00 SN=    0.00 NH3=   50.00 NO3=    0.00 D.O.=    1.71

SEGMENT 21  HAS WGHT =0.0463
SEGMENT 22  HAS WGHT =0.0807
SEGMENT 23  HAS WGHT =0.8729
UB + DB=     9.7129 + 15.9074  TOTAL EX=       6.195
OUFALL SEGMENT= 23  TIDELOCKED ?= T
OUTFALL 15  SUM OF WEIGHTS =1.000
```

OUT 15

```
OUTFALL   ST JULIA  AT= 9.61 MLS,FLOW= 0.220 M.G.D
LOADS=FC  300.00 SC=    0.00 FN=   40.00 SN=    0.00 NH3=  120.00 NO3=    0.00 D.O.=    3.83

SEGMENT 21  HAS WGHT =0.0304
UB + DB=     9.8257 + 15.5692  TOTAL EX=       5.743
OUFALL SEGMENT= 21  TIDELOCKED ?= T
OUTFALL 16  SUM OF WEIGHTS =0.030
```

OUT 16

```
OUTFALL   CAER      AT= 9.10 MLS,FLOW= 0.200 M.G.D
LOADS=FC   80.00 SC=   80.00 FN=   15.00 SN=   15.00 NH3=   40.00 NO3=    1.00 D.O.=    3.51

SEGMENT 21  HAS WGHT =1.0000
UB + DB=     9.8303 + 15.5175  TOTAL EX=       5.687
OUFALL SEGMENT= 20  TIDELOCKED ?= T
OUTFALL 17  SUM OF WEIGHTS =1.000
```

OUT 17

```
OUTFALL   BEAU      AT= 8.81 MLS,FLOW= 0.100 M.G.D
LOADS=FC  130.00 SC=    0.00 FN=   20.00 SN=    0.00 NH3=   40.00 NO3=    1.00 D.O.=    1.77

SEGMENT 21  HAS WGHT =0.0360
UB + DB=     9.7942 + 15.5127  TOTAL EX=       5.719
OUFALL SEGMENT= 19  TIDELOCKED ?= T
OUTFALL 18  SUM OF WEIGHTS =0.036
```

OUT 18

Fig 7.9.E

(v) 19 → 21

```
OUTFALL    E VALL      AT= 7.20 MLS,FLOW= 5.000 M.G.D
LOADS=FC 1300.00 SC= 1300.00 FN=  250.00 SN=  250.00 NH3= 1300.00 NO3=   50.00 D.O.=   90.46

SEGMENT 19  HAS WGHT =0.0271
SEGMENT 20  HAS WGHT =0.0718
SEGMENT 21  HAS WGHT =0.0718
SEGMENT 22  HAS WGHT =0.0718
SEGMENT 23  HAS WGHT =0.0718
SEGMENT 24  HAS WGHT =0.0718
SEGMENT 25  HAS WGHT =0.0718
SEGMENT 26  HAS WGHT =0.0718
SEGMENT 27  HAS WGHT =0.0718
SEGMENT 28  HAS WGHT =0.0718
SEGMENT 29  HAS WGHT =0.0718
SEGMENT 30  HAS WGHT =0.0718
SEGMENT 31  HAS WGHT =0.0718
SEGMENT 32  HAS WGHT =0.0718
SEGMENT 33  HAS WGHT =0.0393
UB + DB=    8.8113 + 15.7735  TOTAL EX=      6.962
OUFALL SEGMENT= 16  TIDELOCKED ?= F
OUTFALL 19  SUM OF WEIGHTS =1.000
```

OUT 19

```
OUTFALL    EBBW       AT=16.10 MLS,FLOW=31.000 M.G.D
LOADS=FC 1000.00 SC= 1000.00 FN=  600.00 SN=  600.00 NH3=  600.00 NO3=   30.00 D.O.= 2209.23

SEGMENT 31  HAS WGHT =0.0889
SEGMENT 32  HAS WGHT =0.1111
SEGMENT 33  HAS WGHT =0.1111
SEGMENT 34  HAS WGHT =0.1111
SEGMENT 35  HAS WGHT =0.1111
SEGMENT 36  HAS WGHT =0.4667
UB + DB=   14.6000 + 19.1000  TOTAL EX=      4.500
OUFALL SEGMENT= 34  TIDELOCKED ?= F
OUTFALL 20  SUM OF WEIGHTS =1.000
```

OUT 20

```
OUTFALL    COUNTY     AT=12.70 MLS,FLOW= 1.200 M.G.D
LOADS=FC 1300.00 SC=    0.00 FN=  170.00 SN=    0.00 NH3=  600.00 NO3=    7.60 D.O.=   19.64

SEGMENT 23  HAS WGHT =0.0416
SEGMENT 24  HAS WGHT =0.0833
SEGMENT 25  HAS WGHT =0.0833
SEGMENT 26  HAS WGHT =0.0833
SEGMENT 27  HAS WGHT =0.0833
SEGMENT 28  HAS WGHT =0.0873
SEGMENT 29  HAS WGHT =0.0833
SEGMENT 30  HAS WGHT =0.0833
SEGMENT 31  HAS WGHT =0.0833
SEGMENT 32  HAS WGHT =0.0833
SEGMENT 33  HAS WGHT =0.0833
SEGMENT 34  HAS WGHT =0.0833
SEGMENT 35  HAS WGHT =0.0422
UB + DB=   10.7504 + 16.7535  TOTAL EX=      6.003
OUFALL SEGMENT= 27  TIDELOCKED ?= F
OUTFALL 21  SUM OF WEIGHTS =1.000
```

OUT 21

FIG 7.9.F : DISTRIBUTED LOADS

FOR EACH COMPONENT BY OUTFALL

NO.	PSTN	FCADD	SCADD	FNADD	SNADD	NH3ADD	NO3ADD	D.O.ADD
1	13.90	32.00	22.00	28.00	14.00	44.00	2.50	8.04
2	13.54	570.00	0.00	70.00	0.00	240.00	0.00	7.24
3	13.43	740.00	0.00	100.00	0.00	340.00	0.00	9.41
4	12.92	500.00	0.00	60.00	0.00	50.00	0.00	6.55
5	12.82	700.00	200.00	100.00	0.00	1350.00	0.00	9.82
6	12.33	530.00	0.00	100.00	0.00	50.00	0.00	6.60
7	12.18	660.00	0.00	100.00	0.00	300.00	2.00	9.07
8	11.72	300.00	0.00	40.00	0.00	130.00	0.00	3.83
9	11.53	450.00	0.00	40.00	0.00	150.00	0.00	4.50
10	11.45	170.00	0.00	20.00	0.00	70.00	0.00	1.85
11	11.41	370.00	0.00	50.00	0.00	175.00	2.00	5.56
12	11.22	30.00	0.00	5.00	0.00	10.00	0.00	0.34
13	11.15	200.00	0.00	25.00	0.00	80.00	1.00	2.53
14	11.08	1510.00	0.00	160.00	0.00	600.00	5.00	12.64
15	10.81	130.00	0.00	20.00	0.00	50.00	0.00	1.71
16	9.61	300.00	0.00	40.00	0.00	120.00	0.00	3.83
17	9.10	80.00	80.00	15.00	15.00	40.00	1.00	3.51
18	8.81	130.00	0.00	20.00	0.00	40.00	1.00	1.77
19	7.20	1300.00	1300.00	250.00	250.00	1300.00	50.00	90.46
20	16.10	1000.00	1000.00	600.00	600.00	600.00	30.00	2209.23
21	12.70	1300.00	0.00	170.00	0.00	600.00	7.60	19.64

SUMMARY OF OUTFALLS/ TRIBUTARIES

FIG 7.9.9

ESTIMATED LOADS RECEIVED BY EACH SEGMENT

DISTANCE	SEG	FCADD	SCADD	FNADD	SLADD	AMMADD	ANO3AD	DOXADD
0.00								
	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.50								
	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.00								
	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.50								
	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.00								
	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.50								
	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.00								
	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.50								
	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.00								
	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.50								
	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5.00								
	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5.50								
	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6.00								
	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6.50								
	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7.00								
	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7.50								
	17	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.00								
	18	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.50								
	19	35.24	35.24	6.78	6.78	48.79	1.36	2.45
9.00								
	20	93.36	93.36	17.95	17.95	129.27	3.59	6.50
9.50								
	21	340.93	173.36	52.16	32.95	235.71	5.07	11.70
10.00								
	22	359.93	93.36	49.16	17.95	236.65	4.27	9.35
10.50								
	23	626.17	99.91	87.56	17.95	365.47	4.69	12.96
11.00								
	24	2273.66	110.26	274.37	17.95	1096.10	10.69	29.86
11.50								
	25	1079.32	110.73	136.59	19.25	585.20	4.44	18.59
12.00								
	26	1308.06	112.26	212.41	19.23	619.84	5.92	23.38
12.50								
	27	1179.33	238.11	164.13	19.23	1233.95	4.45	22.05
13.00								
	28	773.02	95.36	111.53	19.23	444.79	4.45	16.15
13.50								
	29	583.26	199.39	96.59	27.34	363.58	5.99	18.43
14.00								
	30	201.64	93.36	32.11	17.95	179.24	4.22	8.13
14.50								
	31	290.53	192.25	85.45	71.29	232.58	6.89	204.51
15.00								
	32	312.75	204.47	93.78	84.62	245.91	7.56	253.60
15.50								
	33	270.47	162.19	96.65	76.49	137.36	5.93	250.66
16.00								
	34	219.39	111.11	89.83	66.67	116.64	3.97	247.11
16.50								
	35	166.01	111.11	73.85	66.67	92.00	3.65	246.30
17.00								

```
FRESH WATER FLOW IS   330.00 /10.0 =*60
***********  RUNOFF FUNCTION FACTOR =  1.20000E 00
```
F.W.F

```
ARRAY F.W.FLOW .ELEMENT    1  TO   35
(  1 )=  8.46600E 02  (  2 )=  8.46600E 02  (  3 )=  8.46600E 02  (  4 )=  8.46600E 02  (  5 )=  8.46600E 02  (  6 )=  8.46600E 02
(  7 )=  8.46600E 02  (  8 )=  8.46600E 02  (  9 )=  8.46600E 02  ( 10 )=  8.46600E 02  ( 11 )=  8.46600E 02  ( 12 )=  8.46600E 02
( 13 )=  8.46600E 02  ( 14 )=  8.46600E 02  ( 15 )=  8.46600E 02  ( 16 )=  8.46600E 02  ( 17 )=  8.46600E 02  ( 18 )=  8.46600E 02
( 19 )=  8.47032E 02  ( 20 )=  8.51645E 02  ( 21 )=  8.53372E 02  ( 22 )=  8.63765E 02  ( 23 )=  8.71372E 02  ( 24 )=  8.89259E 02
( 25 )=  9.00327E 02  ( 26 )=  9.14544E 02  ( 27 )=  9.27519E 02  ( 28 )=  9.37652E 02  ( 29 )=  9.48866E 02  ( 30 )=  9.53548E 02
( 31 )=  9.76336E 02  ( 32 )=  1.02615E 03  ( 33 )=  1.06431E 03  ( 34 )=  1.10046E 03  ( 35 )=  1.13611E 03  (
```

MIXING TERMS

```
ARRAY MIX COEF .ELEMENT    1  TO   35
(  1 )=  0.00000E 00  (  2 )=  0.00000E 00  (  3 )=  0.00000E 00  (  4 )=  0.00000E 00  (  5 )=  0.00000E 00  (  6 )=  0.00000E 00
(  7 )=  0.00000E 00  (  8 )=  0.00000E 00  (  9 )=  0.00000E 00  ( 10 )=  0.00000E 00  ( 11 )=  3.38640E 02  ( 12 )=  1.18524E 03
( 13 )=  2.03184E 03  ( 14 )=  1.10709E 03  ( 15 )=  6.34950E 02  ( 16 )=  7.90160E 02  ( 17 )=  8.18380E 02  ( 18 )=  1.31446E 03
( 19 )=  2.74181E 03  ( 20 )=  4.32635E 03  ( 21 )=  5.43635E 03  ( 22 )=  2.81523E 03  ( 23 )=  4.88818E 03  ( 24 )=  5.87778E 03
( 25 )=  8.26245E 03  ( 26 )=  1.17198E 04  ( 27 )=  1.15371E 04  ( 28 )=  1.34954E 04  ( 29 )=  2.27202E 04  ( 30 )=  3.29336E 04
( 31 )=  5.06323E 04 ,( 32 )=  8.05520E 04  ( 33 )=  8.46159E 04  ( 34 )=  8.85861E 04  ( 35 )=  9.25920E 04  (
```

D.O.(SAT) from routine DOXSAT

```
ARRAY D.O.SAT. .ELEMENT    1  TO   36
(  1 )=  9.08000E 00  (  2 )=  9.08000E 00  (  3 )=  9.08000E 00  (  4 )=  9.08000E 00  (  5 )=  9.08000E 00  (  6 )=  9.08000E 00
(  7 )=  9.08000E 00  (  8 )=  9.08000E 00  (  9 )=  9.08000E 00  ( 10 )=  9.08000E 00  ( 11 )=  9.07904E 00  ( 12 )=  9.07663E 00
( 13 )=  9.07423E 00  ( 14 )=  9.07182E 00  ( 15 )=  9.06557E 00  ( 16 )=  9.04633E 00  ( 17 )=  9.01025E 00  ( 18 )=  8.93810E 00
( 19 )=  8.84671E 00  ( 20 )=  8.77459E 00  ( 21 )=  8.71444E 00  ( 22 )=  8.65672E 00  ( 23 )=  8.52685E 00  ( 24 )=  8.42824E 00
( 25 )=  8.32964E 00  ( 26 )=  8.24787E 00  ( 27 )=  8.18293E 00  ( 28 )=  8.11078E 00  ( 29 )=  8.04345E 00  ( 30 )=  8.00016E 00
( 31 )=  7.96839E 00  ( 32 )=  7.94125E 00  ( 33 )=  7.93282E 00  ( 34 )=  7.91839E 00  ( 35 )=  7.90396E 00  ( 36 )=  7.90396E 00

ARRAY AKCT.... .ELEMENT    2  TO   35
ARRAY IS SINGLE VALUED AT  2.33560E-01
```

Carbon, Nitrogen, Nitrate, Ammonia

```
ARRAY AKNT.... .ELEMENT    2  TO   35
ARRAY IS SINGLE VALUED AT  2.33560E-01

ARRAY AKNO3T.. .ELEMENT    2  TO   35
ARRAY IS SINGLE VALUED AT  5.10513E-03

ARRAY AKAMMT.. .ELEMENT    2  TO   35
ARRAY IS SINGLE VALUED AT  3.96386E-01
```

FIG 7.9.H

VARIOUS INTERMEDIATE PARAMETERS

```
ARRAY AMMONIAI .ELEMENT    2  TO   35
(  2 )=  9.45714E-01  (  3 )=  9.18571E-01  (  4 )=  8.91429E-01  (  5 )=  8.64286E-01  (  6 )=  8.37143E-01  (  7 )=  8.10000E-01
(  8 )=  7.82857E-01  (  9 )=  7.55714E-01  ( 10 )=  7.28571E-01  ( 11 )=  7.01429E-01  ( 12 )=  6.74286E-01  ( 13 )=  6.47143E-01
( 14 )=  6.20000E-01  ( 15 )=  5.92857E-01  ( 16 )=  5.65714E-01  ( 17 )=  5.38571E-01  ( 18 )=  5.11429E-01  ( 19 )=  4.84286E-01
( 20 )=  4.57143E-01  ( 21 )=  4.30000E-01  ( 22 )=  4.02857E-01  ( 23 )=  3.75714E-01  ( 24 )=  3.48571E-01  ( 25 )=  3.21429E-01
( 26 )=  2.94286E-01  ( 27 )=  2.67143E-01  ( 28 )=  2.40000E-01  ( 29 )=  2.12857E-01  ( 30 )=  1.85714E-01  ( 31 )=  1.58571E-01
( 32 )=  1.31429E-01  ( 33 )=  1.04286E-01  ( 34 )=  7.71429E-02  ( 35 )=  5.00001E-02  (

ARRAY NITRATEI .ELEMENT    2  TO   35
ARRAY IS SINGLE VALUED AT  1.00000E-01
```

INITIAL D.O. ESTIMATE OF SOLUTION

```
ARRAY D.O.INIT .ELEMENT    2  TO   35
(  2 )=  8.45000E 00  (  3 )=  8.42000E 00  (  4 )=  8.39000E 00  (  5 )=  8.36000E 00  (  6 )=  8.33000E 00  (  7 )=  8.30000E 00
(  8 )=  8.27000E 00  (  9 )=  8.24000E 00  ( 10 )=  8.21000E 00  ( 11 )=  8.18000E 00  ( 12 )=  8.15000E 00  ( 13 )=  8.12000E 00
( 14 )=  8.09000E 00  ( 15 )=  8.06000E 00  ( 16 )=  8.03000E 00  ( 17 )=  8.00000E 00  ( 18 )=  7.97000E 00  ( 19 )=  7.94000E 00
( 20 )=  7.91000E 00  ( 21 )=  7.88000E 00  ( 22 )=  7.85000E 00  ( 23 )=  7.82000E 00  ( 24 )=  7.79000E 00  ( 25 )=  7.76000E 00
( 26 )=  7.73000E 00  ( 27 )=  7.70000E 00  ( 28 )=  7.67000E 00  ( 29 )=  7.64000E 00  ( 30 )=  7.61000E 00  ( 31 )=  7.58000E 00
( 32 )=  7.55000E 00  ( 33 )=  7.52000E 00  ( 34 )=  7.49000E 00  ( 35 )=  7.46000E 00  (
```

```
ARRAY  RFC.... .ELEMENT    2  TO   35    FAST CARBON. RATE
 (  2 )=  8.51737E 02  (  3 )=  8.52250E 02  (  4 )=  8.52764E 02  (  5 )=  8.51223E 02  (  6 )=  8.51737E 02  (  7 )=  8.50196E 02
 (  8 )=  8.51223E 02  (  9 )=  8.51223E 02  ( 10 )=  8.52764E 02  ( 11 )=  1.19089E 03  ( 12 )=  2.37664E 03  ( 13 )=  4.07139E 03
 ( 14 )=  3.99324E 03  ( 15 )=  2.59738E 03  ( 16 )=  2.23193E 03  ( 17 )=  2.46182E 03  ( 18 )=  2.98714E 03  ( 19 )=  4.91093E 03
 ( 20 )=  7.93264E 03  ( 21 )=  1.06379E 04  ( 22 )=  9.13640E 03  ( 23 )=  8.61536E 03  ( 24 )=  1.16999E 04  ( 25 )=  1.51089E 04
 ( 26 )=  2.09779E 04  ( 27 )=  2.43389E 04  ( 28 )=  2.61653E 04  ( 29 )=  3.73802E 04  ( 30 )=  5.68950E 04  ( 31 )=  8.49938E 04
 ( 32 )=  1.32929E 05  ( 33 )=  1.67054E 05  ( 34 )=  1.75304E 05  ( 35 )=  1.83496E 05  (
ARRAY  RSC.... .ELEMENT    2  TO   35    SLOW CARBON. RATE
 (  2 )=  8.47627E 02  (  3 )=  8.47730E 02  (  4 )=  8.47833E 02  (  5 )=  8.47525E 02  (  6 )=  8.47627E 02  (  7 )=  8.47319E 02
 (  8 )=  8.47525E 02  (  9 )=  8.47525E 02  ( 10 )=  8.47833E 02  ( 11 )=  1.18637E 03  ( 12 )=  2.37171E 03  ( 13 )=  4.06522E 03
 ( 14 )=  3.98707E 03  ( 15 )=  2.59039E 03  ( 16 )=  2.27377E 03  ( 17 )=  2.45648E 03  ( 18 )=  2.98098E 03  ( 19 )=  4.90558E 03
 ( 20 )=  7.92237E 03  ( 21 )=  1.06244E 04  ( 22 )=  9.11955E 03  ( 23 )=  8.58290E 03  ( 24 )=  1.16642E 04  ( 25 )=  1.50543E 04
 ( 26 )=  2.09132E 04  ( 27 )=  2.42156E 04  ( 28 )=  2.60092E 04  ( 29 )=  3.72076E 04  ( 30 )=  5.66649E 04  ( 31 )=  8.46404E 04
 ( 32 )=  1.32354E 05  ( 33 )=  1.66396E 05  ( 34 )=  1.74503E 05  ( 35 )=  1.82550E 05  (
ARRAY  RFN.... .ELEMENT    2  TO   35    FAST NITROGENOUS RATE
 (  2 )=  8.51737E 02  (  3 )=  8.52250E 02  (  4 )=  8.52764E 02  (  5 )=  8.51223E 02  (  6 )=  8.51737E 02  (  7 )=  8.50196E 02
 (  8 )=  8.51223E 02  (  9 )=  8.51223E 02  ( 10 )=  8.52764E 02  ( 11 )=  1.19089E 03  ( 12 )=  2.37664E 03  ( 13 )=  4.07139E 03
 ( 14 )=  3.99324E 03  ( 15 )=  2.59738E 03  ( 16 )=  2.23193E 03  ( 17 )=  2.46182E 03  ( 18 )=  2.98714E 03  ( 19 )=  4.91093E 03
 ( 20 )=  7.93264E 03  ( 21 )=  1.06379E 04  ( 22 )=  9.13640E 03  ( 23 )=  8.61536E 03  ( 24 )=  1.16999E 04  ( 25 )=  1.51089E 04
 ( 26 )=  2.09779E 04  ( 27 )=  2.43389E 04  ( 28 )=  2.61653E 04  ( 29 )=  3.73802E 04  ( 30 )=  5.68950E 04  ( 31 )=  8.49938E 04
 ( 32 )=  1.32929E 05  ( 33 )=  1.67054E 05  ( 34 )=  1.75304E 05  ( 35 )=  1.83496E 05  (
ARRAY  RSN.... .ELEMENT    2  TO   35    SLOW NITROGENOUS RATE
 (  2 )=  8.47627E 02  (  3 )=  8.47730E 02  (  4 )=  8.47833E 02  (  5 )=  8.47525E 02  (  6 )=  8.47627E 02  (  7 )=  8.47319E 02
 (  8 )=  8.47525E 02  (  9 )=  8.47525E 02  ( 10 )=  8.47833E 02  ( 11 )=  1.18637E 03  ( 12 )=  2.37171E 03  ( 13 )=  4.06522E 03
 ( 14 )=  3.98707E 03  ( 15 )=  2.59039E 03  ( 16 )=  2.27377E 03  ( 17 )=  2.45648E 03  ( 18 )=  2.98098E 03  ( 19 )=  4.90558E 03
 ( 20 )=  7.92237E 03  ( 21 )=  1.06244E 04  ( 22 )=  9.11955E 03  ( 23 )=  8.58290E 03  ( 24 )=  1.16642E 04  ( 25 )=  1.50543E 04
 ( 26 )=  2.09132E 04  ( 27 )=  2.42156E 04  ( 28 )=  2.60092E 04  ( 29 )=  3.72076E 04  ( 30 )=  5.66649E 04  ( 31 )=  8.46404E 04
 ( 32 )=  1.32354E 05  ( 33 )=  1.06396E 05  ( 34 )=  1.74503E 05  ( 35 )=  1.82550E 05  (
ARRAY  RNO3.... .ELEMENT    2  TO   35    NITRATE RATE
 (  2 )=  8.46688E 02  (  3 )=  8.46697E 02  (  4 )=  8.46706E 02  (  5 )=  8.46679E 02  (  6 )=  8.46688E 02  (  7 )=  8.46662E 02
 (  8 )=  8.46679E 02  (  9 )=  8.46679E 02  ( 10 )=  8.46706E 02  ( 11 )=  1.18534E 03  ( 12 )=  2.37059E 03  ( 13 )=  4.06381E 03
 ( 14 )=  3.98567E 03  ( 15 )=  2.58879E 03  ( 16 )=  2.27189E 03  ( 17 )=  2.45525E 03  ( 18 )=  2.97957E 03  ( 19 )=  4.90436E 03
 ( 20 )=  7.92002E 03  ( 21 )=  1.06214E 04  ( 22 )=  9.11570E 03  ( 23 )=  8.57548E 03  ( 24 )=  1.16560E 04  ( 25 )=  1.50418E 04
 ( 26 )=  2.08982E 04  ( 27 )=  2.41875E 04  ( 28 )=  2.59735E 04  ( 29 )=  3.71682E 04  ( 30 )=  5.66123E 04  ( 31 )=  8.45597E 04
 ( 32 )=  1.32223E 05  ( 33 )=  1.66246E 05  ( 34 )=  1.74320E 05  ( 35 )=  1.82334E 05  (
ARRAY  RAMM.... .ELEMENT    2  TO   35    AMMONIA RATE
 (  2 )=  8.55317E 02  (  3 )=  8.56189E 02  (  4 )=  8.57061E 02  (  5 )=  8.54446E 02  (  6 )=  8.55317E 02  (  7 )=  8.52702E 02
 (  8 )=  8.54446E 02  (  9 )=  8.54449E 02  ( 10 )=  8.57061E 02  ( 11 )=  1.19483E 03  ( 12 )=  2.38094E 03  ( 13 )=  4.07676E 03
 ( 14 )=  3.99861E 03  ( 15 )=  2.60346E 03  ( 16 )=  2.29915E 03  ( 17 )=  2.46647E 03  ( 18 )=  2.99251E 03  ( 19 )=  4.91558E 03
 ( 20 )=  7.94160E 03  ( 21 )=  1.06490E 04  ( 22 )=  9.15108E 03  ( 23 )=  8.64365E 03  ( 24 )=  1.17311E 04  ( 25 )=  1.51566E 04
 ( 26 )=  2.10363E 04  ( 27 )=  2.44463E 04  ( 28 )=  2.63014E 04  ( 29 )=  3.75306E 04  ( 30 )=  5.70955E 04  ( 31 )=  8.53018E 04
 ( 32 )=  1.33431E 05  ( 33 )=  1.67627E 05  ( 34 )=  1.76302E 05  ( 35 )=  1.84319E 05  (
ARRAY  MIX  IN  .ELEMENT    1  TO   34    MIXING 'IN' ARRAY
 (  1 )=  8.46600E 02  (  2 )=  8.46600E 02  (  3 )=  8.46600E 02  (  4 )=  8.46600E 02  (  5 )=  8.46600E 02  (  6 )=  8.46600E 02
 (  7 )=  8.46600E 02  (  8 )=  8.46600E 02  (  9 )=  8.46600E 02  ( 10 )=  8.46600E 02  ( 11 )=  1.18524E 03  ( 12 )=  2.03184E 03
 ( 13 )=  2.37844E 03  ( 14 )=  1.93369E 03  ( 15 )=  1.40155E 03  ( 16 )=  1.63676E 03  ( 17 )=  1.66498E 03  ( 18 )=  2.16106E 03
 ( 19 )=  3.59079E 03  ( 20 )=  5.17800E 03  ( 21 )=  6.29472E 03  ( 22 )=  3.67900E 03  ( 23 )=  5.75955E 03  ( 24 )=  6.76704E 03
 ( 25 )=  9.16294E 03  ( 26 )=  1.26343E 04  ( 27 )=  1.24650E 04  ( 28 )=  1.44330E 04  ( 29 )=  2.36691E 04  ( 30 )=  3.38871E 04
 ( 31 )=  5.16156E 04  ( 32 )=  2.15781E 04  ( 33 )=  8.56301E 04  ( 34 )=  8.96866E 04  (
ARRAY  MIX  OUT .ELEMENT    2  TO   35    MIXING 'OUT' ARRAY
 (  2 )=  8.46600E 02  (  3 )=  8.46600E 02  (  4 )=  8.46600E 02  (  5 )=  8.46600E 02  (  6 )=  8.46600E 02  (  7 )=  8.46600E 02
 (  8 )=  8.46600E 02  (  9 )=  8.46600E 02  ( 10 )=  8.46600E 02  ( 11 )=  1.18524E 03  ( 12 )=  2.37048E 03  ( 13 )=  4.06368E 03
 ( 14 )=  3.98553E 03  ( 15 )=  2.58864E 03  ( 16 )=  2.27171E 03  ( 17 )=  2.45514E 03  ( 18 )=  2.97944E 03  ( 19 )=  4.90425E 03
 ( 20 )=  7.91986E 03  ( 21 )=  1.06211E 04  ( 22 )=  9.11534E 03  ( 23 )=  8.57479E 03  ( 24 )=  1.16552E 04  ( 25 )=  1.50406E 04
 ( 26 )=  2.08968E 04  ( 27 )=  2.41848E 04  ( 28 )=  2.59702E 04  ( 29 )=  3.71645E 04  ( 30 )=  5.66074E 04  ( 31 )=  8.45521E 04
 ( 32 )=  1.32210E 05  ( 33 )=  1.66232E 05  ( 34 )=  1.74302E 05  ( 35 )=  1.82314E 05  (
FC-SC-FN-SN SOLUTIONS  COMPUTED

                                                                    C.P.U. TIME USED =    44.094 ETUS
```

FIG 7.9.1

RATE ARRAYS

FIG 7.9.J HISTORY OF CONVERGANCE
(MODERATE TUNING OF PARAMETERS)

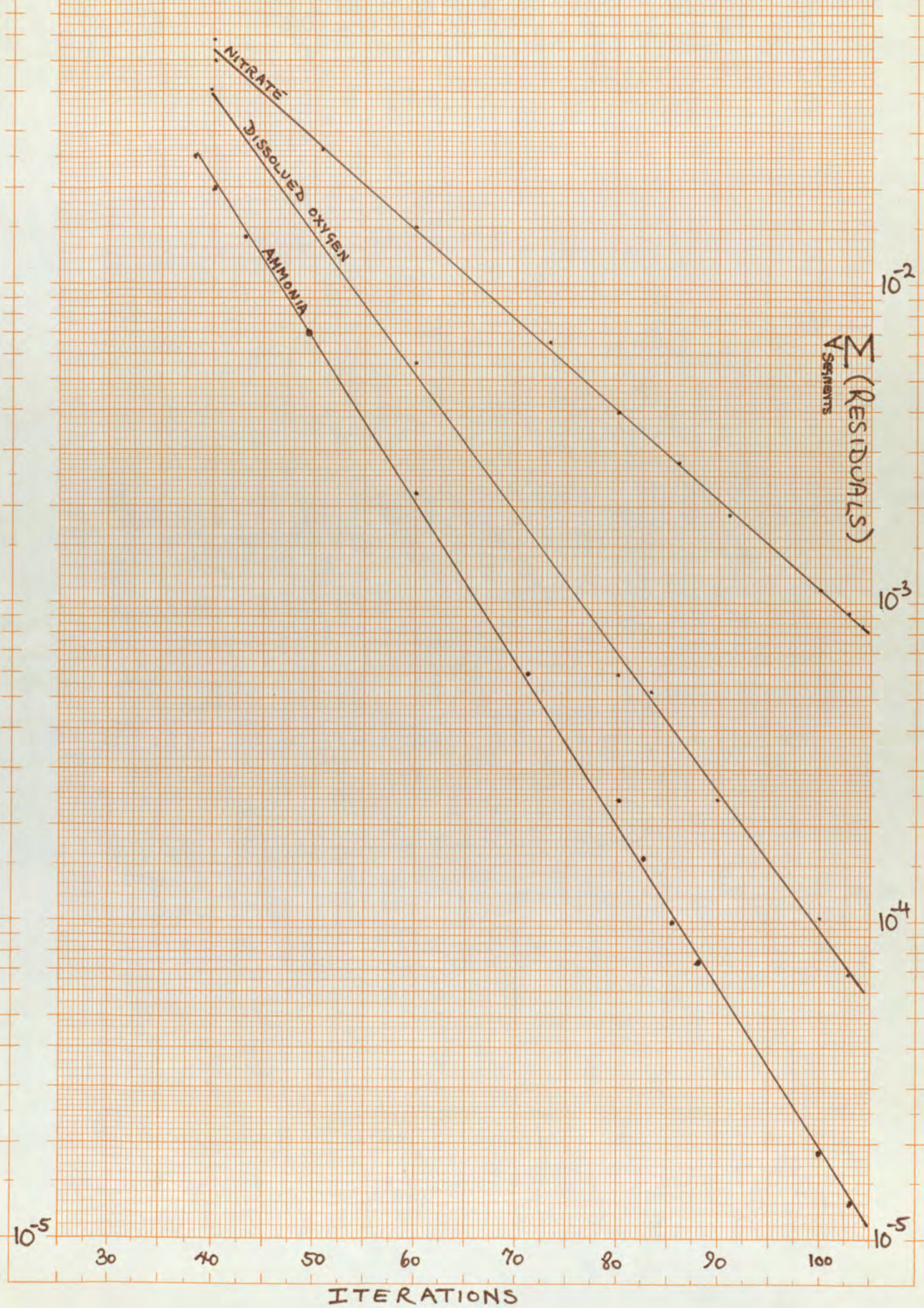


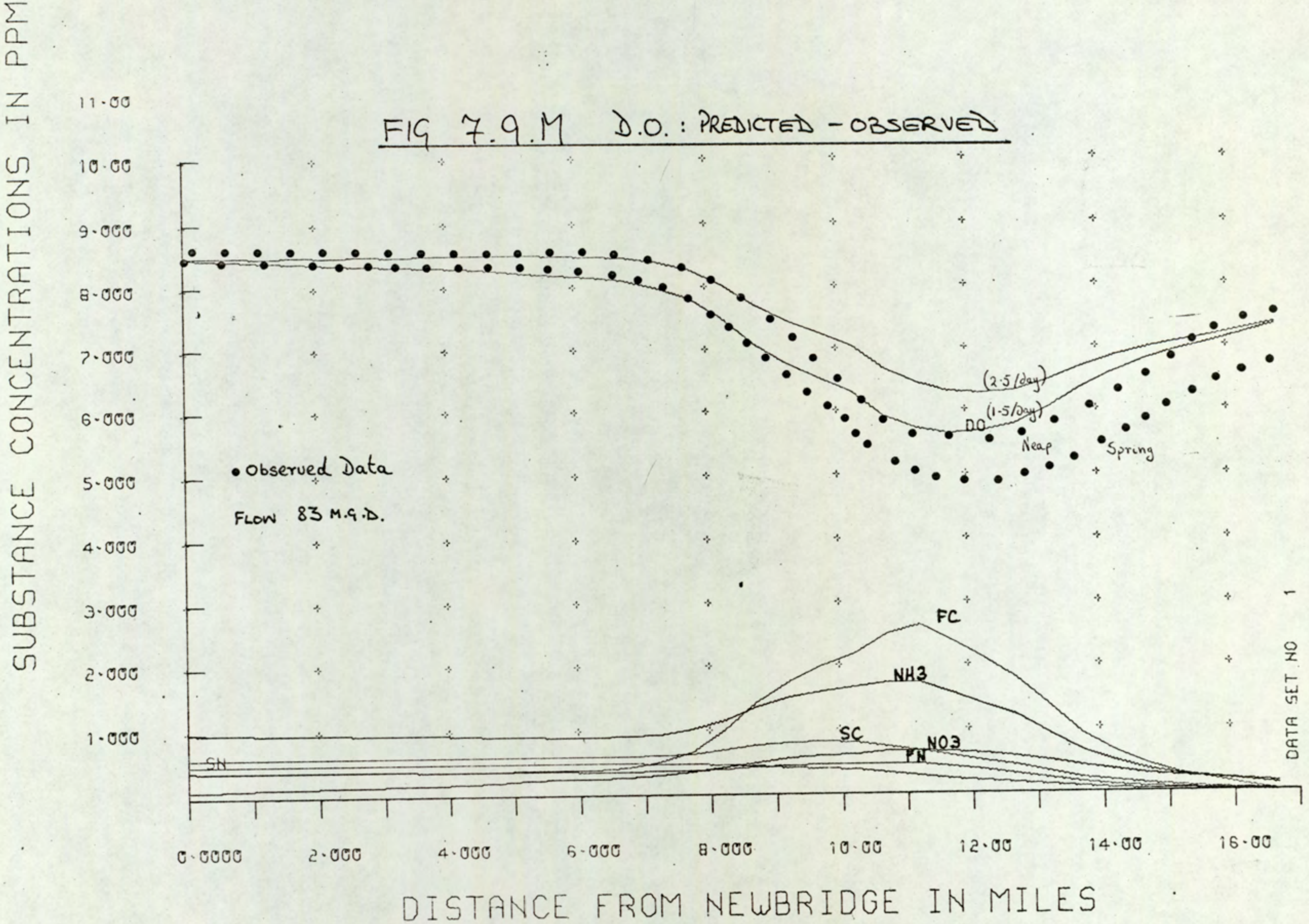

FIG 7.9.K HISTORY OF CONVERGANCE
(FINE TUNE & INITIAL APPROXIMATION)
NO3
D.O.
AMMONIA
Σ (RESIDUALS)
10-4
10-5
10-6
10-7
5
10
15
20
25
30
35
40
ITERATIONS
LIMIT OF MACHINE / METHOD OF COMPUTATION ACCURACY

POSITION	SEG	FAST--CARBON--SLOW		FAST-NITROGEN-SLOW		AMMONIA	NITRATE	MG/L D.O.	%SAT.	U	W	*FLG
0.000	1	0.400	0.600	0.400	0.500	1.000	0.100	8.510	93.722	0.000	0.000	
0.500	2	0.398	0.599	0.398	0.499	0.993	0.110	8.483	93.422	0.000	0.000	
1.000	3	0.395	0.598	0.395	0.499	0.985	0.121	8.451	93.068	0.000	0.000	
1.500	4	0.392	0.598	0.392	0.498	0.976	0.133	8.418	92.704	0.000	0.000	
2.000	5	0.390	0.597	0.390	0.497	0.970	0.142	8.399	92.504	0.000	0.000	
2.500	6	0.388	0.596	0.388	0.497	0.963	0.152	8.374	92.228	0.000	0.000	
3.000	7	0.386	0.596	0.386	0.496	0.958	0.159	8.360	92.072	0.000	0.000	
3.500	8	0.384	0.595	0.384	0.496	0.952	0.168	8.339	91.839	0.000	0.000	
4.000	9	0.382	0.594	0.382	0.495	0.946	0.177	8.323	91.663	0.000	0.000	
4.500	10	0.379	0.594	0.379	0.495	0.938	0.188	8.298	91.385	0.000	0.000	
5.000	11	0.379	0.593	0.375	0.493	0.926	0.209	8.264	91.023	0.000	0.000	
5.500	12	0.384	0.593	0.370	0.491	0.915	0.234	8.219	90.547	0.000	0.000	
6.000	13	0.391	0.593	0.367	0.490	0.910	0.250	8.182	90.164	0.000	0.000	
6.500	14	0.399	0.595	0.366	0.489	0.909	0.261	8.156	89.903	0.000	0.000	
7.000	15	0.424	0.598	0.365	0.486	0.914	0.279	8.104	89.395	0.000	0.000	
7.500	16	0.507	0.611	0.368	0.481	0.944	0.312	7.990	88.319	0.000	0.000	
8.000	17	0.669	0.637	0.377	0.472	1.014	0.354	7.825	86.843	0.000	0.000	
8.500	18	0.999	0.690	0.399	0.455	1.166	0.424	7.500	83.916	0.000	0.000	
9.000	19	1.422	0.753	0.429	0.435	1.366	0.500	7.099	80.246	0.000	0.000	
9.500	20	1.747	0.800	0.451	0.416	1.512	0.555	6.787	77.349	0.000	0.000	
10.000	21	2.002	0.814	0.468	0.397	1.610	0.592	6.546	75.114	0.000	0.000	
10.500	22	2.190	0.796	0.476	0.374	1.669	0.619	6.341	73.255	0.000	0.000	
11.000	23	2.504	0.728	0.480	0.317	1.736	0.659	5.953	69.811	0.000	0.000	
11.500	24	2.637	0.658	0.470	0.271	1.734	0.664	5.764	68.385	0.000	0.000	
12.000	25	2.407	0.574	0.418	0.224	1.570	0.647	5.673	68.111	0.000	0.000	
12.500	26	2.113	0.493	0.363	0.185	1.385	0.612	5.698	69.079	0.000	0.000	
13.000	27	1.739	0.422	0.305	0.152	1.204	0.568	5.793	70.788	0.000	0.000	
13.500	28	1.357	0.325	0.231	0.116	0.922	0.494	6.024	74.274	0.000	0.000	
14.000	29	0.922	0.229	0.158	0.081	0.649	0.402	6.334	78.753	0.000	0.000	
14.500	30	0.629	0.164	0.109	0.058	0.469	0.334	6.574	82.178	0.000	0.000	
15.000	31	0.417	0.115	[illegible]	0.041	0.340	0.277	6.775	85.024	0.000	0.000	
15.500	32	0.274	0.079	[illegible]	0.029	0.253	0.235	6.933	87.237	0.000	0.000	
16.000	33	0.181	0.053	[illegible]	0.020	0.198	0.204	7.052	88.894	0.000	0.000	
16.500	34	[illegible]	0.037	[illegible]	0.013	0.146	0.171	7.181	90.691	0.000	0.000	
	35	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]					

FIG 7.9.L

PREDICTIONS GENERATED BY STEADY STATE MODEL

FIG 7.9.M D.O.: PREDICTED - OBSERVED



URD STEADY STATE MODEL OUTPUT MWR

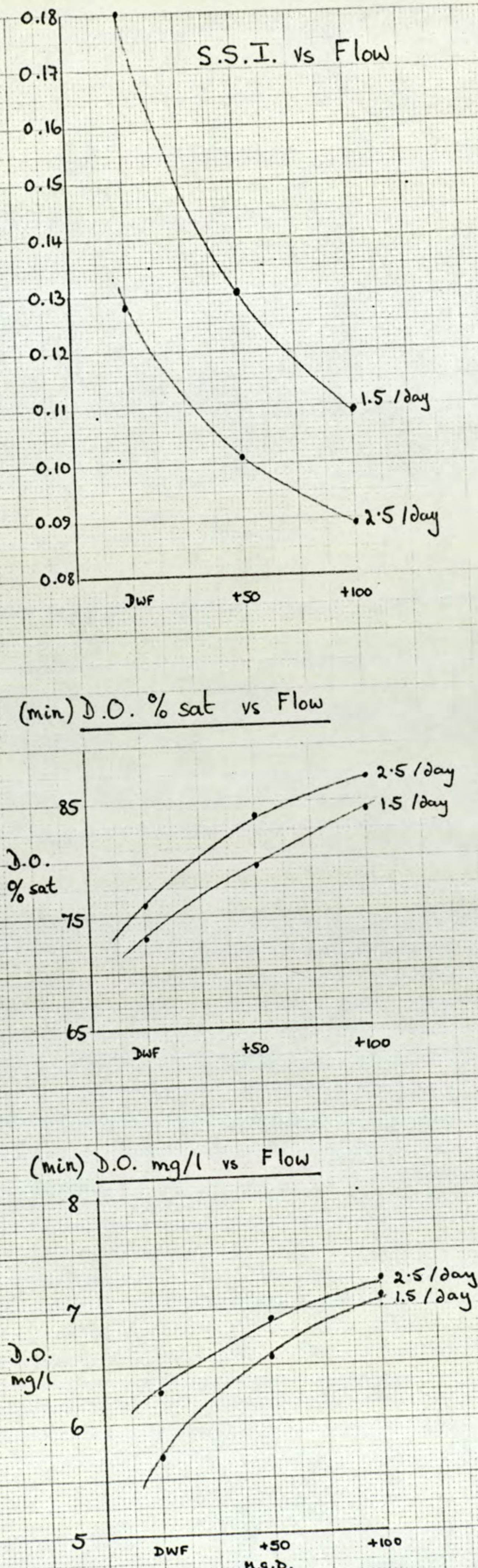


Fig 7.9.N
Summary of
Remaining
Predictions

7.10 Simulation of Low Flow Conditions Model.

As the actual levels of flow below 3-4 cumecs are unlikely to be maintained for the required period, these are essentially pseudo-steady state simulations.

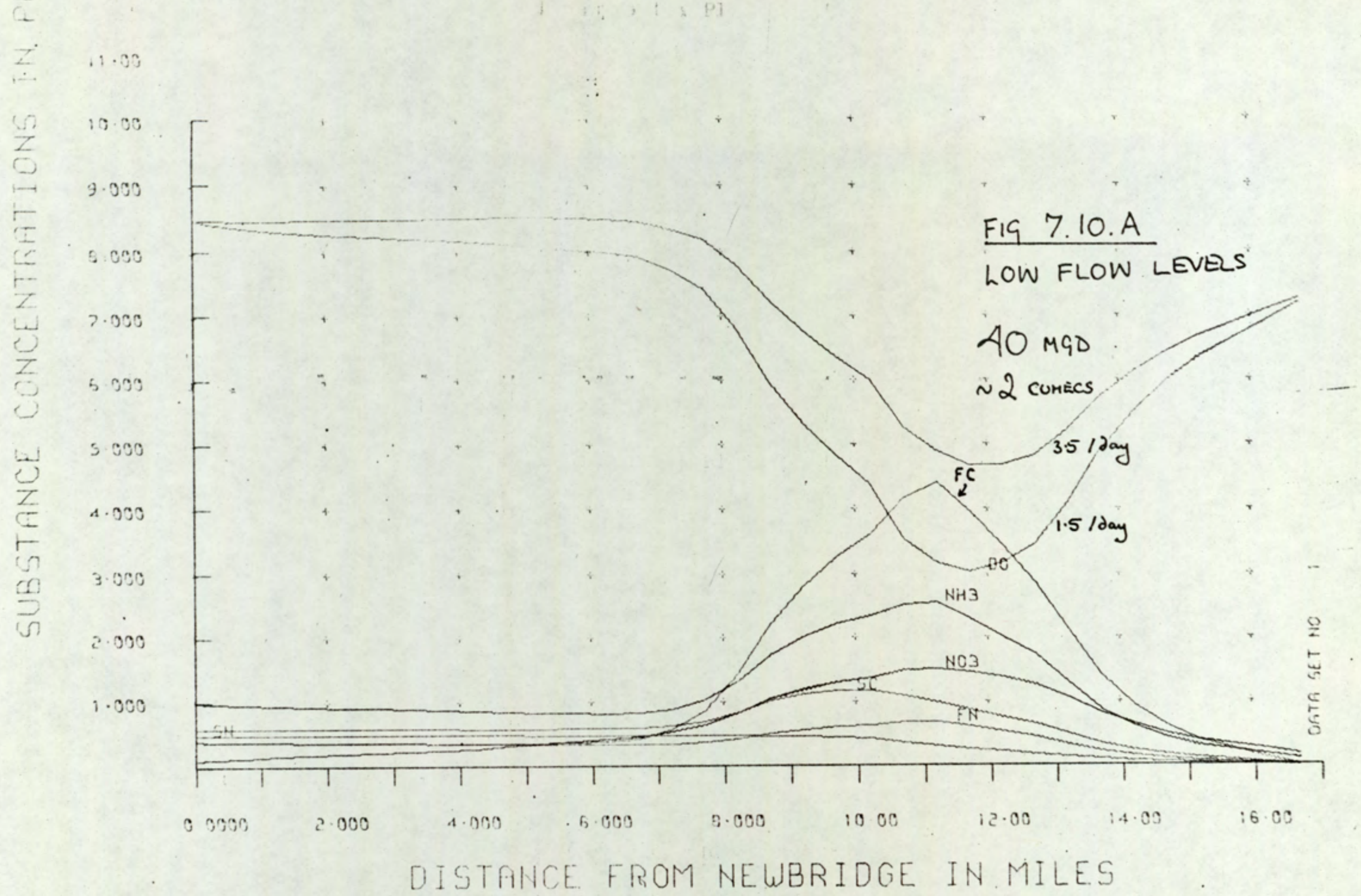
Flows were simulated from 40 mgd (50% of Thoms-Wain's DWF definition) to 160 mgd (twice DWF) in steps of 15 mgd (i.e. 2 cumecs to 8.4 cumecs in steps of 0.8 cumecs).

Figures 7.10. A to D highlight the overall radical dependence of quality on a sustained flow level. Below 85 mgd the mean D.O. curve is markedly depressed for re-aeration of 1.5 per day. The 3.5 per day upper curve on each figure shows the beneficial effect of a higher rate if it can be maintained.

Fig. 7.10. E shows that above 120 mgd maintained flow little benefit is gained in terms of D.O. sags. There is an 8% drop in D.O. (absolute) likely for the proposed redefined DWF[11].

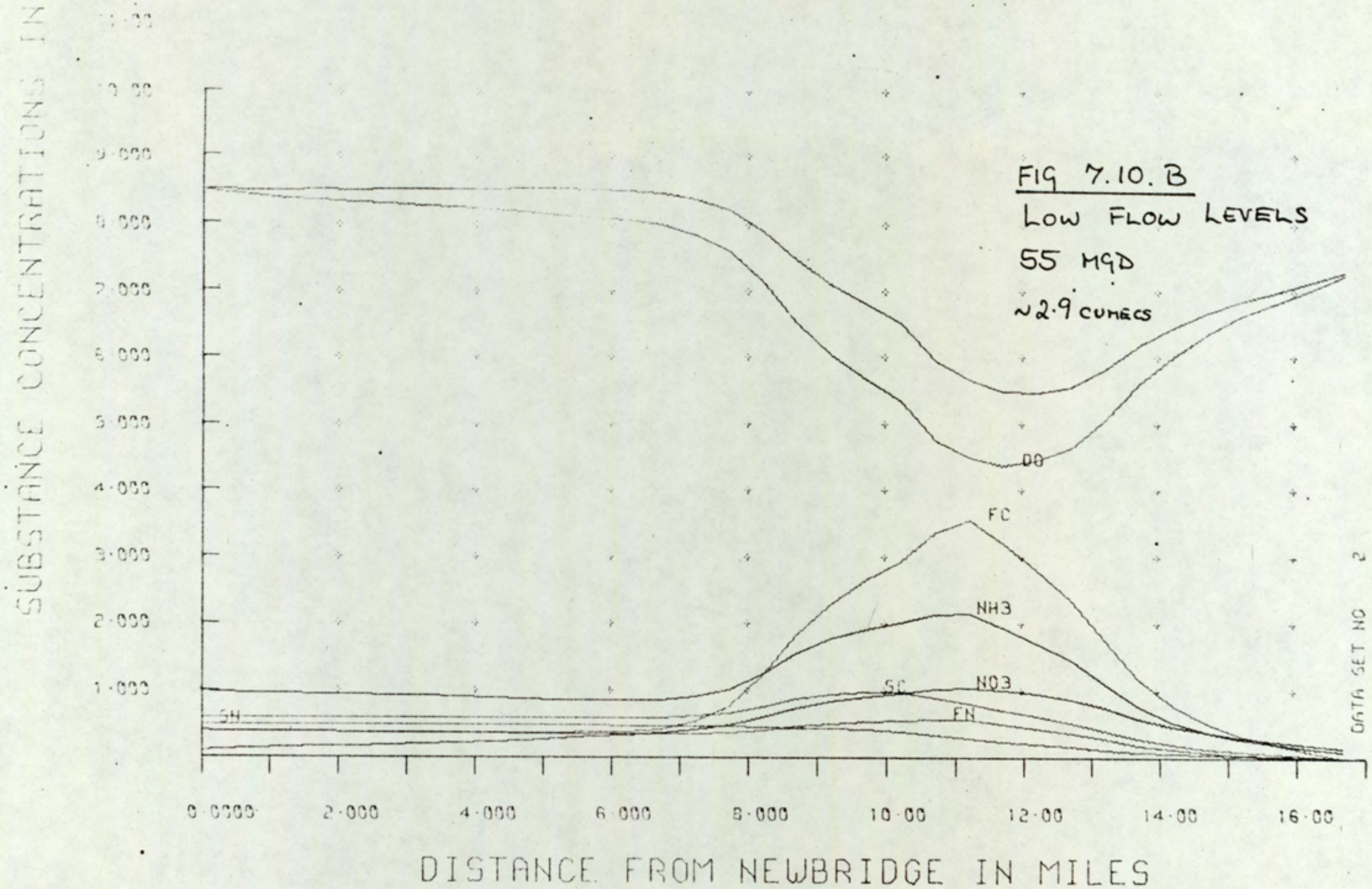
It should be emphasised that the flow refers to headwater input only. In practice, because of correlative flows (Chapter 6) flows from tributaries would be modified accordingly.

It appears that flows below 60 mgd for any period of time would certainly cause severe fisheries problems in the estuary.



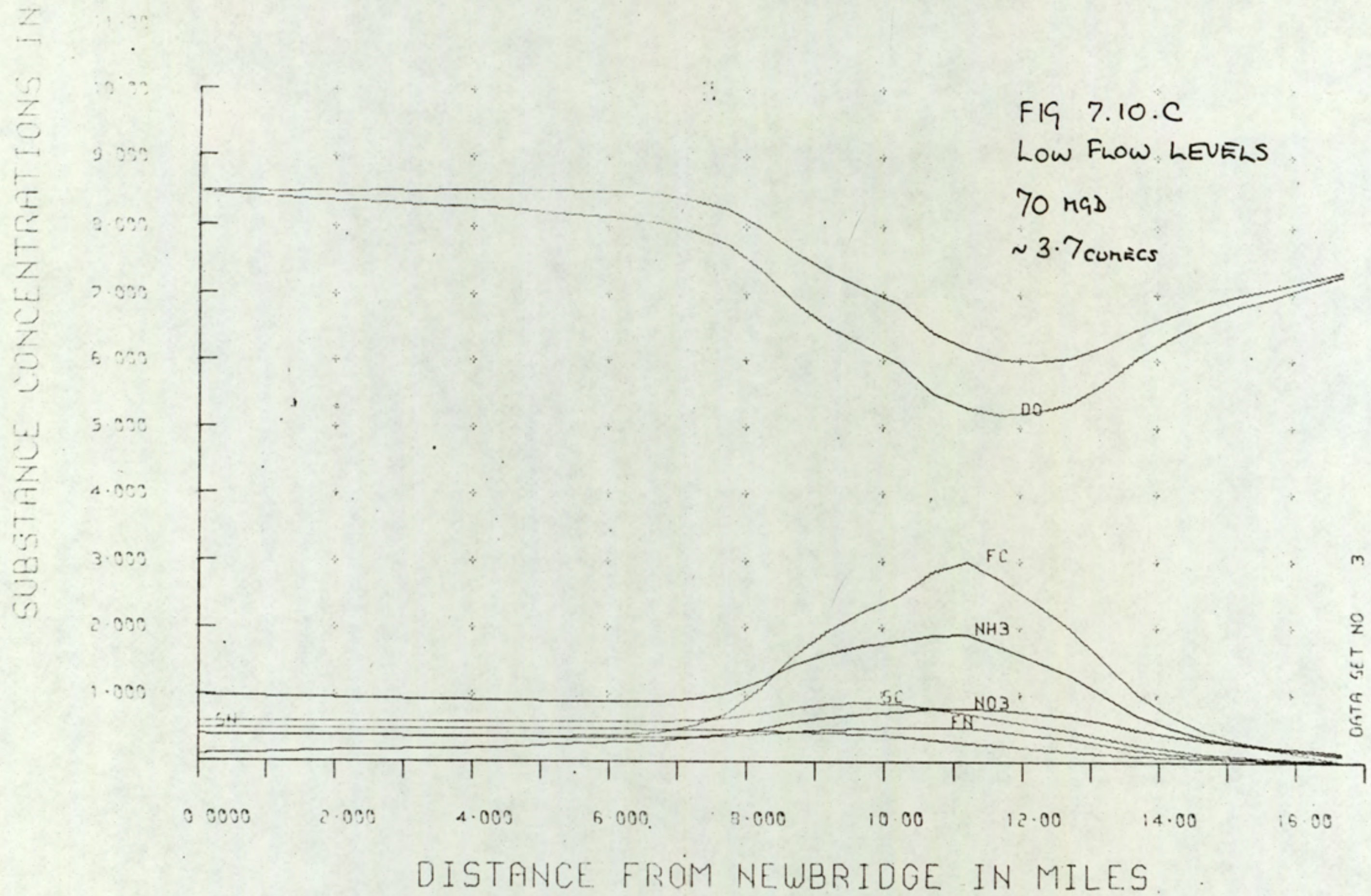
URD STEADY STATE MODEL OUTPUT MWR

URDOOL URD01.PLT001[N0001]



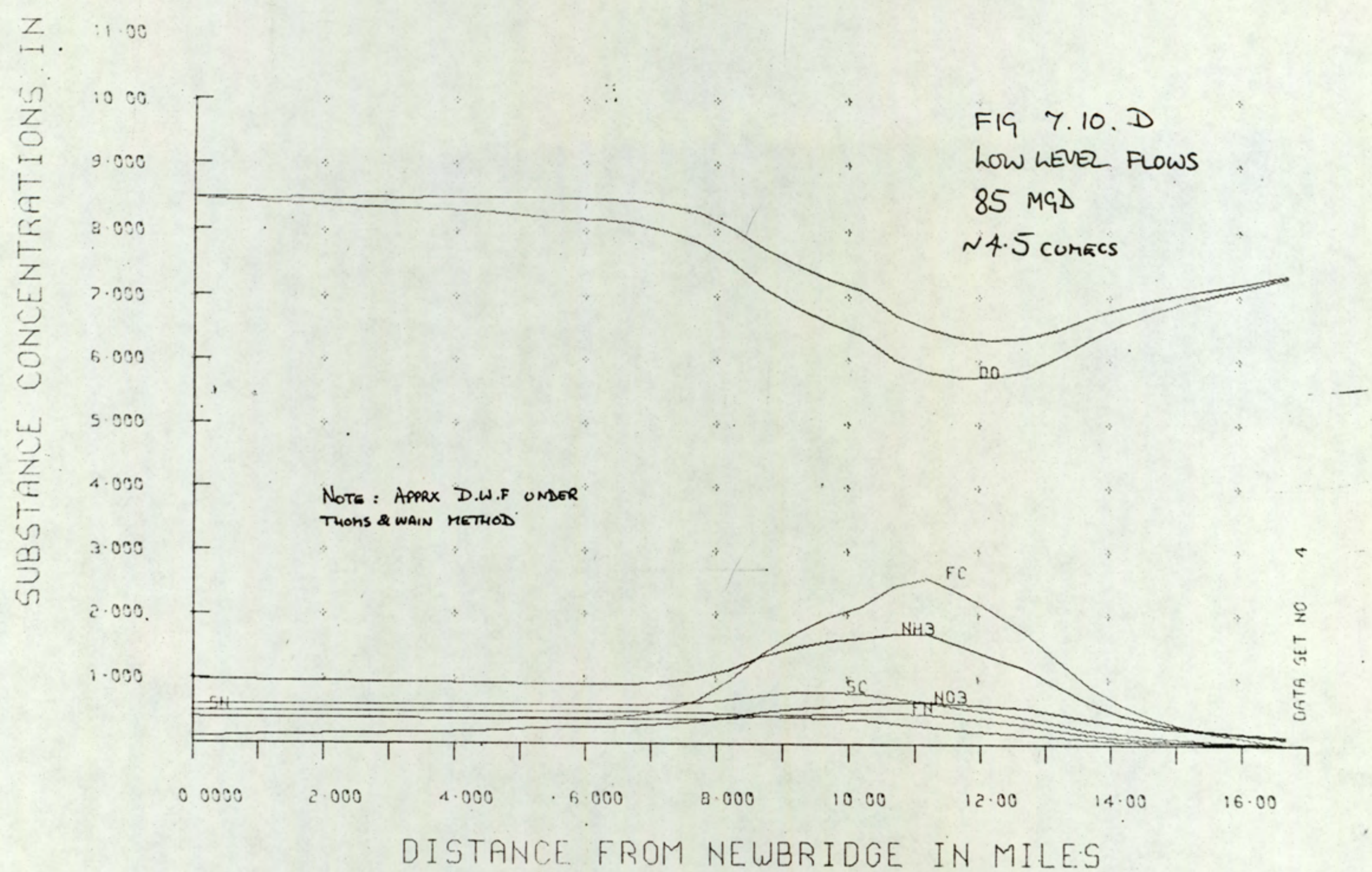
URD STEADY STATE MODEL OUTPUT MWR

URDOOL URD01·PLT002[N0001]



URD STEADY STATE MODEL OUTPUT MWR

URDOOL URD01-PLT003[N0001]

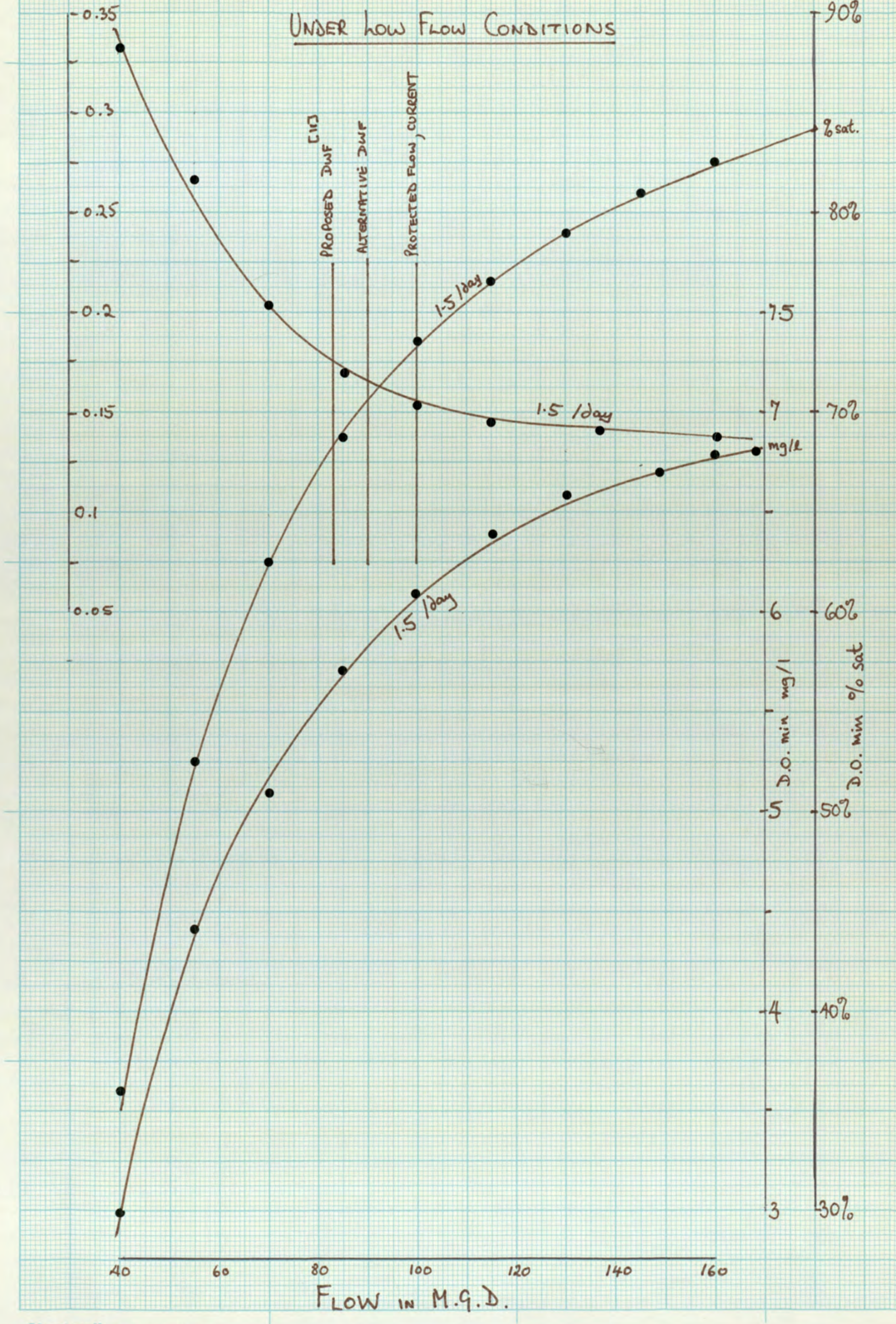


URD STEADY STATE MODEL OUTPUT MWR

URD00L URD01·PLT004[N0001]

Fig 7.10.E Minimum D.O.'s & SSI

Under Low Flow Conditions



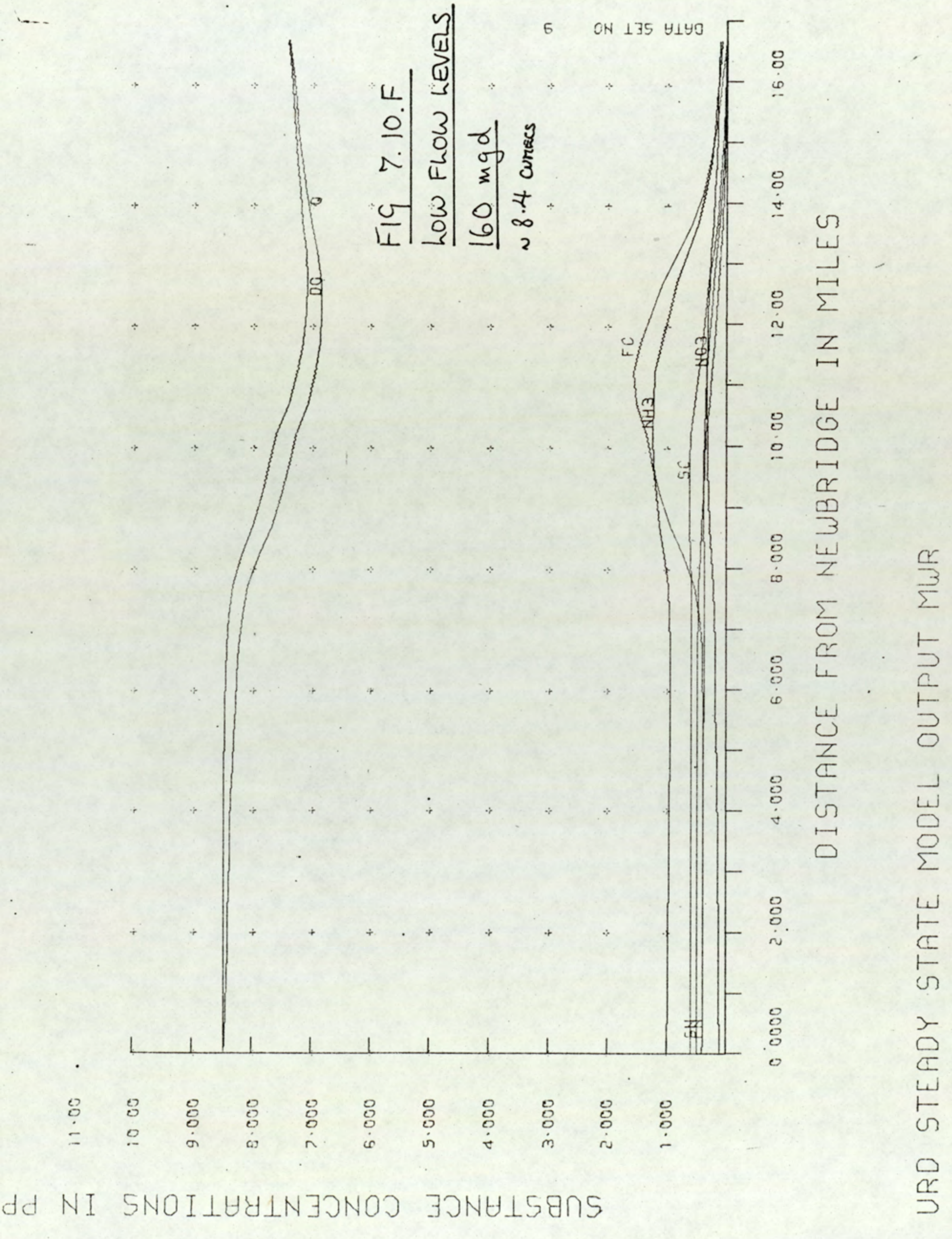


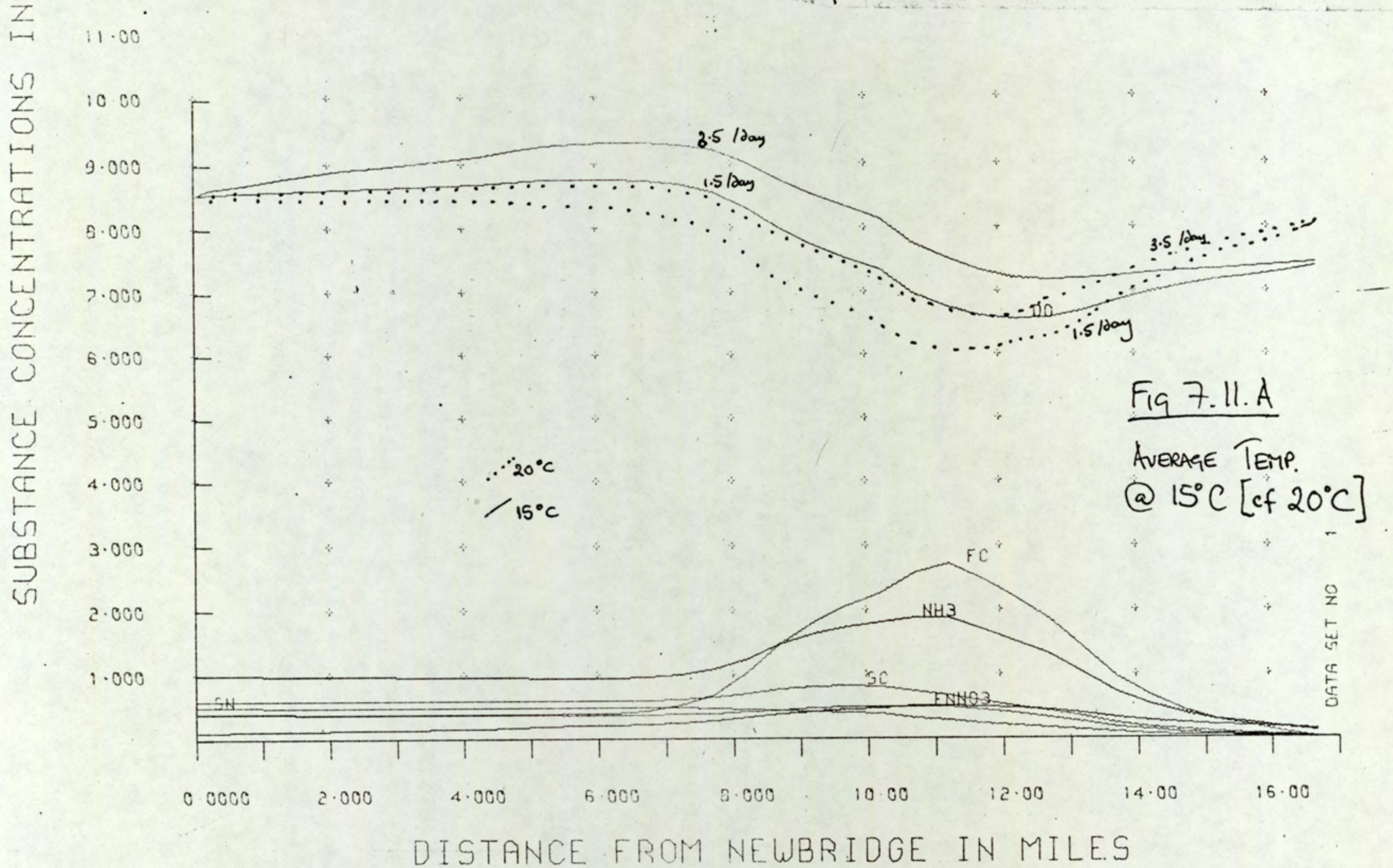
FIG 7.10.F
LOW FLOW LEVELS
160 mgd
~ 8.4 cumecs
DATA SET NO 9
DO
FC
NH3
NO3
SC
FN
0.0000
2.000
4.000
6.000
8.000
10.00
12.00
14.00
16.00
1.000
2.000
3.000
4.000
5.000
6.000
7.000
8.000
9.000
10.00
11.00
SUBSTANCE CONCENTRATIONS IN PP
DISTANCE FROM NEWBRIDGE IN MILES
URD STEADY STATE MODEL OUTPUT MWR

7.11 Varying Temperatures

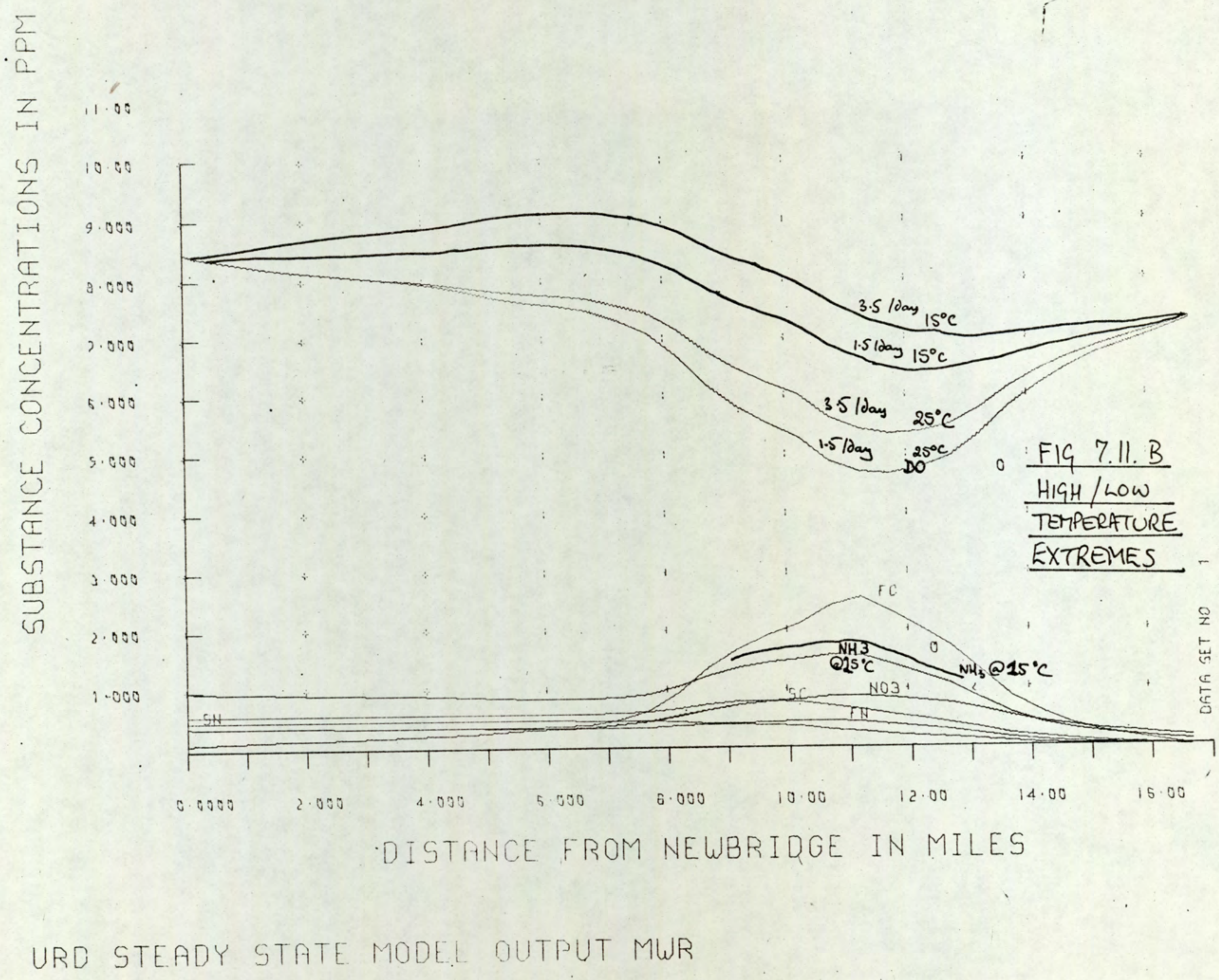
Temperature variations within one to two degrees had little effect (the rate constants varying less than 2.5% per degree). A step of $5^{o}C$ to $15^{o}C$ produces a delayed sag as expected, and not so severe (fig. 7.11. A). $20^{o}C$ was chosen as a standard to represent moderately severe conditions. Recorded estuary temperatures are in the range $12^{o}C$ to $24^{o}C$.

At $25^{o}C$ the effect on D.O. is pronounced (fig. 7.11. B) especially when combined with low flows (both figures are for DWF), but such a sustained temperature in an estuary would only result if other river temperatures were in excess of this. In that event, the potential loss of the rivers would be a more pressing problem.

Figure 7.11.C simulates a 500 year drought situation. The computer run was on 31 Dec 1975, just ahead of a 200 year drought ! The low flow / high temperature combination is highlighted for 40 mgd to 60 mgd. These frequencies are beyond most design criteria unless major projects are involved.



URD STEADY STATE MODEL OUTPUT MWR


SUBSTANCE CONCENTRATIONS IN PPM
11·00
10·00
9·000
8·000
7·000
6·000
5·000
4·000
3·000
2·000
1·000
0·0000
2·000
4·000
6·000
8·000
10·00
12·00
14·00
16·00
DISTANCE FROM NEWBRIDGE IN MILES
3·5 /day 15°C
1·5 /day 15°C
3·5 /day 25°C
1·5 /day 25°C
DO
FIG 7.11. B
HIGH / LOW
TEMPERATURE
EXTREMES
DATA SET NO 1
FC
NH3 @25°C
NH_3 @15°C
NO3
SC
FN
SN
URD STEADY STATE MODEL OUTPUT MWR

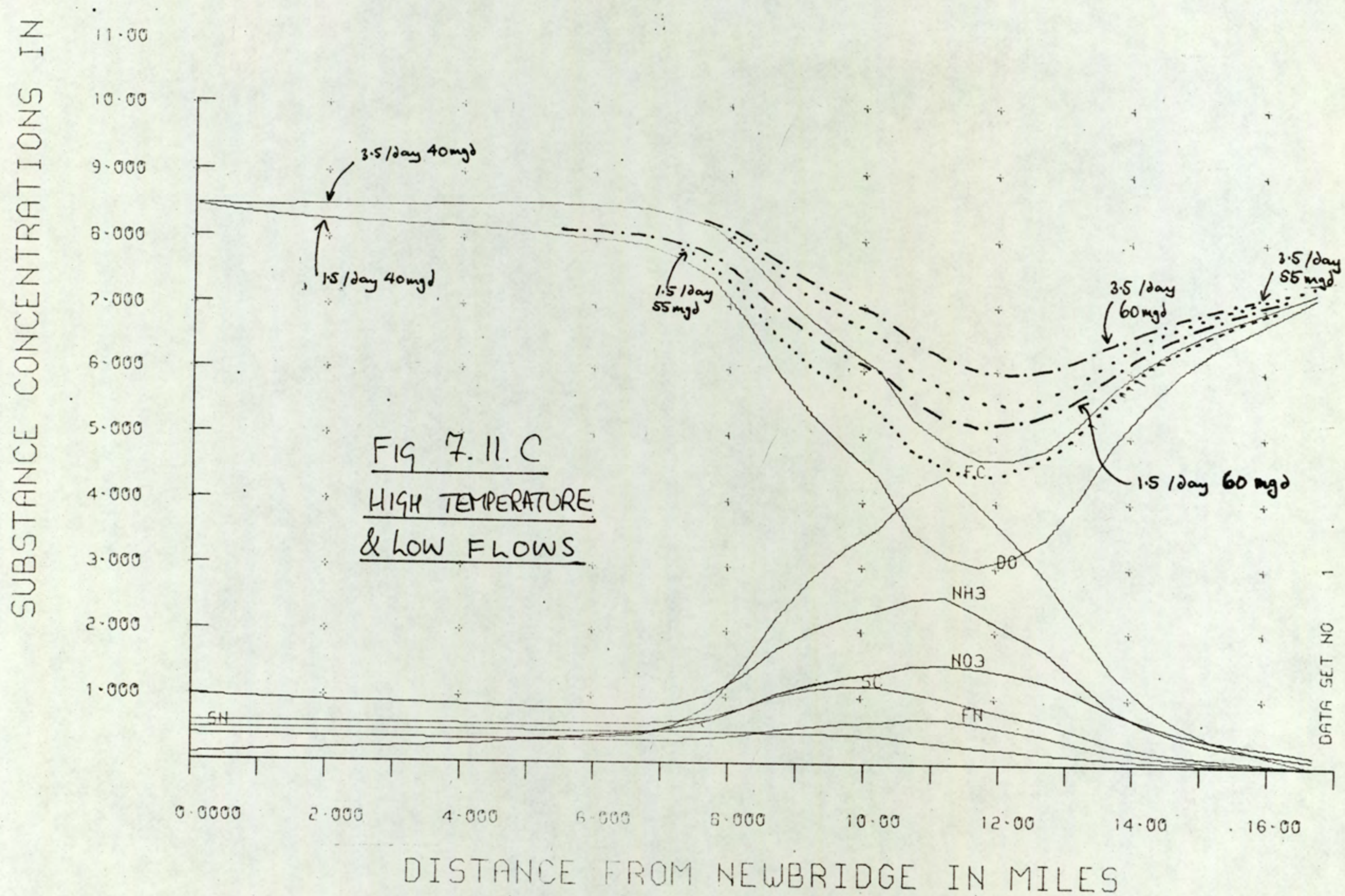


Fig 7.11.C
HIGH TEMPERATURE
& LOW FLOWS

URD STEADY STATE MODEL OUTPUT MWR

7.12 An Alternative to the Newport Main Drainage Scheme ?

It is recognised that the fact that 18 discharges are tidelocked for various lengths of time has a derogatory effect on the overall dissolved oxygen profile.

What would the effect of having only pumped discharges be? The net loading would not alter but the distribution of received loads would alter radically and 'larger' segments receive a greater proportion. Fig. 7.12. A shows the redistribution of the fast carbonaceous component. The peak load is dampened over more downstream segments.

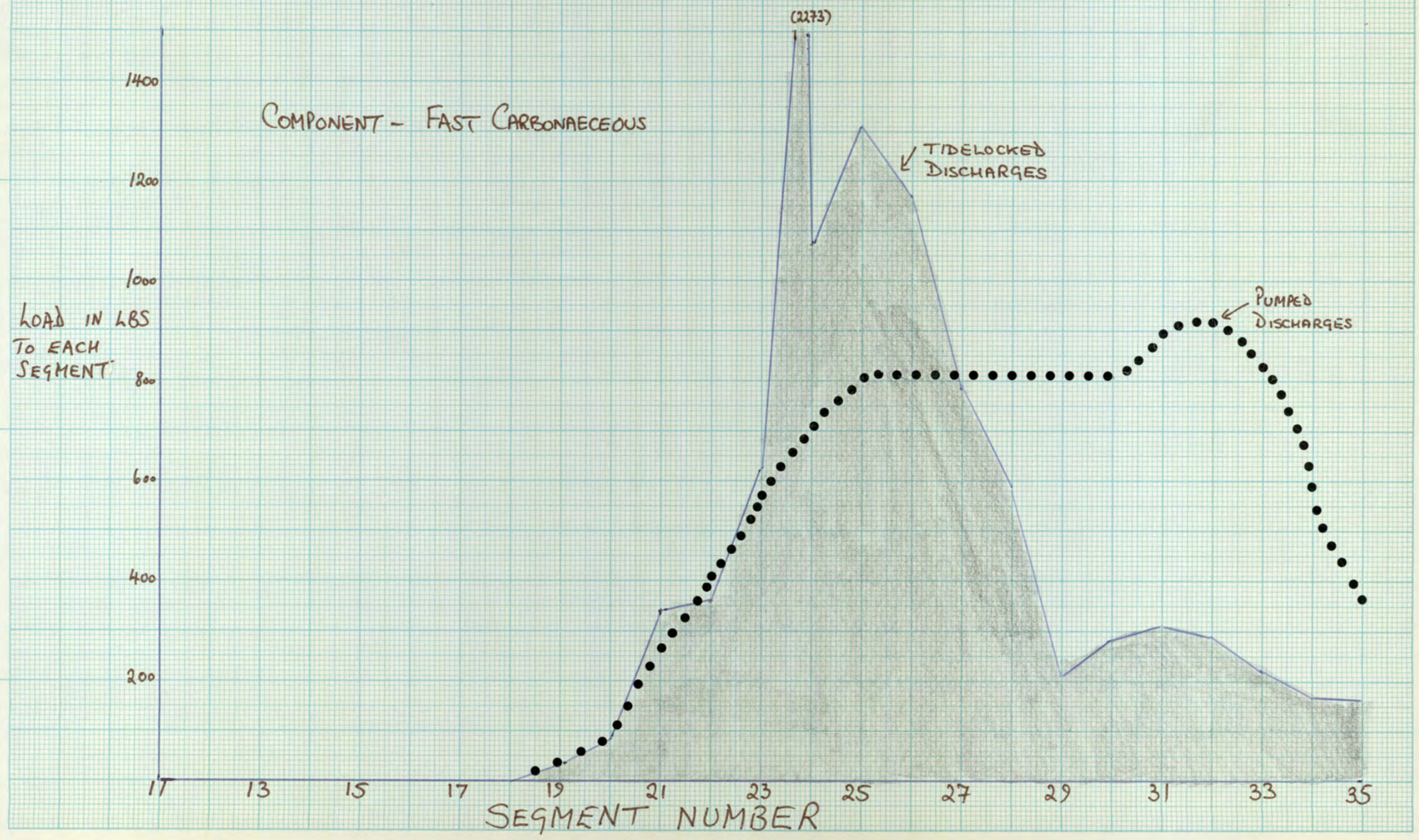
The net effect is summarised in terms of percentage (relative) improvement of dissolved oxygen and SSI.

Table 7.12. A Improvements in Min. D.O. and SSI.

		min.D.O. mg/l			SSI		
Flow	Re-aer.	from	to	%+	from	to	%+
83	1.5	5.7	6.0	5.3	.1807	.1629	9.9
83	3.5	6.3	6.6	4.8	.1284	.1157	9.9
133	1.5	6.6	6.8	3.0	.1302	.1217	6.5
133	3.5	6.9	7.1	2.9	.1010	.0943	6.6
183	1.5	7.0	7.1	1.4	.1089	.1038	4.7
183	3.5	7.2	7.3	1.4	.0889	.0846	4.8

The improvement indicated in the SSI suggest a possible intermediate solution if an immediate improvement is required for a low capital outlay; installation of pumps at the main tidelocked discharges.

FIG 7.12.A - REDISTRIBUTION OF LOADINGS WITH PUMPED DISCHARGES



7.13 Varying Re-aeration Rates

As re-aeration is the only positive process for introduction of oxygen to the system, the importance of the rate of this process is apparent. Fig. 7.13.A shows the D.O. distribution for varying rates of re-aeration from 0.5 to 2.1 per day. The more pronounced the initial sag, the more effective the different rates become, as they are indicative of a net deficit forcing process. Fig. 7.13.B shows the effect on the overall D.O. profile. The net difference decreases with increasing rate.As flows increase, the effect of varying re-aeration rates is greatly reduced because of four factors:

a) increased dilution

b) more mixing

c) shorter retention

d) lower deficits

By the time flows exceed 2 x DWF, re-aeration rates in the Usk do not have a great effect relative to the overall D.O.(%) profile. The effects above however are better compared to deficits in an estuary where the overall profile is relatively healthy (SSI less than 0.2).

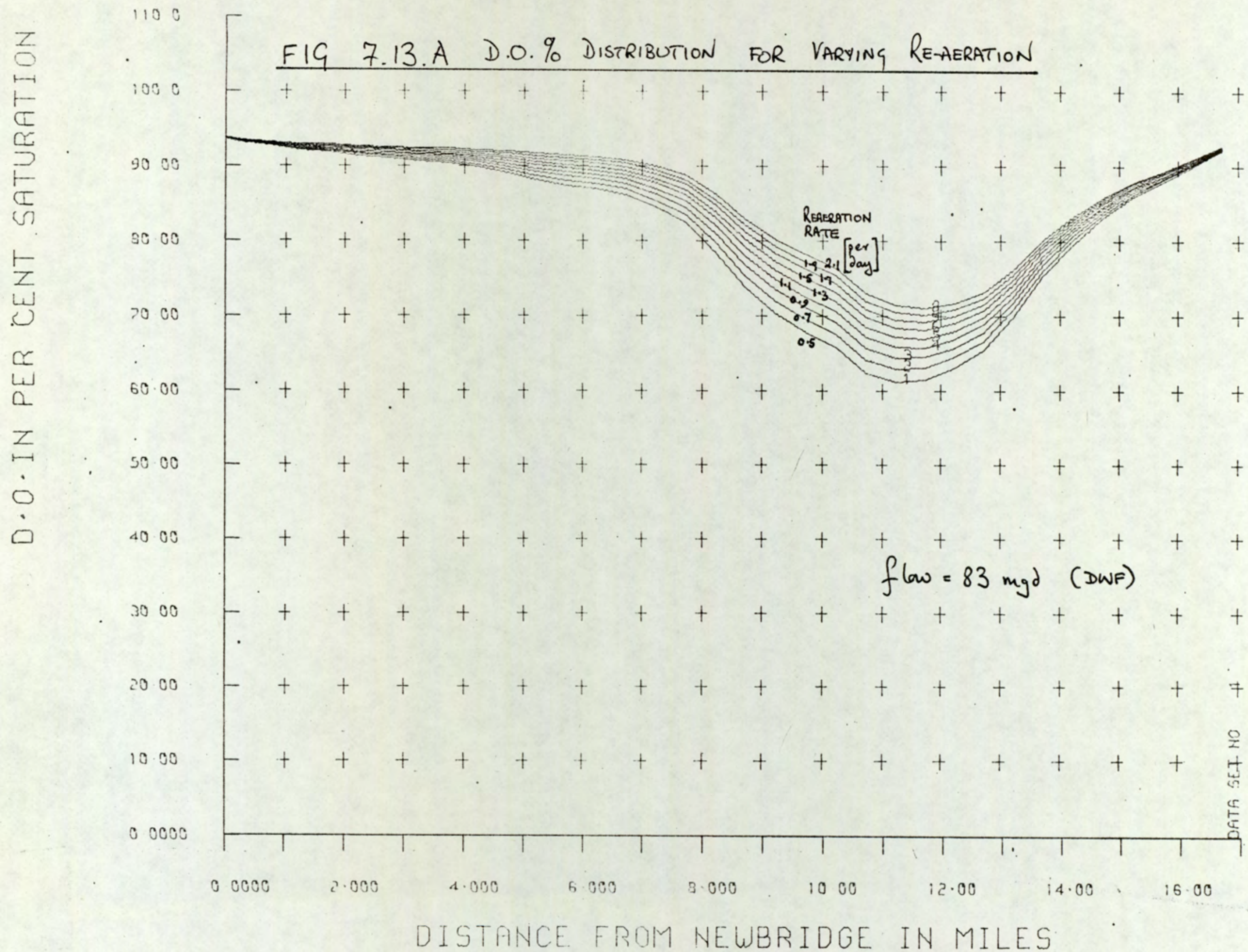
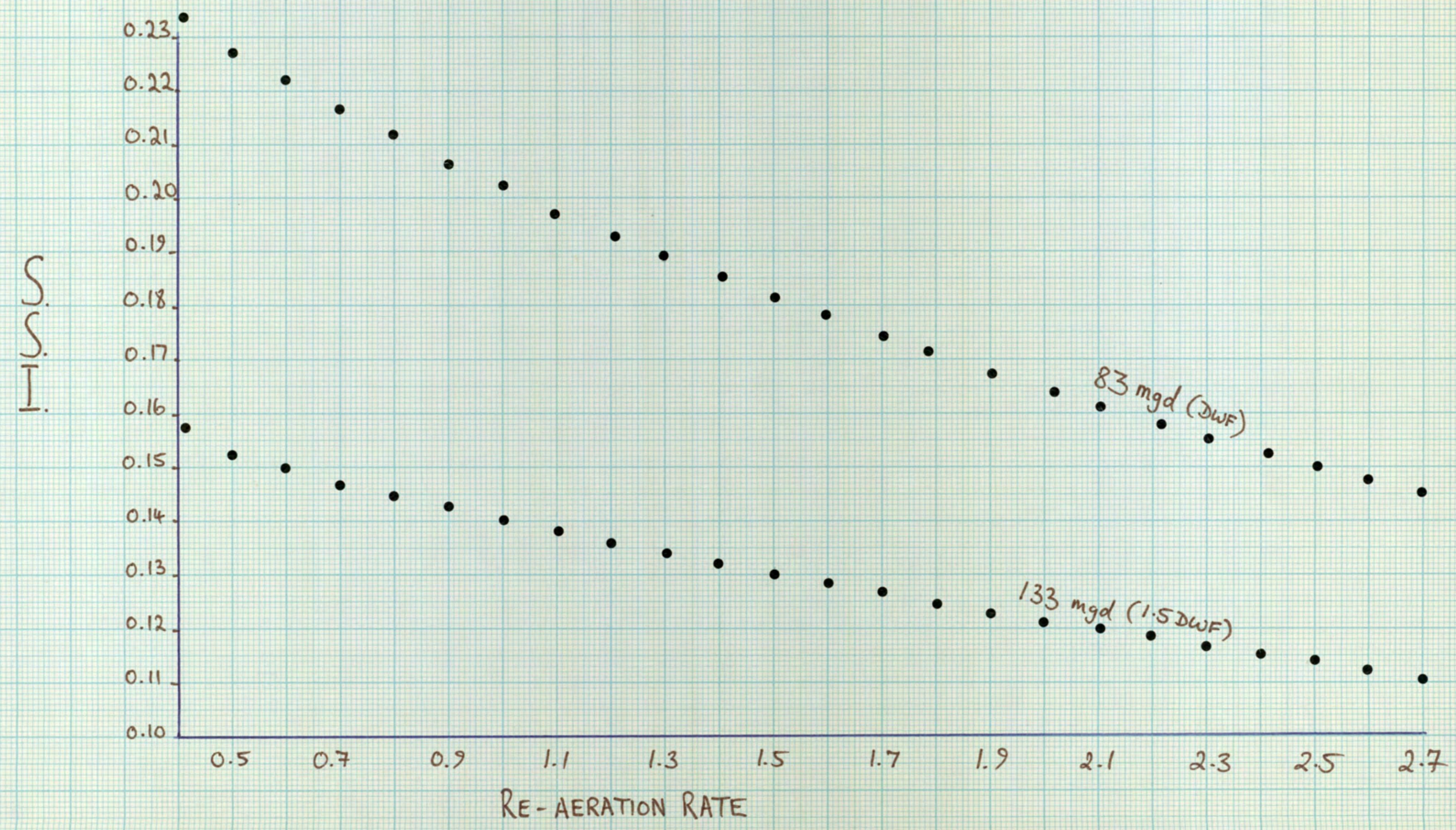


FIG 7.13.A D.O.% DISTRIBUTION FOR VARYING RE-AERATION
D·O·IN PER CENT SATURATION
110 0
100 0
90 00
80 00
70 00
60 00
50 00
40 00
30 00
20 00
10 00
0 0000
REAERATION RATE [per Day]
1·9 2·1
1·5 1·7
1·1 1·3
0·9
0·7
0·5
flow = 83 mgd (DWF)
DATA SET NO
0 0000
2·000
4·000
6·000
8·000
10 00
12·00
14·00
16·00
DISTANCE FROM NEWBRIDGE IN MILES

Fig. 7.13.B THE EFFECT OF RE-AERATION ON THE WHOLE D.O. PROFILE



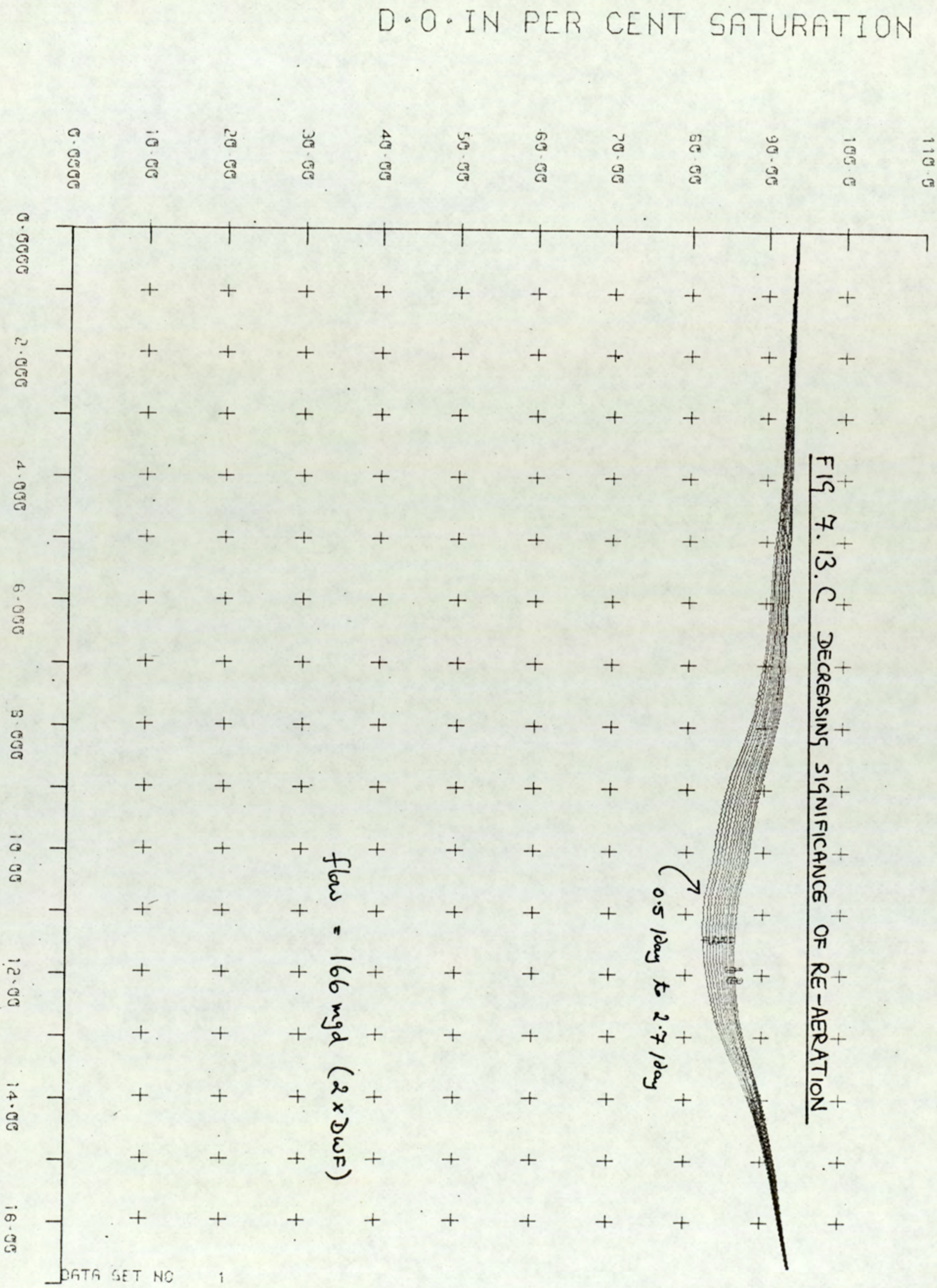


FIG 7.13.C DECREASING SIGNIFICANCE OF RE-AERATION

7.14 Seaward Boundary Conditions

Because of the large tidal prism of the Usk, the oxygen content of it is important as a source of dissolved oxygen.

The effect of simulating a reduction of the D.O. boundary condition from 7.46 (94% sat) to 6.5 (82% sat) is seen in fig. 7.14.A. The upper portion of the system remains unaffected. When the D.O. sag appears, the decline is steeper and the recovery shallower. The higher re-aeration rate appears to be less effective here as the rate is forced to the same end point irrespectively.

Comparing respective SSI's (0.1089 to 0.1621 and 0.0889 to 0.132) demonstrates the overall radical effect in that a 48.6% worsening of the index is predicted.

Similarly, increasing the downstream boundary condition marginally (94% to 100%) radically improved the overall SSI by 24.5% (0.1089 to 0.0843 and 0.0889 to 0.0655). The solution to the Usk Estuary pollution problem could well be the solution of the Severn Estuary pollution problem[14][15].

At lower flows the boundary condition becomes more important as a larger tidal prism is admitted and the penetration is greater (Table 7.14.A)

Similar effects are simulated on varying the upstream boundary. A 16% saturation reduction affects the SSI by 40% while lowering the minimum D.O. by 8.3%[16].

Table 7.14.A SSI at Different Flows for Low/High Boundary Conditions.

Flow	SSI low B.C.	SSI high B.C.
40	0.387	0.294
	(+21.7)	(+26.5)
55	0.303	0.216
	(+15.5)	(+20.4)
70	0.255	0.172
	(+11.7)	(+16.3)
85	0.225	0.144
	(+ 9.3)	(+13.9)
100	0.204	0.124
	(+ 7.3)	(+11.3)
115	0.189	0.110

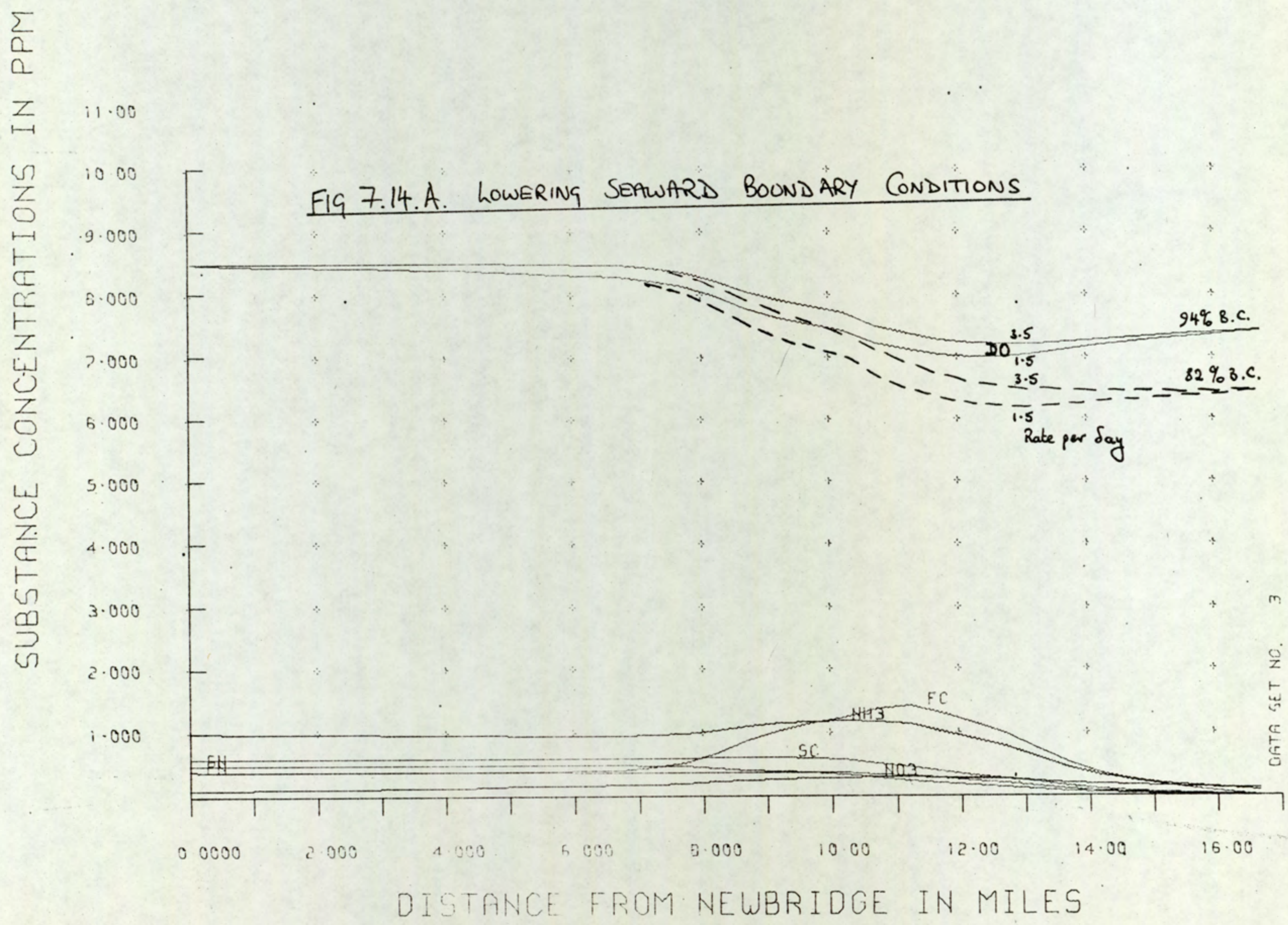
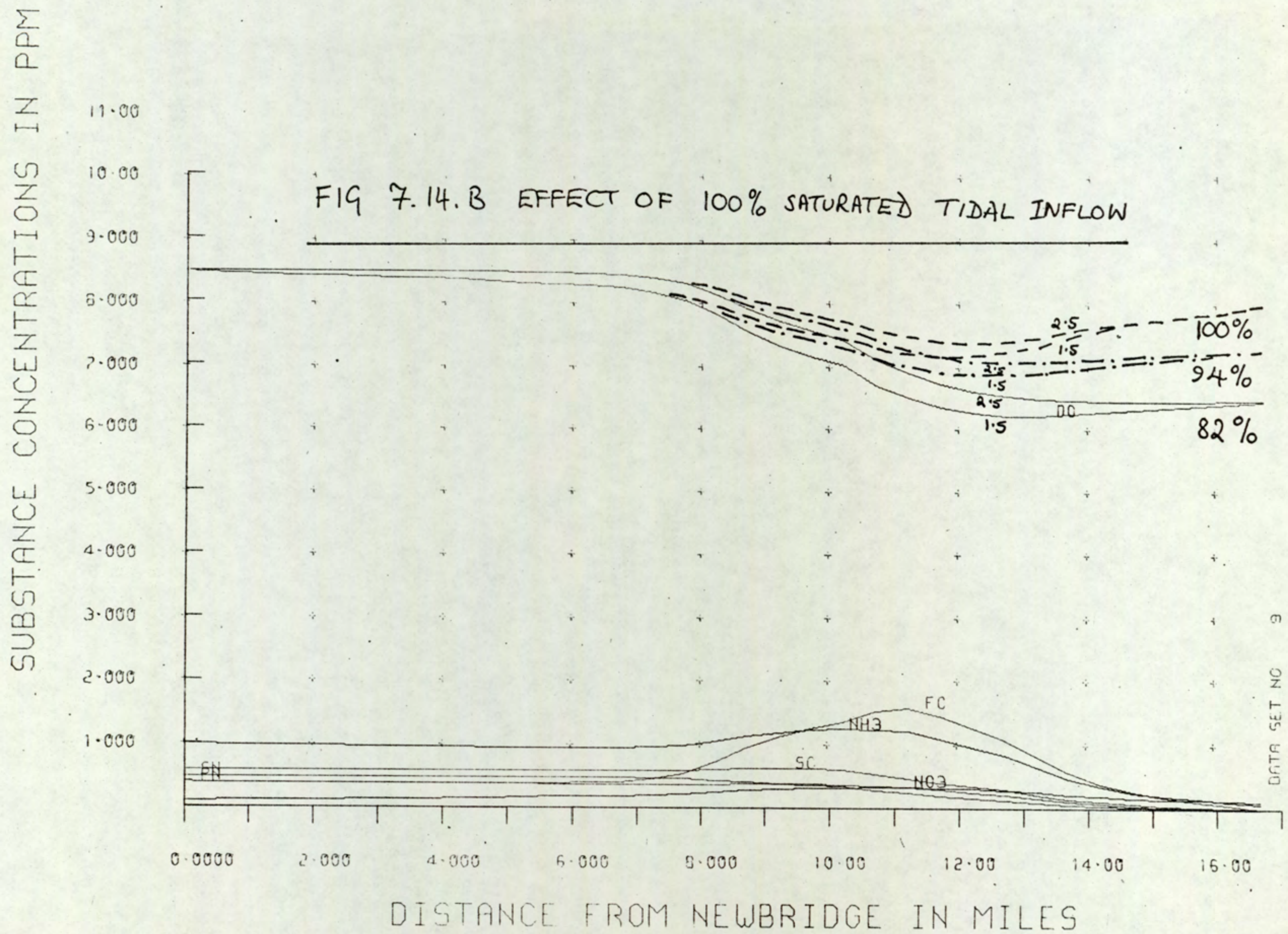


FIG 7.14. A. LOWERING SEAWARD BOUNDARY CONDITIONS
SUBSTANCE CONCENTRATIONS IN PPM
11·00
10·00
9·000
8·000
7·000
6·000
5·000
4·000
3·000
2·000
1·000
0·0000
2·000
4·000
6·000
8·000
10·00
12·00
14·00
16·00
DISTANCE FROM NEWBRIDGE IN MILES
94% B.C.
3.5
DO
1.5
3.5
82% B.C.
1.5
Rate per Day
FC
NH3
SC
NO3
FN
DATA SET NO. 3

FIG 7.14.B EFFECT OF 100% SATURATED TIDAL INFLOW



7.15 A Tidal Barrage

Correspondance about a possible tidal barrage across the Usk and /or Severn Estuary dates back to 1948 in the files of the old Usk River Board, predecessor of the U.R.A. and U.R.D. of W.N.W.D.A.

Recent advances in extraction of energy from water movement[18] and the high cost of conventional oil power since the oil crisis has stimulated further discussion.

A barrage situation is possibly the only case where this system will reach a true steady state.

The primary proposal for the Usk is for a sealing barrage to reduce tidal influences and maintain certain levels in the estuary.It is argued that this will help Newport's development as a South Wales Dock and have recreational benefits.

Simulations show that a partial barrage in the upper reaches is possible (in the upper 10 miles) without an appreciable reduction in the minimum D.O. although the SSI is increased by 5.4%. Were the lower reaches to be barraged, the cumulative loading of Newport's discharges would cause an estimated 5 mile anaerobic section in the lower half of the town, clearly reducing recreational benefit.

7.16 The Venture Carpet Factory[17]

In 1975 the URD received a major planning proposal for a new factory in the middle of Newport employing a large number of local people. The factory was engaged in processing fibres for carpet manufacture.

Outline effluent standards were submitted and the Pollution Control Dept. was able to recommend in a matter of days that , as the overall effect on the SSI was 1.2%, by ensuring that the effluent contained at least 30% dissolved oxygen , there would be no net effect on the estuary.

7.17 The Graig Goch Reservoir Scheme

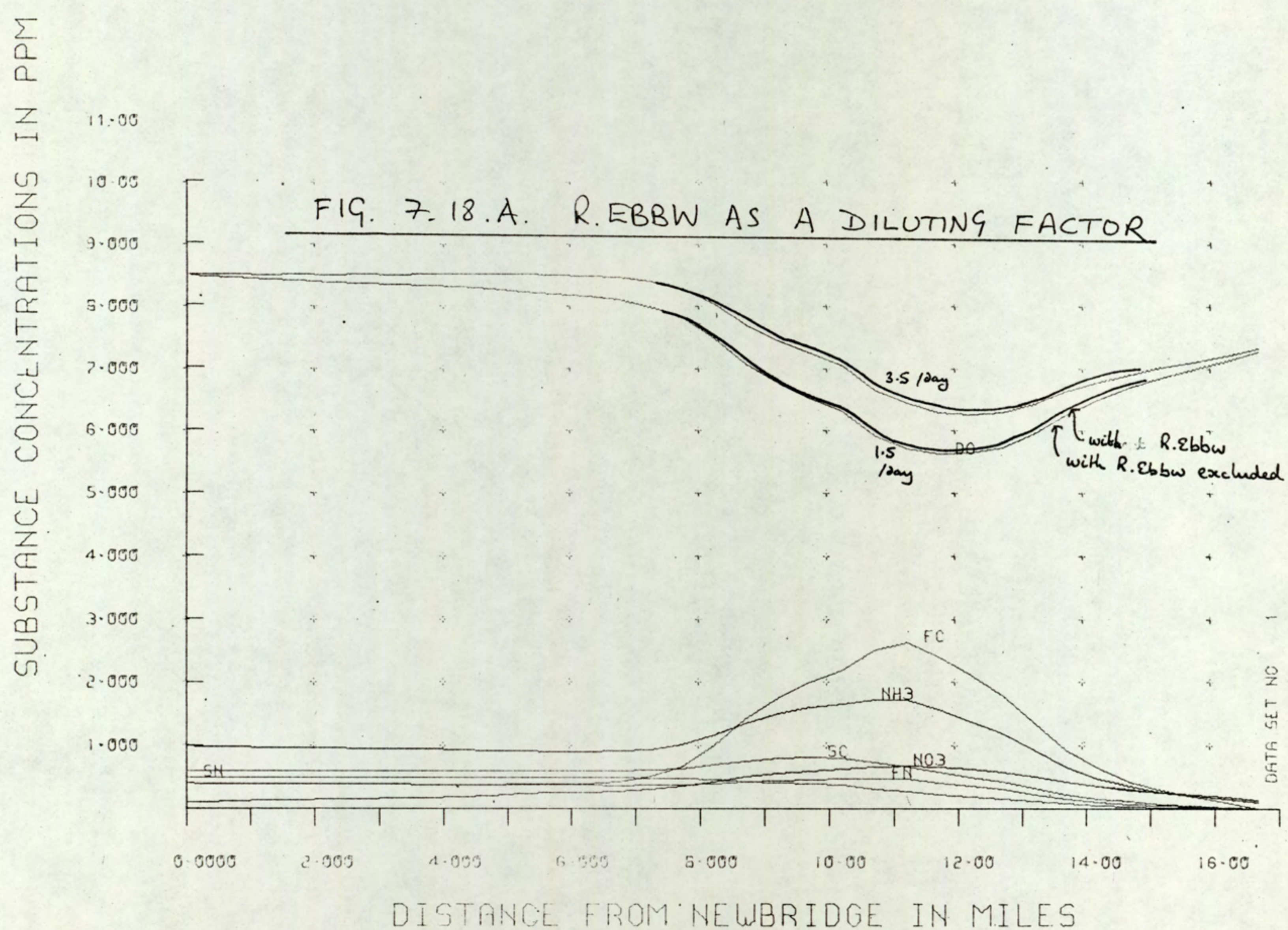
As the protected flow was fixed arbitrarily for the Usk, it was hoped that a large proportion of the flow could be used to supplement the inputs to the reservoir.Section 7.10 shows that low flow conditions will create problems. Whereas a reduction to 60-70 mgd may be acceptable, flows lower than this will for some time be totally unacceptable, until the Newport Main Drainage Scheme is fully implemented.

Summary Predictions for Very Low Flows with Newport Main Drainage Scheme Phase 1.

Flow	Re-aeration	D.O. min(%)	SSI
30	1.5	18.5	0.43
	3.5	45.5	0.24
40	1.5	36.4	0.33
	3.5	56.0	0.20

7.18 The Ebbw as a Diluting Influence

It was thought that the Ebbw was a useful dilution influence. Simulation of the discharges without the Ebbw was not greatly different, as fig. 7.18 shows. The overall SSI was raised by only 2.4%. The reason is principally that the River Ebbw water is fairly heavily loaded with pollutants from a domestic sewage discharge relative to other 'river' water.



URD STEADY STATE MODEL OUTPUT MWR

7.19 Eastern Valleys Discharge

The proposed increase of this discharge is the primary reason for the whole project. Initially discharging about 5 mgd at 90 ppm, a proposed increase to 8 mgd was desired in the first instance.

Figures 7.19.A to C show the increasing D.O. sag for 50%, 100% and 200% increases in loading.

Table 7.19.A Summarised Simulations for Eastern Valleys

		D.O.(min)mg/l				SSI			
Flow	Reaer-ation	Basic	*0.5	*2.0	*3.0	*0.5	*2.0	*3.0	Basic
83	1.5	5.7	5.5	5.3	5.0	0.191	0.200	0.218	0.180
	3.5	6.3	6.2	6.0	5.7	0.136	0.143	0.156	0.128
133	1.5	6.6	6.5	6.4	6.2	0.135	0.140	0.150	0.130
	3.5	6.9	6.8	6.7	6.6	0.105	0.109	0.117	0.101
183	1.5	7.0	6.9	6.8	6.7	0.112	0.115	0.122	0.109
	3.5	7.2	7.1	7.0	7.0	0.092	0.094	0.100	0.089

(* is load multiple)

Fig. 7.19.D illustrates that for most flows, a 50% increase in loadings can be tolerated if necessary. If some more dilution water were available then the situation would be noticeably better as the discharge reach is relatively small.

Above 50% increases, the SSI increases at low flows over the critical 0.2 mark.

Also, less room for contingencies would be available. For example, consider a raising of ammonia in the freshwater inflow. This could happen if works to the freshwater reach become non-operational due to industrial action. Fig. 7.19.E shows the cumulative effects of increased loads and high inflow ammonia. There is a distinct possibility of major industrial action.

FIG 7.19.A EASTERN VALLEYS PROPOSED DISCHARGE

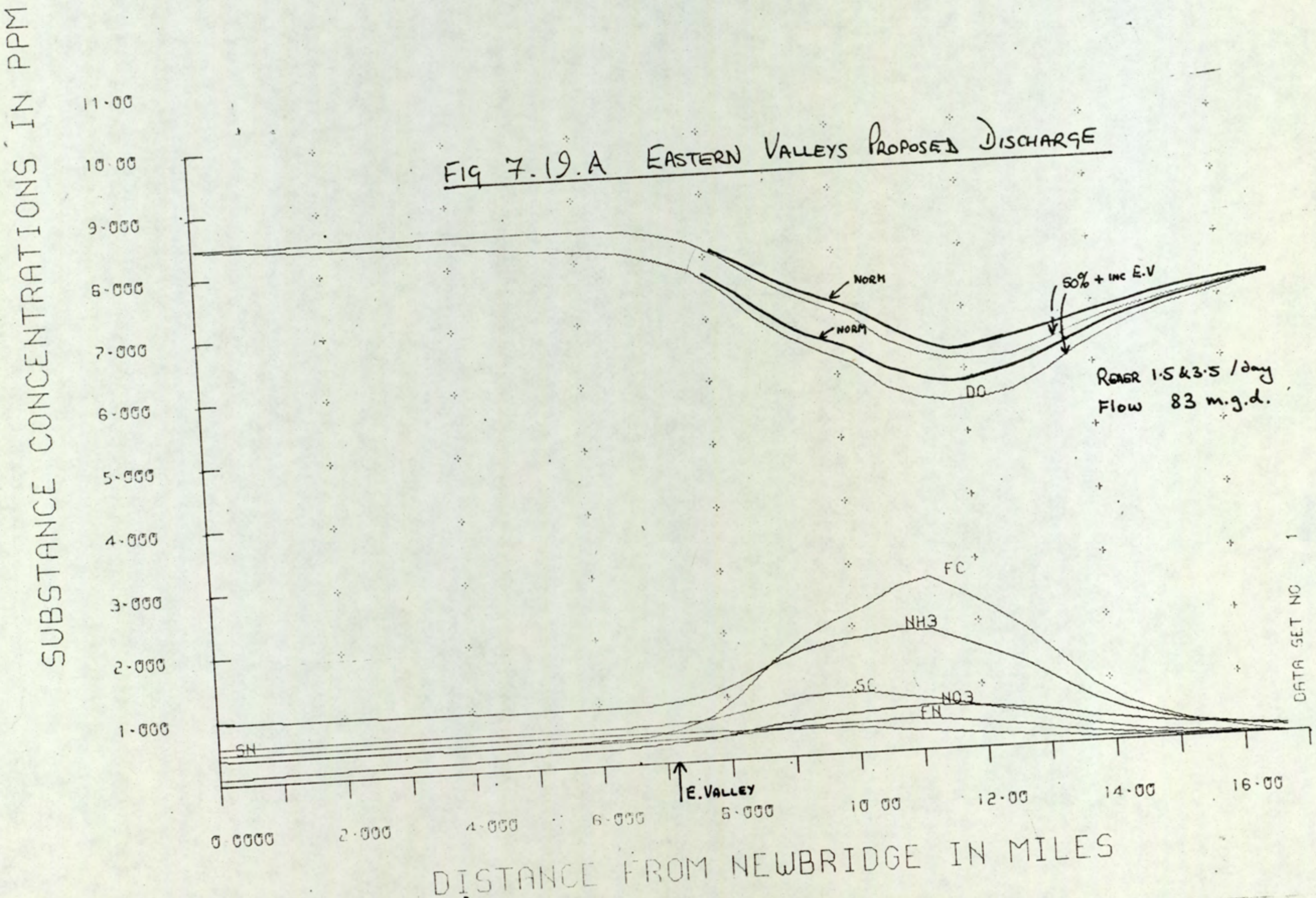
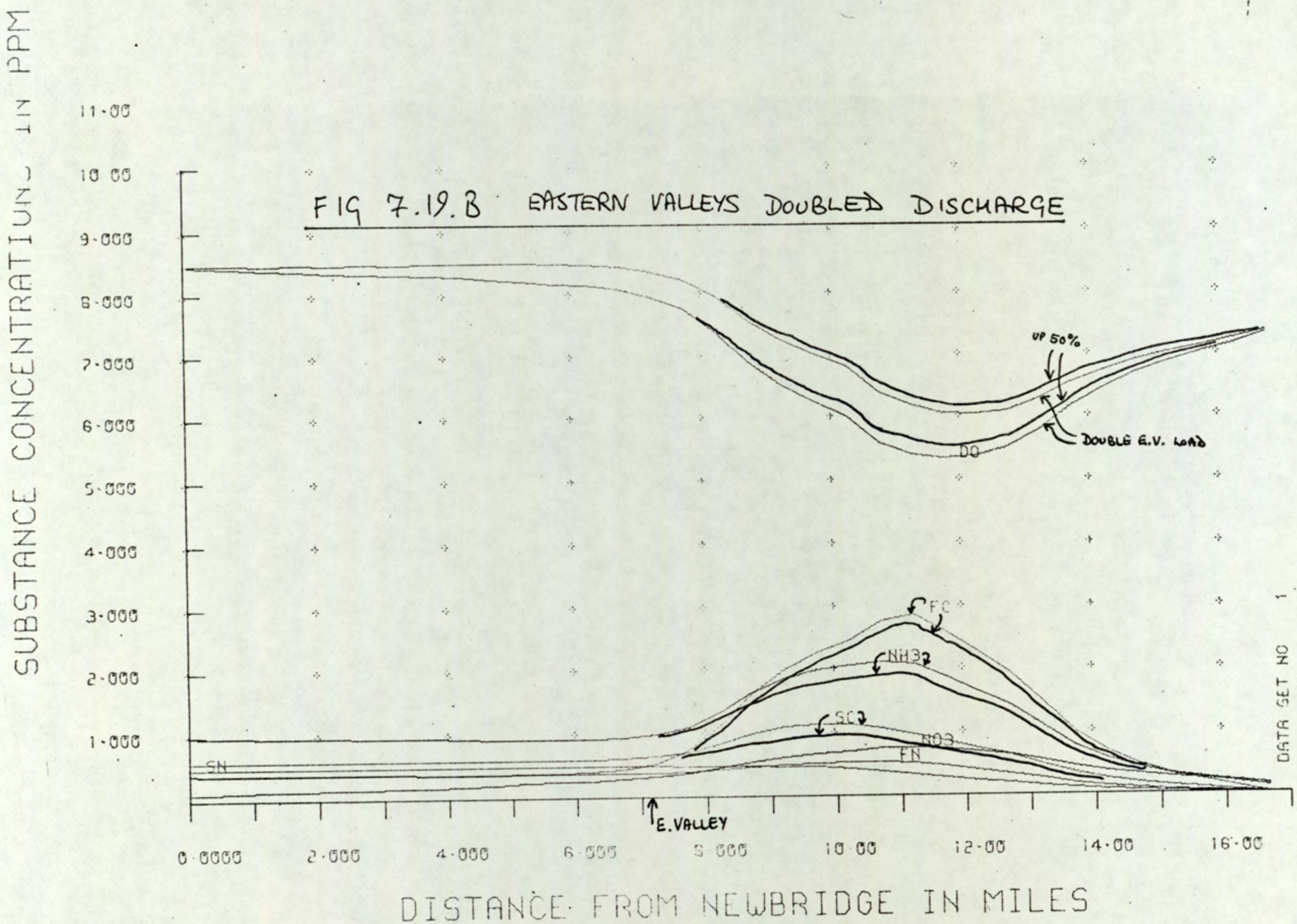


FIG 7.19.B EASTERN VALLEYS DOUBLED DISCHARGE



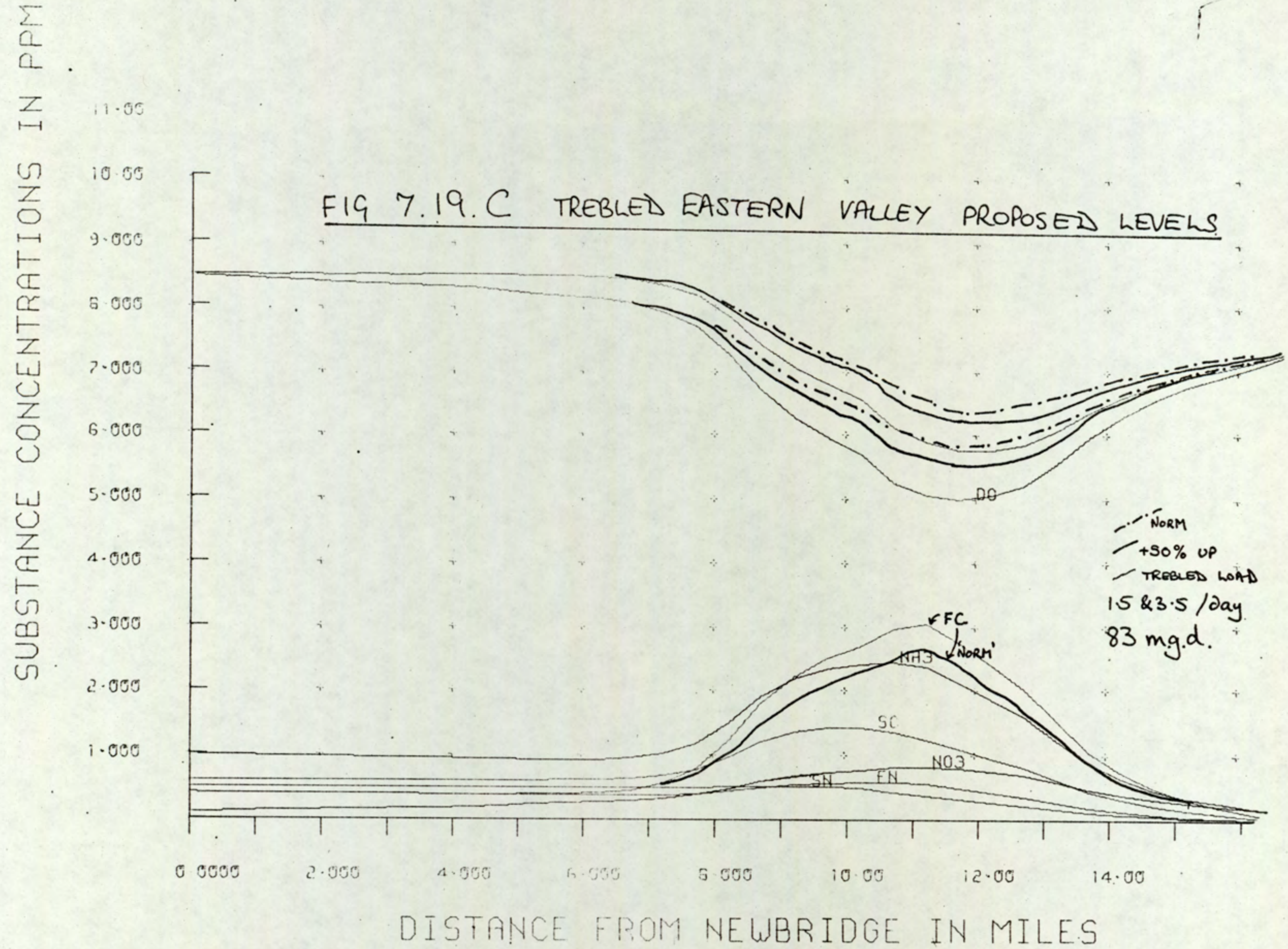


FIG 7.19.C TREBLED EASTERN VALLEY PROPOSED LEVELS

FIG. 7.19. D SAG SEVERITY INDEX FOR LOAD VARIATIONS FOR EASTERN VALLEY.

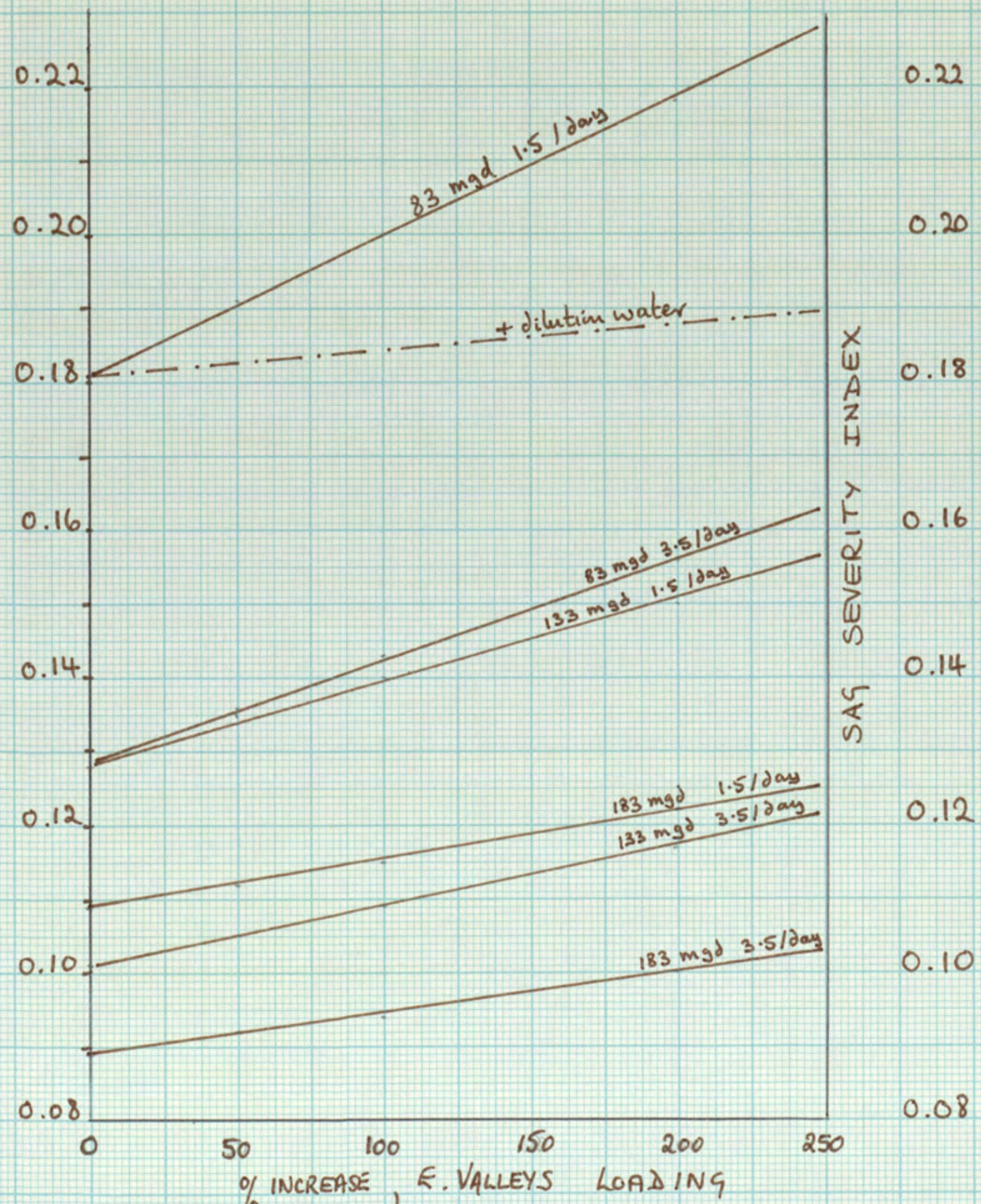
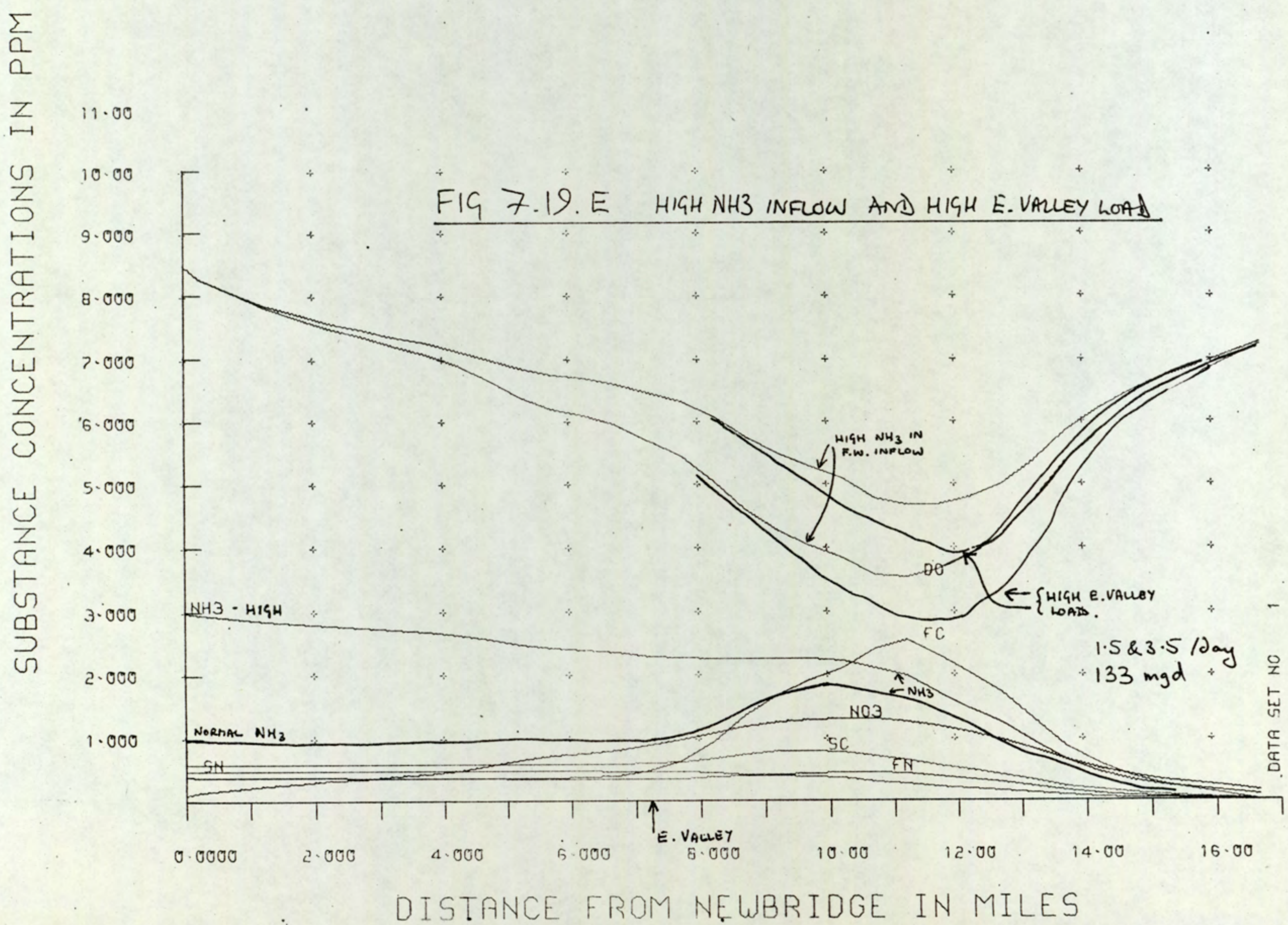


FIG 7.19. E HIGH NH3 INFLOW AND HIGH E. VALLEY LOAD



7.20 The Newport Main Drainage Scheme

As a long term solution to the pollution levels received by the estuary, Newport Borough Council formulated a multiphase plan to re-route each of the principal discharges within its jurisdiction (not Caerleon or Eastern Valleys).(The details are outlined in references in Chapter 6, numbers [10], [11],[130],[131].)

The stage by stage implementation shows a steady improvement of both the minimum D.O. levels (5.7 to 6.5 to 7.4mg/l) associated with a SSI improvement from 0.1807 to 0.1555 to 0.1019, i.e. a 43.6% improvement[16].

The works will be situated just upstream of the B.A.C. Water Quality Monitor and will offer primary treatment to all domestic sewage in Newport.

There are plans for a discharge point to be at the works, or for a small additional cost relative to the works, a discharge to the Severn Estuary.

Even with a discharge to the Estuary proper, a considerable improvement will be effected. Tentatively it could be argued that the pollution problem for the estuary has been resolved.

Unfortunately the continuing presence of the Eastern Valley outfall requires a maintained level of flow to ensure dissipation to reasonable levels.

7.21 Cost of Simulations

An experienced operator or manager will be able to generate data for most queries from a basic data set within 20 minutes of establishing contact with the system via EDITOR.

Table 7.21.A shows the actual times of some main simulations. The Time units are for an ICL 4-70, 768K store, running under MJ1500 (Multijob). The URD were required to pay 1p per time unit. The time taken includes digital plots of simulations as well as graph plots generated for off-line plotting. It is seen that one flow/ one reaeration rate simulation takes 5-10 time units, depending on the degree of tuning of the convergence parameters and the severity of simulation from norm.

For comparison, 1 time unit here is roughly 3 time units on an ICL 4.50 and 0.25 units on an ICL 19045, and 2 time units on an IBM 360 / 65.

The cost of a simulation can be reckoned as

30 minutes staff time (Grade 6-8), say	£ 2.00
Core time costs of several simulations	£ 2.00
Telephone charges	£ 0.90
Posting of output	£ 0.35
TOTAL	£ 5.25

This compares favourably with the cost of processing one typical estuarine sample at £10-£15.

Table 7.21.A. Core Time Requirements of Simulations

Simulation Name	Main loops (Flow)	Sub-loops (Reaeration)	Time units
Tidal Barrage	3	6	143.5
Eastern Valleys	9	18	499.2
Low Flows	8	16	429.3
Vary Temperature	6	12	322.6
500 year Drought	8	16	431.3
Reaeration	2	48	501.0
Low B.C. Flow	8	16	438.7
Basic Data	3	6	160.7

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Chapter 8

Applying the Time Dependant, Mixed Dimension Model

Chapter 8

Contents

8.0 Introduction

The development of the software for this model and its testing against published test data[1][2] took the major share of computer time and thus financial resource.The models required 46% of the budget, compared to th 14% of the Steady State Model, 10 Miscallaneous and 30% of the models ST/ST2. After extensive tests to establish correspondance, it was concluded that the results were driven by other data not available. Essentially results were sensible for the input data and for the theoretical system being considered[1]. The current state of the models is such as to be bulky and not geared towards general use. Volume of output can be prohibitive and expensive.

The two dimensional bay phase is based on a routine established by Leendertse used as a sub-model without internal iteration and variable input boundaries[3]

There was a distinct lack of management interest in this suite of models. This was voiced to be due to their complexity. It would eventually have to be used unsupported. There seems little effectiveness in a management information project that will not be used willingly. As a project develops, the question of management training should not be overlooked. So when the model is available it will receive use through acceptance and valid judgements and not with predjudice through non-comprehension and built in mistrust of the unfamiliar.

Again, the policy of using best data available is maintained for the reason that scarcity of field data would make tuning against any one parameter unreasonable for later simulations. Bed friction is the one exception in that coefficients for established types of bottom are vague within known limits.

8.1 The Severn Estuary Extension

The Severn Estuary can be considered to extend from the town of Gloucester (Maisemore Weir, just upstream of the town) to Lydney/Sharpness, where the two dimensional nature begins to predominate, then on to Penarth/Weston-super-Mare at 98km from Maisemore Weir. For this distance, the direction of the system is south-west. From Penarth/Weston the estuary continues due west until just beyond Nash Point(nr. Swansea)/Porlock Bay. Then the change in coastline of both banks open up the width to about 25km. This gives an estuary of 140km length (88 miles), of which the section kilometer 57 (Severn Bridge) to 103 (Penarth /Steep Holme Island/Weston-super-Mare) is of interest for the Usk Estuary[4], shown in fig. 8.1.A

Dpeths were abstracted from the continuously updated Admiralty Charts(Potters of London,Tower Hamlets) and British Transport Docks Board Soundings Charts[5] [6]. The area was split into square sections of 8000 ft. Note that overlapping charts are of different scales for sections of the system. For eachsquare, all soundings available were converted to a common level and averaged to give the mean interpoint depth.It is possible to put a case for using the mode depth here, were it not for the possible multiplicity of this statistic.For practical use, all depths likely to dry out during a simulation have to be lowered to remain wet. Fig. 8.1.B shows the mean depths using data from 1939 to 1972 charts. Likely changes occur through regular dredging of shipping channels and through the building of a port complex at Avonmouth.

Fig. 8.1.C shows a typical tide record available from Swansea, Avonmouth and Newport Docks , all of the BTDB(South Wales-West Area). Usually gauges are unattended for 14-28 days and as a result, records are cramped and often with a considerable cumulative error in the latter portion.Newport Gauge was not maintained and records were wholly illegible, whereas Avonmouth data was quite reliable on the whole. The illustration is for 1615 hrs. on 21/08/70 to 1540 on 15/09/70.

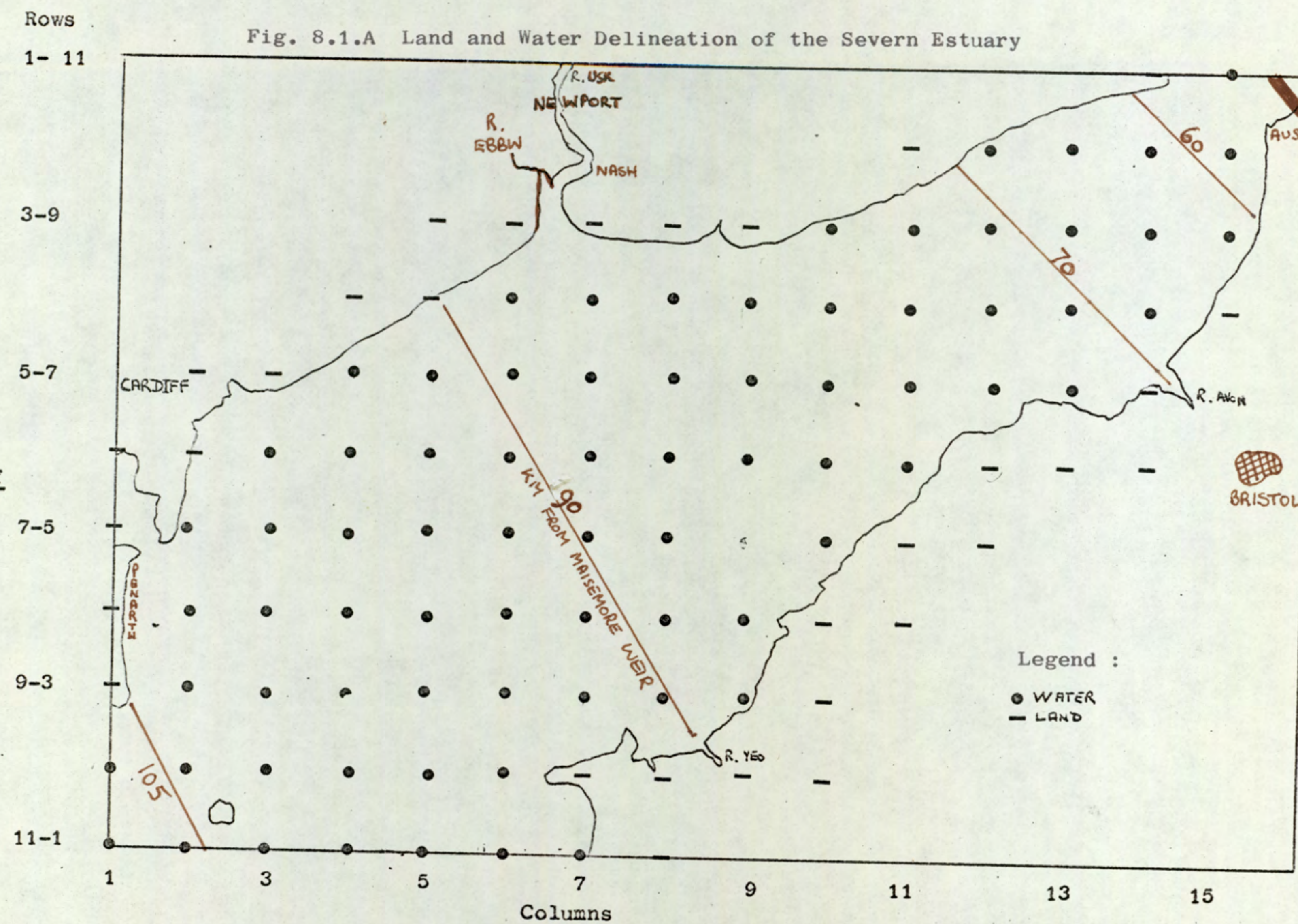
Fig. 8.1.D shows the mean tidal ranges and limits for spring and neap tides[4]. The broadly funnel shape of the Severn causes a net increase in tide height up to kilometer 50. This is the source of the Severn Bore[8], a wave front up to 6ft. high travelling from near Sharpness to Maisemore Weir at best. This is a characteristic of bays with depths about 70ft (20m) due to the oscillatory motion of the first node, amplified by noted width constriction[16]

Fig. 8.1.E shows the significance of the Usk and Severn in terms of potential development areas and thus flow increases.[7]Summarising potential development into three local areas gives

Bristol -	16-18 mgd	domestic	sewage	additional
Newport -	12-15 mgd	"	"	"
Cardiff Levels -	1.5-2.5 mgd	"	"	"

Development of any sort in a high unemployment area is a sensitive political issue. The population increase will require treated sewage capability before

Fig. 8.1.A Land and Water Delineation of the Severn Estuary



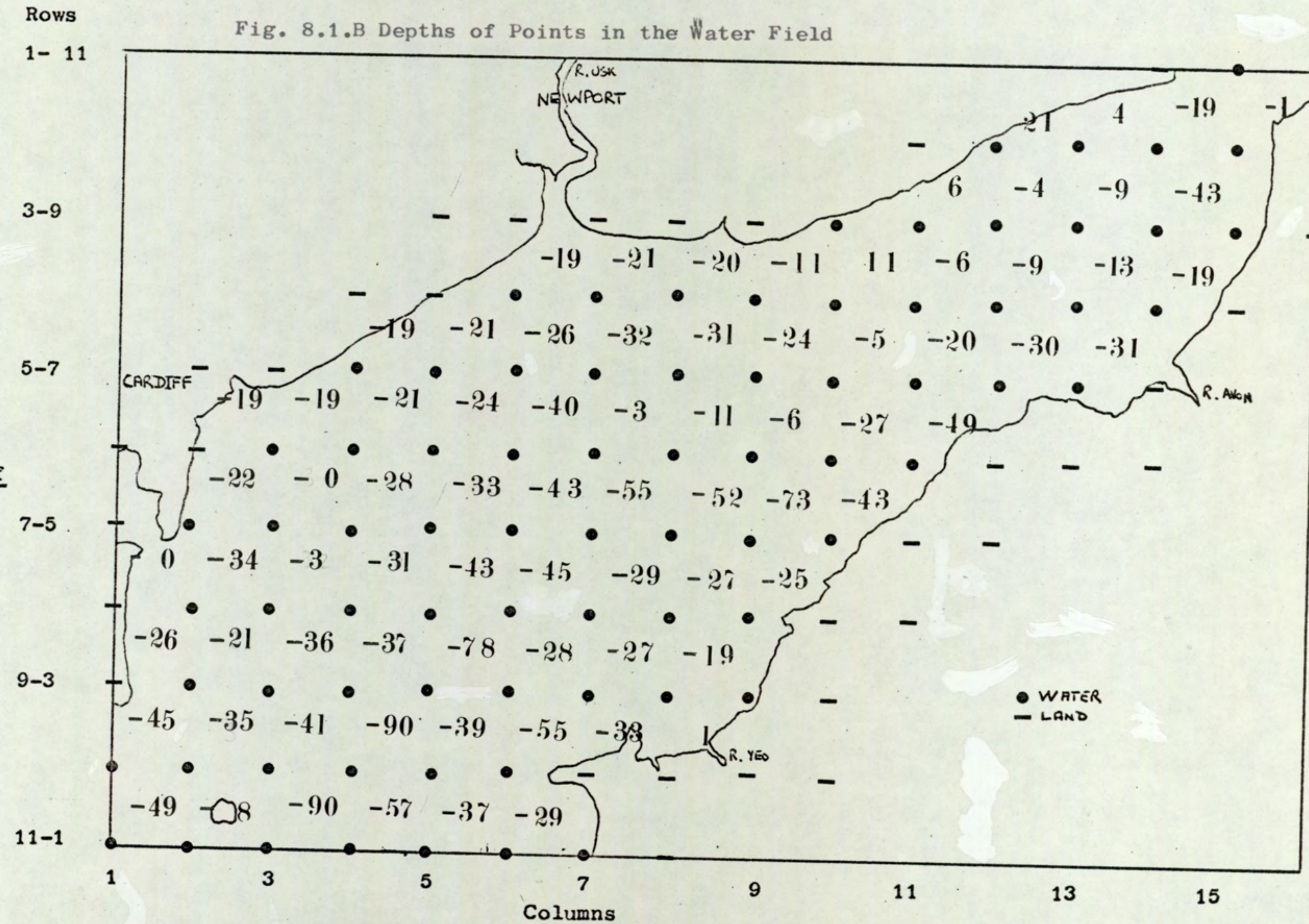
Fischer Model

Segmentation of

2D- Phase.

Fischer Model Segmentation of 2D- Phase.

Fig. 8.1.B Depths of Points in the Water Field



Note - depths of water field are in feet above mean water level

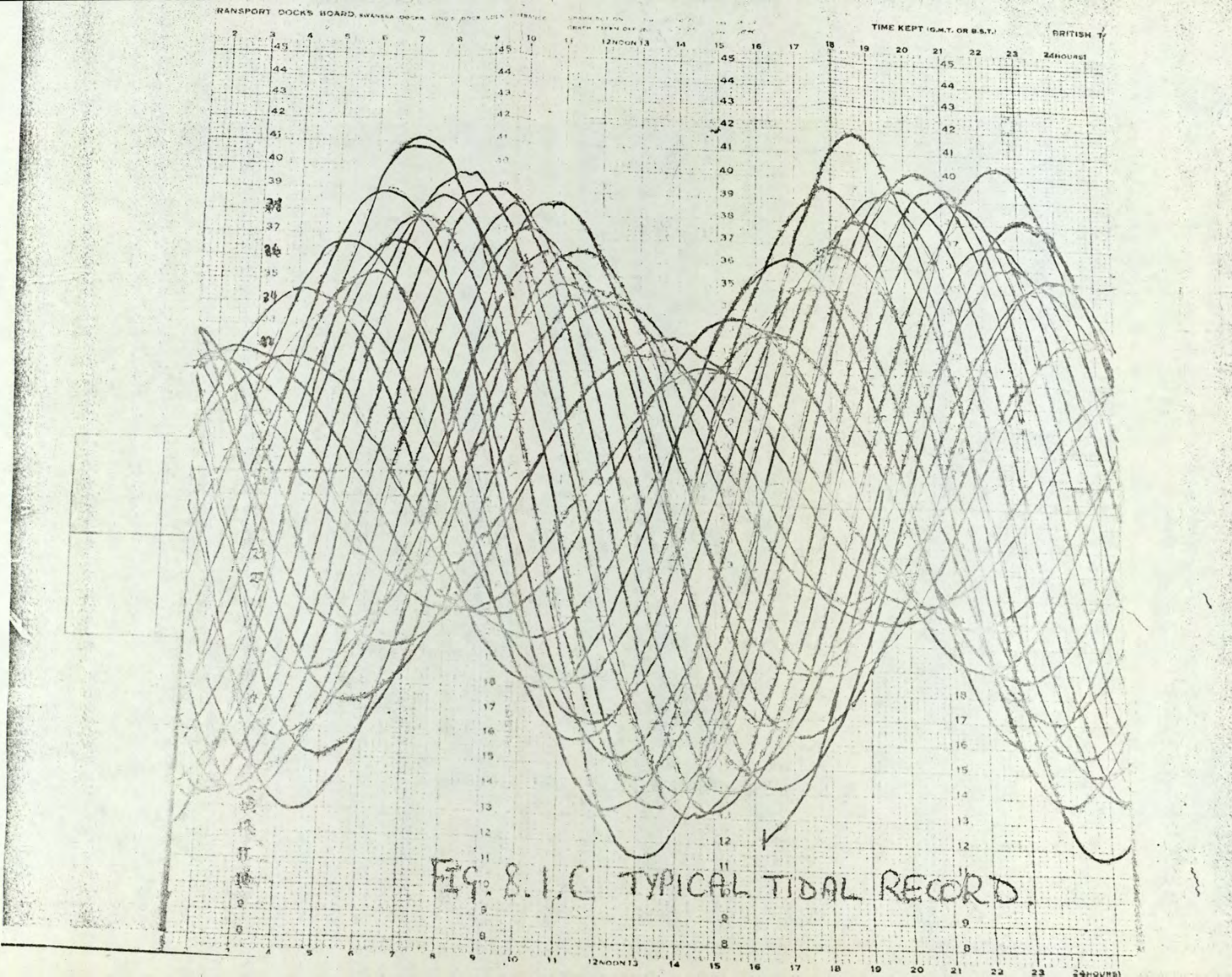
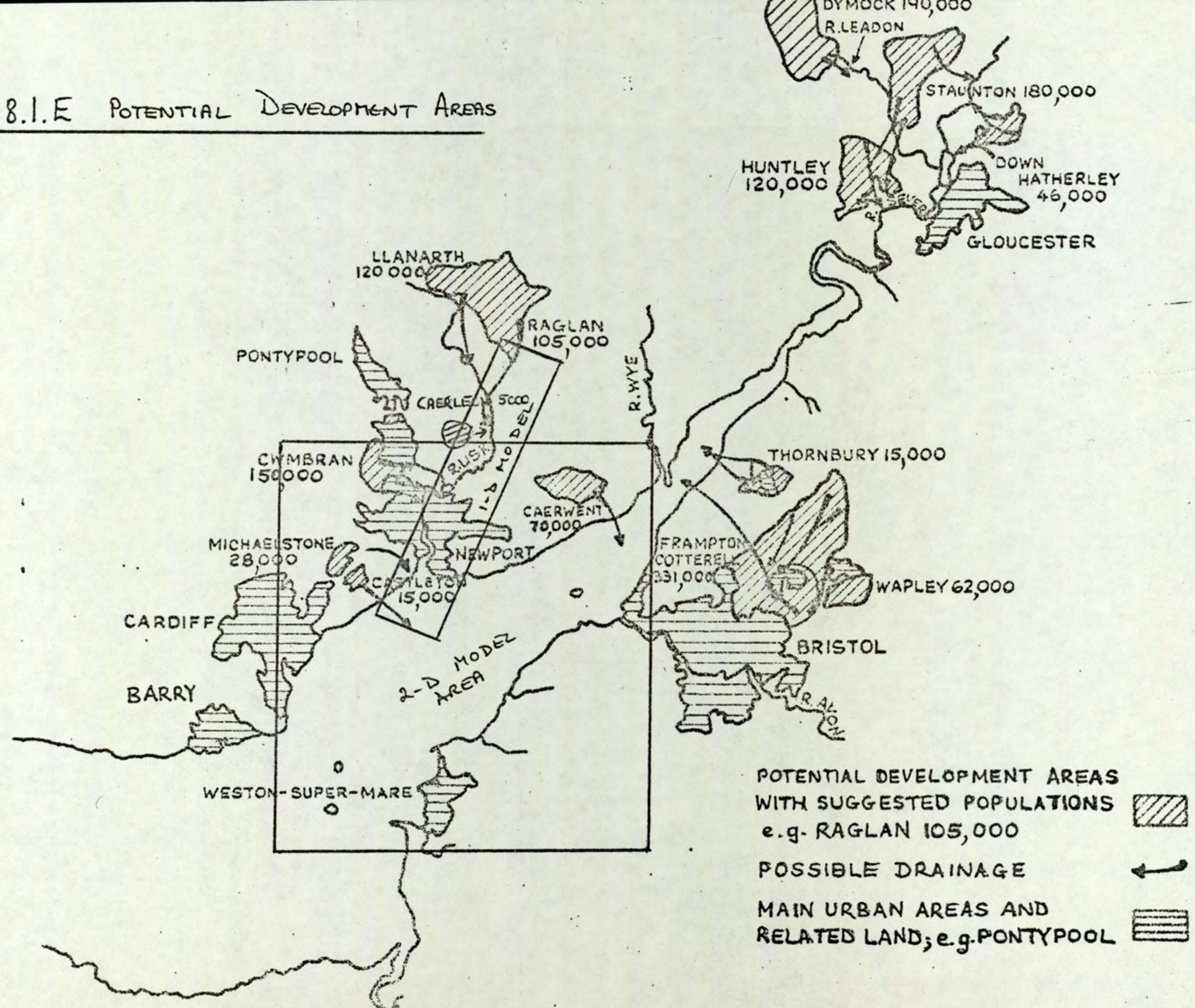


FIG. 8.1.C TYPICAL TIDAL RECORD.

Fig. 8.1.E Potential Development Areas



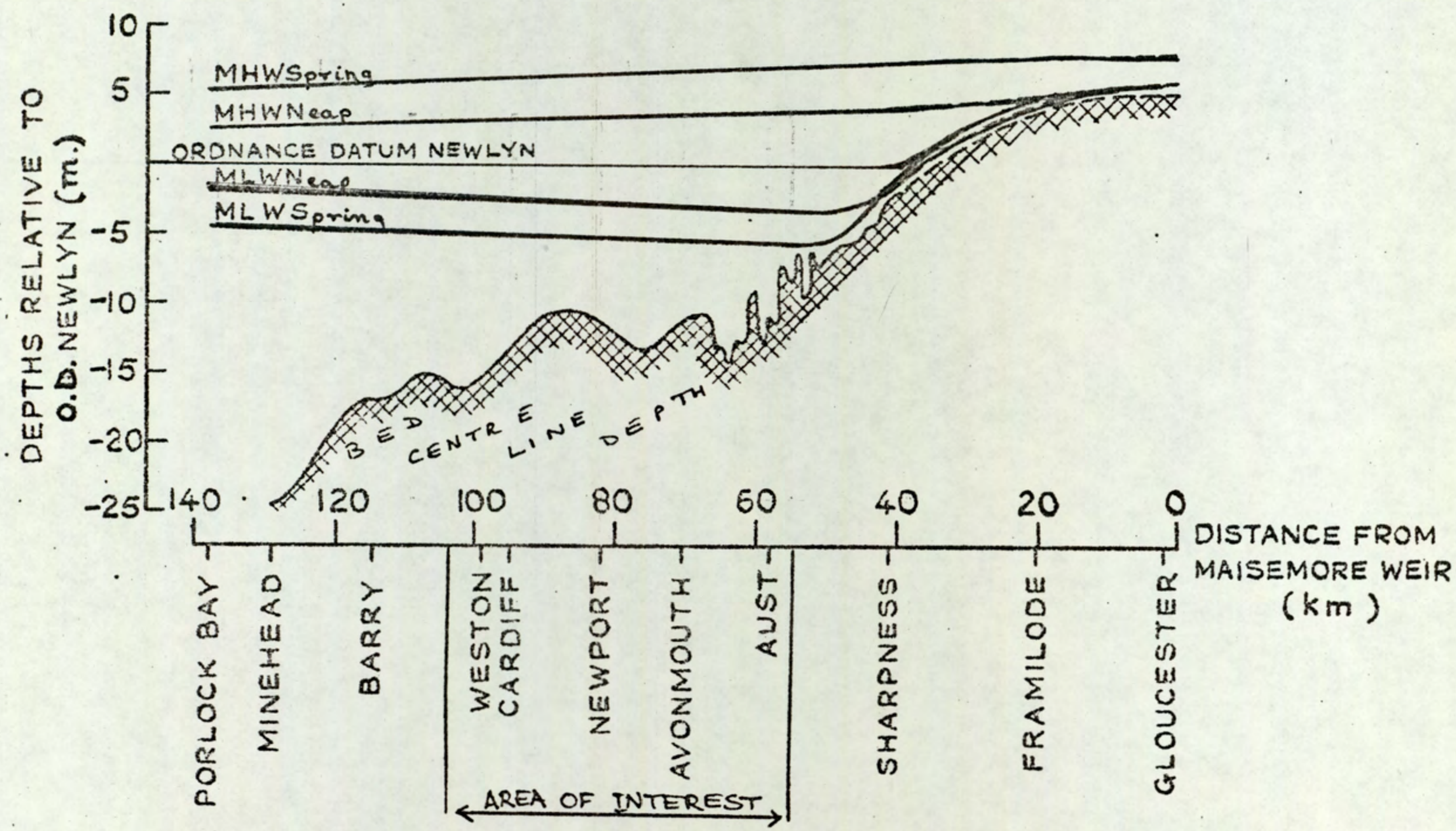


Fig. 8.1.D AVERAGE TIDAL RANGES - SEVERN ESTUARY

FIG 8.1.F

INPUT FOR BAY PHASE

RUN PARAMETERS & BOUNDARIES

```
00006800  +12.6 0. 0. 26 51.2 8000. 1 10 25. 2 7 0
00006900  6 4 7 4 1 10 1 11 2 11 3 11 4 11 5 11 6 11
00007000   13.1 12.6 11.1 9.1 6.3 3.18 0.02 -3.15 -5.98 -3.32
00007100  -10.4 -11.32 -12.4 -12.23 -11.07 -9.4 -7.4 -4.82
00007200  -1.9 1.27 4.1 6.77  9.27  11.18 12.6 13.1
```

WATER-LAND FIELD BOUNDARIES

```
00007300   0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
00007400   0 0 0 0 0 0 0 0 0 0 0 0 1 1 1 0 0 0 0 0 0
00007500   0 0 0 0 0 0 0 0 0 0 0 1 1 1 1 0 0 0 0 0 0
00007600   0 0 0 0 0 1 1 1 0 0 1 1 1 1 1
00007700   0 0 0 0 1 1 1 1 1 1 1 1 1 1 0
00007800   0 0 0 1 1 1 1 1 1 1 1 0 0
00007900   0 1 1 1 1 1 1 1 1 1 0 0 0
00008000   0 1 1 1 1 1 1 1 1 0 0 0 0
00008100   0 1 1 1 1 1 1 1 1 0 0 0 0
00008200  -0 1 1 1 1 1
00008300  -0-0-0-0-0-0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
```

DEPTHS OF WATER FIELD

```
00008400                                                     0    0    0   17
00008500                                                     0  +14  +14   19
00008600                       25  +14                     +14  +14  +14   20
00008700                 +14  +22   22   20  +14  +14   19   22   22   17
00008800            +14  +14   28  +14  +14  +14   35   48
00008900       +14  +14  +14   41   29   35   45   40   36
00009000  +14  +14  +14   41   29   35   45   40   36
00009100  +14   28   20   17   55   30   20   17  +14
00009200  +14   26   24   30   60   17   20   17
00009300   25   42   26   72   22   33   24  +14
00009400   33   52   78.  40   25   24
```

CHEZY FRICTION

```
00009500                                       60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00009600                                       60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00009700                               60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00009800                           60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00009900                     , 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00010000          60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00010100      60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00010200      60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00010300      60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00010400      60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
00010500      60. 60. 60. 60. 60. 60.         60. 60. 60. 60. 60. 60. 60. 60. 60. 60. 60.
```

BAY OUTLINE

```
00010600  52    1. 9.3    1.2 9.3    1.3 7.5
00010700        1. 6.5    2.1 7.2    2.7 6.3    6 3.4    6.5 3.4    6.6 1
00010800  6.7 1    6.9 3    8.8 4.2    10.4 4.2    14 1.1    14.4 1    15.6 1.75
00010900  15. 3.5    14.2  4.7    15. 5.4    15 5.6    14.1 4.9    11.8 5.5
00011000  9.5 8.    8.6 9.7    6.6    10.    7.    11.00000
```

they are resident. These models should help ensure the correct answer in terms of capital cost and accrued benefit to the commuinty is found before heavy planning committments are made.

Fig. 8.1.F shows a complete set of bay data for input to F1/F2.

8.2 De Chezy Friction Coefficient

To increase the accuracy of solution, the boundary effects are considered in the motion of fluid along a slope under gravity. A water mass moving with velocity u along a shallow slope α, at cross-section area A with wetted perimeter W and hydraulic radius d (= a/W) will move under the action of the gravity component in the vertical direction.

The compo nent of the weight along the line of the slope is balanced by the total frictional resistance, so

$$\alpha.g.A = k.W.u^{n} \qquad (8.2.A)$$

where k is a constant of proportionality, therefore

$$u = [\ \alpha.g.D\ /k]^{1/n} \qquad (8.2.B)$$

n was found to be nearly 2. De Chezy also observed that for fully turbulent flow in rivers the following held :

$$u = c\sqrt{D.\alpha} \qquad (8.2.C)$$

Consequently, the de Chezy friction coefficient C is defined as

$$C^{2} = 2.g\ /\ F \qquad \text{unit of } C^{2}\text{-m/sec}^{2} \qquad (8.2.D)$$

where F is a dimensionless friction coefficient. A more advanced formula is available :

$$c = \left[\sqrt{g}/k\right] \log \left[12.d/\beta\right] \qquad (8.2.E)$$

where d is water depth and β is a function of the heights of the bottom irregularities[9][10][11], k is the von Karman Coefficient[13] and roughly equal to 0.4. This requires greater verification effort to using 8.2.C

Values of the de Chezy coefficient recorded vary from 40 to 55 for a Rhine tributary[12]. For the silt beds of Rotterdam Waterways, values up to 60-65 are recorded. For the Severn, a common value of 60 $m^{1/2}$/sec. was selected, although lower values in the upper reaches may be more reasonable due to the nature of the bed there.

8.3 Bay Boundary Conditions

As the design was to allow any geographical form of bay, and because of the method of solution employed, elevation and stream points do not coincide. Consequently , some 20 special boundary conditions are recognised as being exceptions (fig. 8.3.A) , involving 12 land-bay boundaries and 8 with land,bay and open sea combinations. Boundary intersections can only occur at multiples of 90 degrees. All thses exceptions have to accomodated in the software and details of the precise numerical approximations are given elsewhere[14][15].

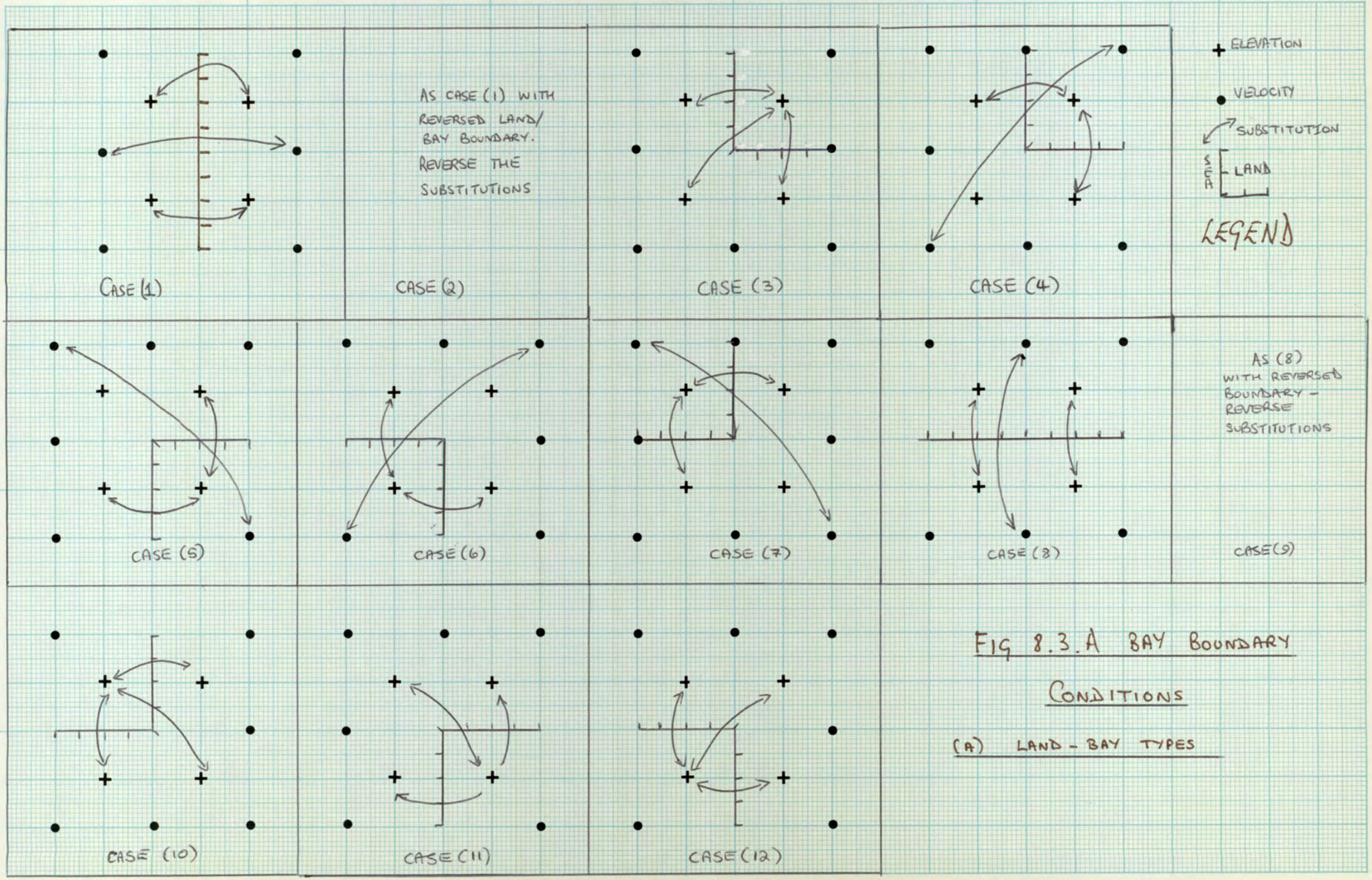


FIG 8.3.A BAY BOUNDARY CONDITIONS

(A) LAND - BAY TYPES

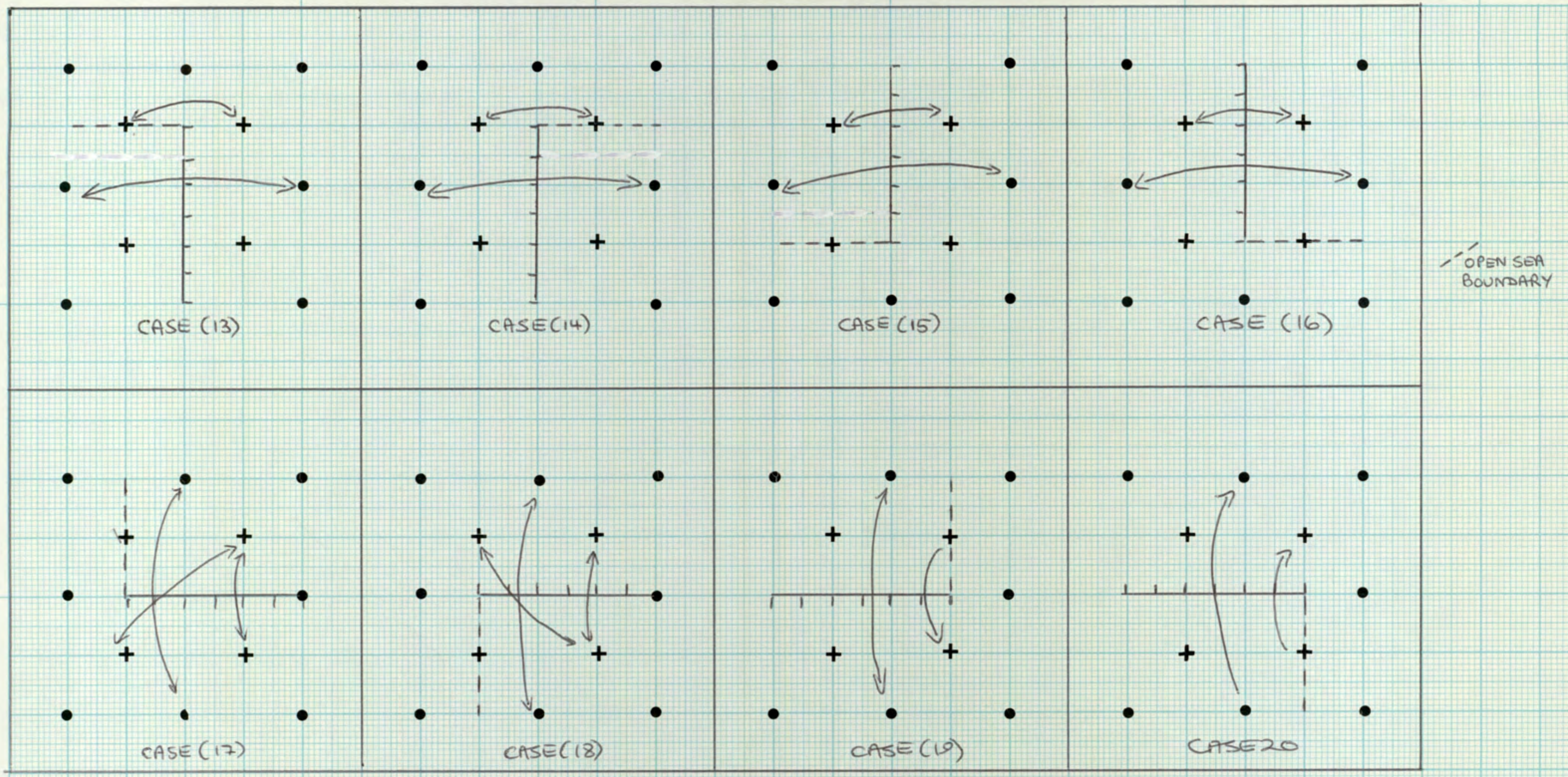


Fig 8.3.A BAY BOUNDARY CONDITIONS

(B) LAND-BAY-SEA TYPES

8.4 Usk Estuary Data for Hydrodynamic Phase

The whole data set is shown in fig. 8.4.A . The data set shown is for input to model F2, the full version with mixed dimension (Appendix B).

For model F1 a different forcing function is required , but the discharge boundaries are not required.

The bulk of the data consists of depth-width tables for each segment. Fig. 8.4.B shows the breakdown scheme of the Usk and Ebbw branched system. Arrows indicate the bank on which the feature is located.

8.5 The Bay Simulation

Initially , all elevations are set to 12.6 ft (specified by data input) in the bay. This is slightly higher than the initial tidal forcing function at 13.1ft The tide is at high water however, and so the 12.6ft level is a very stable starting point. Fig. 8.5.A shows that after only 2 hours of real time simulatio a high degree of stability and reaction exists. For many purposes, data from the 6th hour can be taken as stable if generated with a good initial approximation.

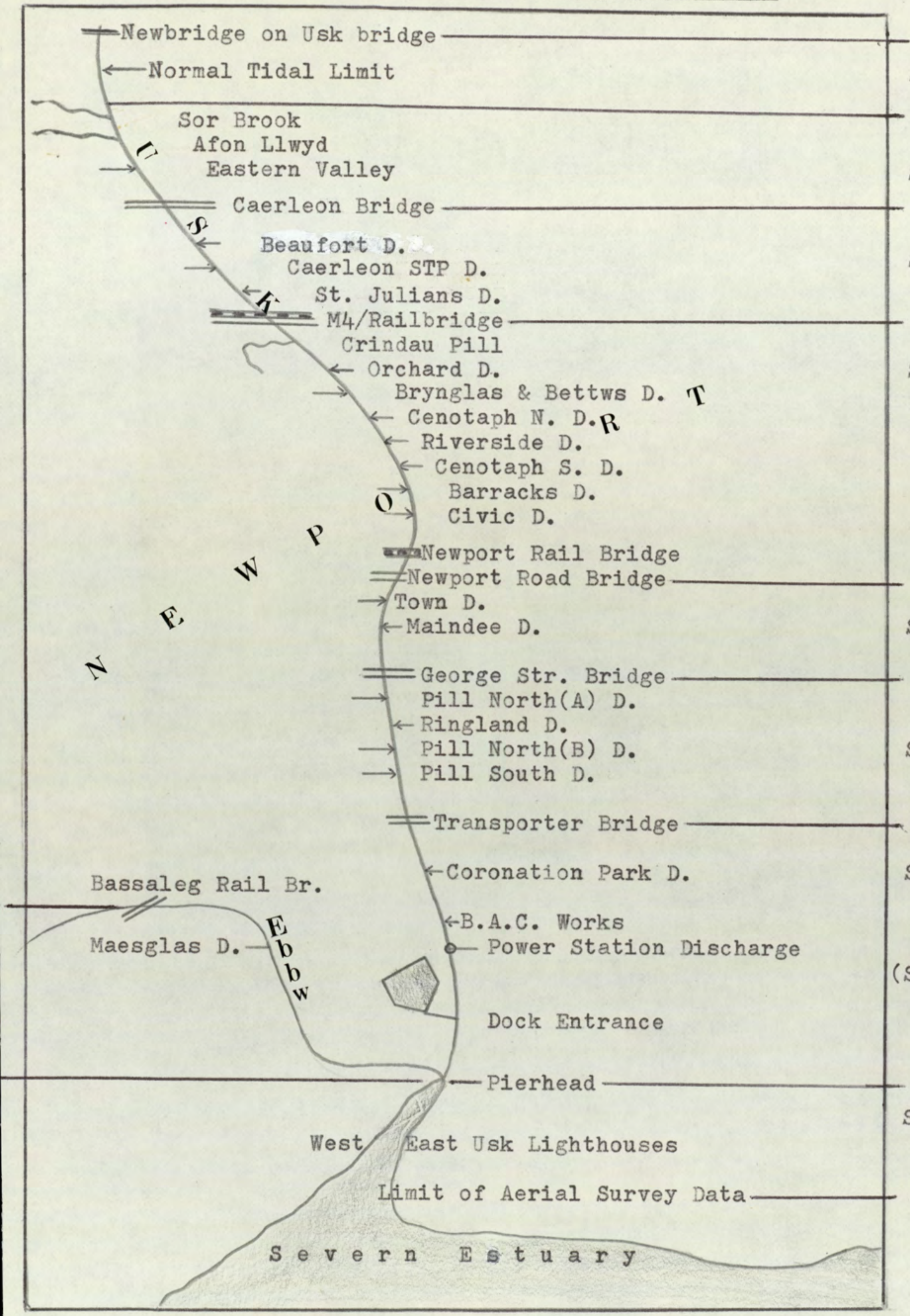
Fig. 8.5.B follows four selected points for a double tidal cycle of 25 hours. Interesting points are the time delay of high water at various points. These agree with published figures to within 15 minutes[4].Another point is the relative tide heights of high water at points moving towards the upper reaches of the bay [(5,11) and (2,15)]. It is observed that the heights are larger than the forcing function, and that the effect is accentuated towards the Severn Bridge . This is the beginning of the Severn Bore and correlates with observed effects (section 8.1).

```
 60. 1 3  1440   60 0.3 +12.6 0.0 96 98 BACKGROUND DATA
1 1 1 240 16 11
1975 11 20 13 05  DATE/TIME OF START OF SIMULATIONS
USK-SEVERN.13.1 FT AVERAGE TIDE AT BARRY=40.7 FT AT NEWPORT   TITLE
 60  1440  60    } TIMES OF PRINTING  BAY / RIVER
  60  1440  60   }   AND PLOTTING
 2880  1440   855
 1 1101061
 1 0306070                                                     MQBD ROW DISCH BDF
 1 0210110                                                     NOBD COL WATER BDF
 0     0                                                       NQBD COL DISCH BDF
    9      MMAX
     111      0.0       0.0       0.0       0.0       0.0       0.0      28      0.0 QMEAS
S1 NEWBRIDGE-CASTLEM 100     0    0     2    0     9      19812    3 .025   .0006915 -3
      0.0      125.5            0.7      125.85           SEGMENT A NEWBRIDGE-CASTLE M
      2.7      128.8            4.7      132.1            SEGMENT A NEWBRIDGE-CASTLE M
      6.7      135.4            8.7      141.7            SEGMENT A NEWBRIDGE-CASTLE M
     10.7      147.9           16.7      171.4            SEGMENT A NEWBRIDGE-CASTLE M
     26.7      260.6            100      260.5            LENGTH     FRICTION   SLOPE
S2 2 CAERLEON BRIDGE     1     0    0     3    0     9      13593    3 .025   .0005150  3
      0.0        143            3.7      146.4            SEG B  CASTLE MILL-CAERLEON
      6.7      153.7            9.7      164.4            SEG B  CASTLE MILL-CAERLEON
     12.7      176.7           16.7      197.6            SEG B  CASTLE MILL-CAERLEON
     20.7      220.9           26.7      263.8            SEG B  CASTLE MILL-CAERLEON
     33.7      324.0            100      324.0
S3 2 M4 BRIDGE           2     0    0     4    0     9      14916    3 .025   .0003750
      0.0      198.5            4.3      200.9            SEG C  CAERLEON BRIDGE - M4
      6.3      210.6            9.3      224.5            SEG C  CAERLEON BRIDGE - M4
     14.3      248.7           19.3      278.8            SEG C  CAERLEON BRIDGE - M4
     24.3      308.8           29.3      343.5            SEG C  CAERLEON BRIDGE - M4
     39.3      448.9            100      448.9
S4 2 NEWPORT TOWN RD     3     0    0     5    0     9      05406    2 .025   .0004069 1
      0.0      249.8            3.5      258.0            SEG D M4 TO NEWPORT ROAD BR
      6.5      267.2           11.5      300.5            SEG D M4 TO NEWPORT ROAD BR
     15.5      316.8           20.5      341.6            SEG D M4 TO NEWPORT ROAD BR
       26      370.3           31.5      397.1            SEG D M4 TO NEWPORT ROAD BR
     41.5      491.7            100      491.7
S5 2 GEORGE STR          4     0    0     6    0     9      03458    2 .025   .0005784 1
      0.0      244.8            5.5      262.5            SEG E NEWPORT BR TO GEORGE
      9.5      289.4           13.5      316.4            SEG E NEWPORT BR TO GEORGE
     17.5      339.7           22.5      377.0            SEG E NEWPORT BR TO GEORGE
     27.5      402.3           33.5      421.8            SEG E NEWPORT BR TO GEORGE
     43.5      450.1            100      450.1
S6 2 TRANSPORTER BR.     5     0    0     7    0     9      07310    2 .025   .0004788 1
      0.0      267.7              4      280.25           SEG F GEORGE STR-TRANSPORTE
        9      323.9             14      380.45           SEG F GEORGE STR-TRANSPORTE
       19      427.8             23      472.05           SEG F GEORGE STR-TRANSPORTE
       27      511.9             37      561.70           SEG F GEORGE STR-TRANSPORTE
       47      609.4            100      609.4
S7 2 PIERHEAD            6     0    0     9    0     9      13079    4 .025   .0005887 2
      0.0      385.7            4.7      409.4            SEG G TRANSPORTER-PIERHEAD
      9.7      458.3           14.7      547.9            SEG G TRANSPORTER-PIERHEAD
     19.7      790.3           24.7      956.4            SEG G TRANSPORTER-PIERHEAD
     34.7     1230.1           44.7     1518.5            SEG G TRANSPORTER-PIERHEAD
     54.7     1786.4            100     1786.4
S8 EBBW TO MAESGLAS    100     0    0     9    0     9      13343    4 .025    .003163 2
      0.0       70.8            4.7       70.8            SEG I  EBBW
      9.7       70.8           14.7       90.95           SEG I  EBBW
     19.7      111.1           24.7      131.25           SEG I  EBBW
     34.7      167.9           44.7      258.35           SEG I  EBBW

MULTIJOB PRINT OF URDOOL;URD01M.F2DATA(S20006)/N  FOR USER URDOOL AT TERMINAL 16

     54.7      577.2            100      577.2
S9 PIER TO A.SURVEY      7     8    0     0    0     9      04677    2 .025 .00006414 2
      0.0      606.5              5      677.1            SEG H PIERHEAD-BASE LINE
       10      769.7             15      949.8            SEG H PIERHEAD-BASE LINE
       20     1383.5             25     1865.0            SEG H PIERHEAD-BASE LINE
       35     3744.5             45     4928.5            SEG H PIERHEAD-BASE LINE
       55     5966.0            100     5966.0
```

FIG. 8.4.A COMPLETE DATA FOR F2

FIG 8.4. B

Schematic Segmentation of the Usk Estuary System



It should be noted in the delineation of the bay for the upper reaches , a generously wide water field was allocated. Strictly only one point should be allocated. However, this so accented the bore effect and also as it was a reflective boundary (which in reality it is not) oscillations set up that soon swamped the whole are and the method failed.

The forcing function is input along the lower left (south-west) edge and the lower left vertical boundary (west). Considering the depth structure of the bay and practical knowledge, simulations were expected to show that the main part of the tide stays on the south west shore. Fig. 8.5.C shows the profile along selected columns at input high water for maximum flood for most points along the system. Tidal effects are emphasized along the south-west rows as anticipated (higher row number)

A complete cycle of elevations at all points is given in fig. 8.5.D (6 parts). As the bay is not of primary interest for this project, validation is needed only in the broadest terms. For a similar tide observed at Avonmouth, predictions were within 30 minutes and within 2.5ft of observed data.This was considered reasonable as the simulation had no wind effects, whereas the day of the tide had a fresh breeze (500 run miles). Any disagreement can be tuned by use of the de Chezy coefficient, in the region and the stream leading to it.

The value of this phase is when the whole Severn region is to be modelled.

A working party to gather cohesive data drawn from the three principal Regional Water Authorities (Severn Trent, South West, Welsh N.W.D.A.) already exists with a view to a concerted policy for the area. Currently laboratories are being standardized for analytical techniques and cross-correlation.

Initial problems were encountered through drying out. It was thought the method could accomodate virtual water blocks. The tempoary expedient: lowering the depths of such points to ensure their continued 'wetness' throughout the cycle.This obviously affects elevation predictions and velocity projections Velocities would in general be predicted low through this, similarly for the elevations. Were the bay to be modelled accurately, this would not be acceptable The solution is to use a one-step look ahead algorithm. This performs the calculation of elevation at each point as previously, but then checks the available depth. If the elevation is too negative, the point has dried out. The step is retracted and the point set to a land point for the time being. All steps in the bay are checked this way. If no drying out is sensed, the step is accepted and the method proceeds as before. If drying out is sensed, the step is retracted and then repeated with the new tempoary land-sea delineation. The additional computing effort required can vary considerably, but will be dependant on the ratio of drying area to total area. In the Severn the ratio is 0.45 to 0.75 in certain areas depending on the tides, and so it is anticipated that additional computing effort will be of the order of 50% to 70% , as well as additional core store overheads.

The primary area of interest is the bay/estuary interface about column 7. This feeds the one dimensional phase with forcing functions. As mouths are generally constrictions in reality, the area is again slightly enlarged to encourage stability. Furthermore, the interface is not merely the immediate point on the edge of the estuary, but is defined as average values over any number of relevant neighbouring points. Fig. 8.5.E (2parts) shows the 20 hour period hour 39 to 59. The flow rate is in cubic feet per second, velocity in ft/sec. , height in feet. The velocity will bear no relevance to any measured in the tidal race channel , as it is the smeared average over several points some of which are imaginary, because of the drying out. This table is part retained in the file set up by F1 and later used as input to PT or PT1. The ebb and flood duration of 7.2 , 5.3 hours respectively agrees well with observed figures at Newport Docks.

Fig. 8.5.F (12 parts) are the predicted velocity components in the x and y direction for a complete tidal cycle. There is no validation data available for this data, but the elevation observation correlations will suffice.

Fig. 8.5.G (8 parts) are the plotted distributions of velocity after 3,6 hours and the cycle 112-138 hours. Comparing the earlier output with the latter again shows how rapidly the expedient choice of initial conditions can assisst in reaching stability.

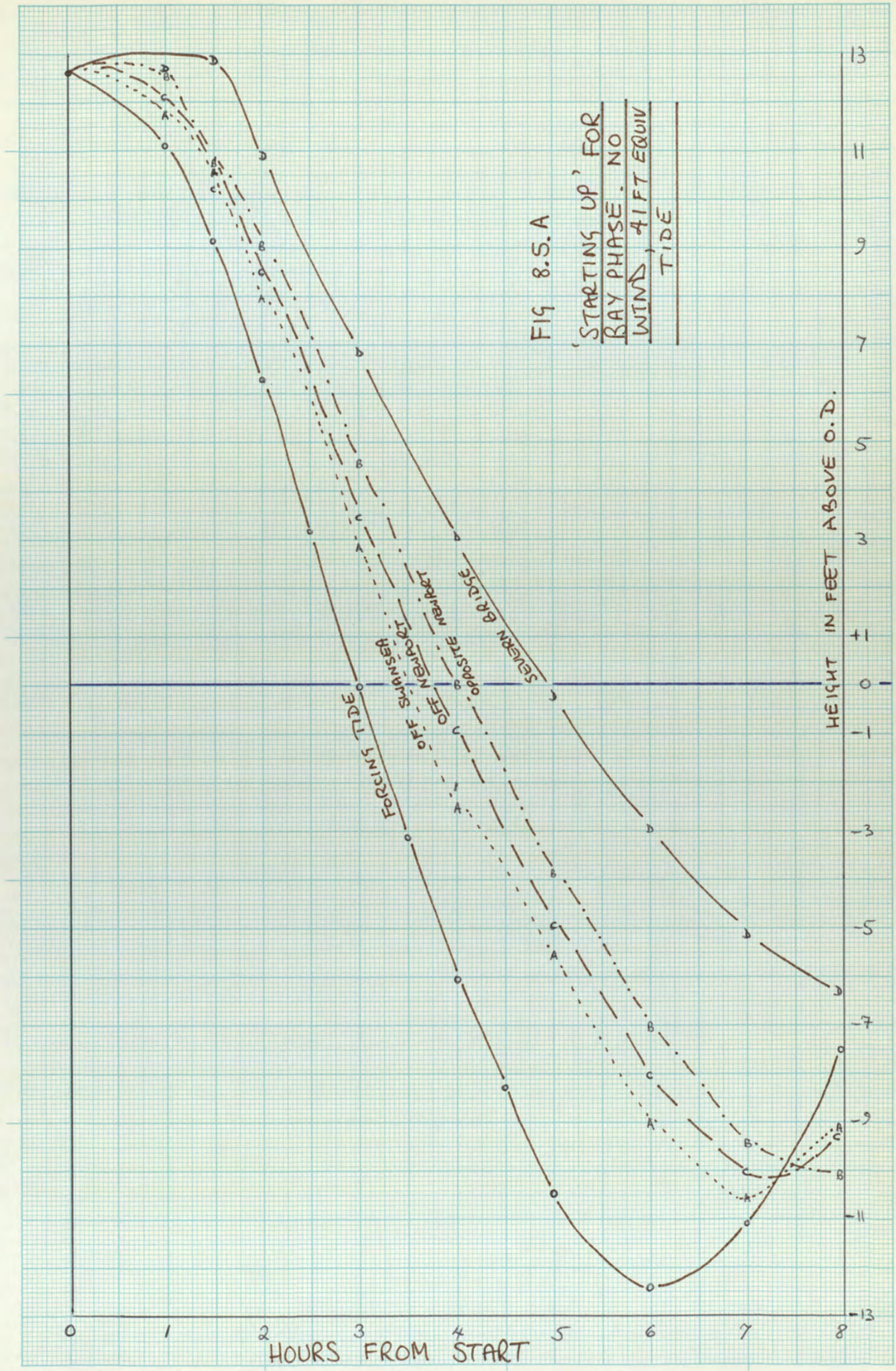
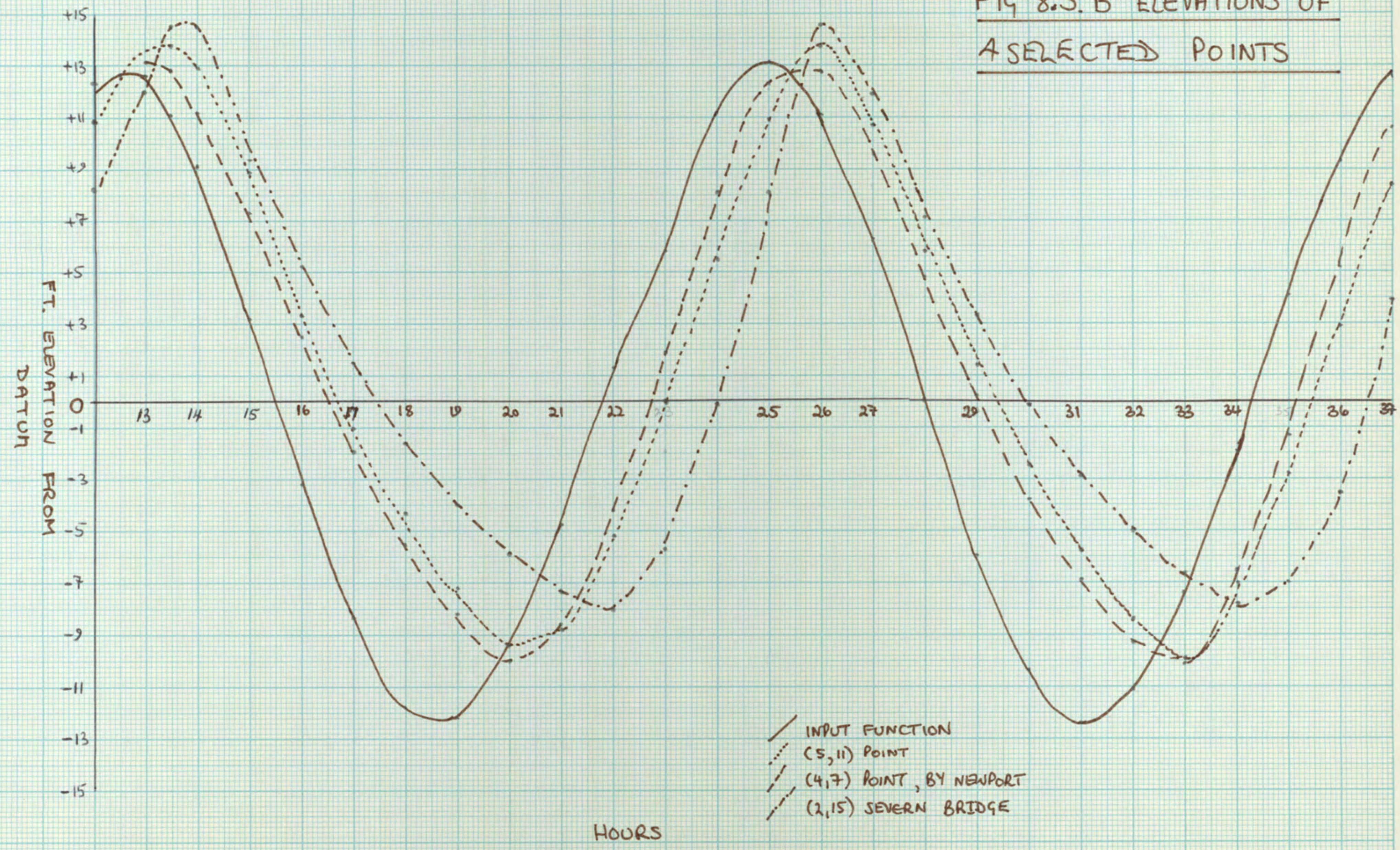


FIG 8.5.A
'STARTING UP' FOR
BAY PHASE. NO
WIND, 41FT EQUIV
TIDE
HEIGHT IN FEET ABOVE O.D.
HOURS FROM START
FORCING TIDE
OFF SWANSEA
OFF NEWPORT
OFF NEWPORT
OPPOSITE
SEVERN BRIDGE
0
1
2
3
4
5
6
7
8
13
11
9
7
5
3
+1
0
-1
-3
-5
-7
-9
-11
-13

FIG 8.5.B ELEVATIONS OF A SELECTED POINTS



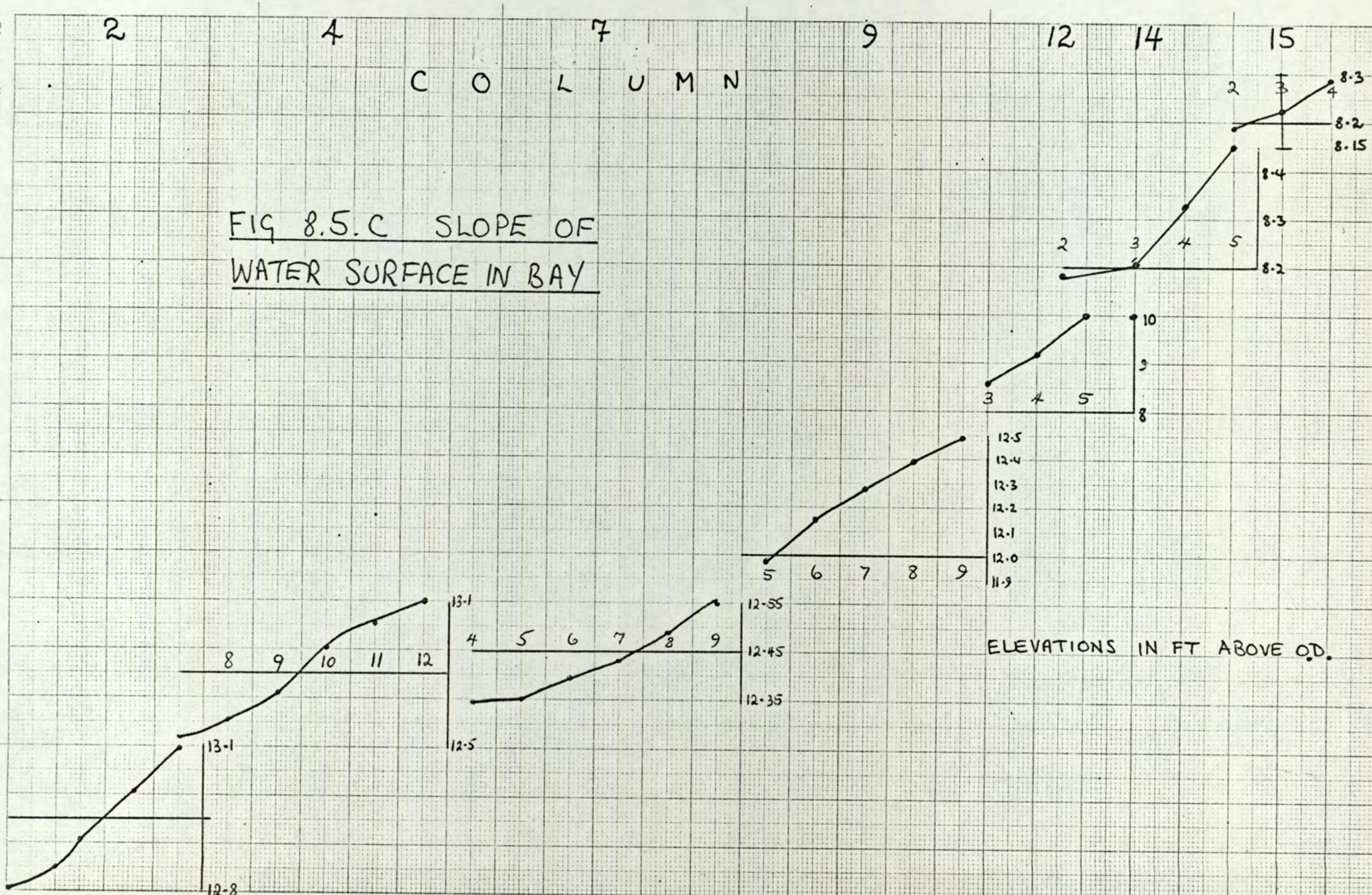


FIG 8.5.C SLOPE OF WATER SURFACE IN BAY

BAY	DATA ELEVTN	AT 25.000 HRS			5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	8.182	8.180	8.198	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	8.605	8.281	8.208	8.227	0.000
4	0.000	0.000	0.000	0.000	0.000	12.351	12.349	12.307	0.000	0.000	9.986	9.215	8.571	8.332	8.292	0.000
5	0.000	0.000	0.000	0.000	12.429	12.366	12.356	12.281	11.973	11.419	10.811	10.041	8.699	8.452	0.000	0.000
6	0.000	0.000	0.000	12.541	12.483	12.455	12.392	12.305	12.150	11.774	11.399	0.000	0.000	0.000	0.000	0.000
7	0.000	12.808	12.767	12.612	12.580	12.526	12.438	12.363	12.281	12.134	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	12.852	12.852	12.718	12.702	12.680	12.481	12.421	12.395	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	12.911	12.936	12.903	12.846	12.827	12.556	12.500	12.490	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	13.100	13.036	13.052	13.037	13.013	13.018	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	13.100	13.100	13.100	13.100	13.100	13.100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY	DATA ELEVTN	AT 25.500 HRS			5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	11.945	11.973	11.997	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	11.995	11.927	11.952	11.989	0.000
4	0.000	0.000	0.000	0.000	0.000	13.145	13.162	13.156	0.000	0.000	12.442	12.170	11.996	11.971	11.995	0.000
5	0.000	0.000	0.000	0.000	13.075	13.101	13.144	13.169	13.084	12.926	12.750	12.461	12.027	12.000	0.000	0.000
6	0.000	0.000	0.000	12.949	13.001	13.059	13.104	13.142	13.141	13.063	12.937	0.000	0.000	0.000	0.000	0.000
7	0.000	12.803	12.831	12.876	12.952	13.018	13.034	13.128	13.162	13.151	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	12.798	12.831	12.857	12.922	12.995	13.039	13.097	13.134	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	12.757	12.784	12.812	12.865	12.932	13.012	13.098	13.147	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	12.600	12.688	12.705	12.716	12.736	12.777	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	12.600	12.600	12.600	12.600	12.600	12.600	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY	DATA ELEVTN	AT 26.000 HRS			5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	14.414	14.456	14.486	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	14.220	14.312	14.395	14.446	0.000
4	0.000	0.000	0.000	0.000	0.000	12.718	12.759	12.803	0.000	0.000	13.918	14.073	14.217	14.330	14.402	0.000
5	0.000	0.000	0.000	0.000	12.514	12.618	12.723	12.875	13.077	13.376	13.688	13.911	14.140	14.263	0.000	0.000
6	0.000	0.000	0.000	12.179	12.321	12.453	12.605	12.781	12.970	13.227	13.396	0.000	0.000	0.000	0.000	0.000
7	0.000	11.673	11.768	11.976	12.137	12.299	12.497	12.667	12.834	12.988	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	11.629	11.689	11.847	11.977	12.117	12.340	12.514	12.628	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	11.533	11.561	11.638	11.773	11.897	12.193	12.407	12.527	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	11.100	11.332	11.342	11.372	11.417	11.466	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	11.100	11.100	11.100	11.100	11.100	11.100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY	DATA ELEVTN	AT 26.500 HRS			5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	14.397	14.458	14.497	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	14.024	14.200	14.334	14.404	0.000
4	0.000	0.000	0.000	0.000	0.000	11.142	11.174	11.234	0.000	0.000	13.383	13.694	13.992	14.186	14.301	0.000
5	0.000	0.000	0.000	0.000	10.982	11.060	11.142	11.337	11.647	12.238	12.853	13.274	13.953	14.060	0.000	0.000
6	0.000	0.000	0.000	10.665	10.776	10.880	11.018	11.222	11.461	11.906	12.249	0.000	0.000	0.000	0.000	0.000
7	0.000	10.117	10.205	10.429	10.570	10.706	10.918	11.087	11.260	11.466	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	10.017	10.049	10.238	10.343	10.441	10.723	10.922	11.012	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	9.302	9.786	9.814	9.973	10.017	10.512	10.815	10.935	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	9.100	9.414	9.407	9.428	9.477	9.534	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	9.100	9.100	9.100	9.100	9.100	9.100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

FIG 8.5.D - ELEVATIONS FOR ONE TIDAL CYCLE

(i)

BAY DATA ELEVTN AT 27.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	11.911	11.920	11.921	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	11.742	11.834	11.883	11.887	0.000
4	0.000	0.000	0.000	0.000	0.000	9.552	9.562	9.596	0.000	0.000	11.244	11.456	11.722	11.829	11.858	0.000
5	0.000	0.000	0.000	0.000	9.302	9.308	9.459	9.587	9.787	10.235	10.676	11.023	11.647	11.803	0.000	0.000
6	0.000	0.000	0.000	8.857	8.990	9.116	9.266	9.435	9.584	9.986	10.359	0.000	0.000	0.000	0.000	0.000
7	0.000	8.060	8.155	8.486	8.676	8.851	9.139	9.316	9.469	9.674	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	7.802	7.831	8.166	8.308	8.412	8.976	9.168	9.292	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	7.431	7.395	7.439	7.693	7.747	8.572	9.026	9.204	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	6.300	6.747	6.743	6.789	6.892	7.023	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	6.300	6.300	6.300	6.300	6.300	6.300	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 27.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	9.436	9.438	9.428	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	9.289	9.372	9.412	9.411	0.000
4	0.000	0.000	0.000	0.000	0.000	7.200	7.297	7.322	0.000	0.000	8.771	8.974	9.250	9.358	9.384	0.000
5	0.000	0.000	0.000	0.000	7.023	7.128	7.192	7.311	7.489	7.867	8.214	8.549	9.165	9.333	0.000	0.000
6	0.000	0.000	0.000	6.529	6.653	6.790	6.965	7.128	7.262	7.609	7.986	0.000	0.000	0.000	0.000	0.000
7	0.000	5.505	5.677	6.072	6.231	6.481	6.803	7.003	7.151	7.367	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	5.215	5.222	5.679	5.819	5.914	6.519	6.864	7.093	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	4.609	4.639	4.663	4.996	5.071	6.143	6.696	6.910	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	3.180	3.725	3.726	3.789	3.949	4.160	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	3.180	3.180	3.180	3.180	3.180	3.180	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 28.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	7.231	7.240	7.238	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	7.042	7.151	7.204	7.209	0.000
4	0.000	0.000	0.000	0.000	0.000	4.842	4.842	4.979	0.000	0.000	6.446	6.676	7.001	7.134	7.172	0.000
5	0.000	0.000	0.000	0.000	4.526	4.653	4.730	4.968	5.070	5.491	5.839	6.196	6.896	7.094	0.000	0.000
6	0.000	0.000	0.000	3.951	4.101	4.261	4.474	4.663	4.818	5.192	5.605	0.000	0.000	0.000	0.000	0.000
7	0.000	2.871	2.975	3.443	3.681	3.917	4.294	4.528	4.694	4.947	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	2.400	2.413	3.097	3.150	3.250	3.975	4.384	4.543	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	1.805	1.737	1.765	2.172	2.277	3.557	4.194	4.442	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.020	0.617	0.630	0.718	0.934	1.226	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	0.020	0.020	0.020	0.020	0.020	0.020	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 28.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	5.172	5.190	5.188	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	4.946	5.079	5.147	5.155	0.000
4	0.000	0.000	0.000	0.000	0.000	2.450	2.452	2.499	0.000	0.000	4.247	4.513	4.903	5.068	5.114	0.000
5	0.000	0.000	0.000	0.000	2.099	2.242	2.329	2.488	2.718	3.195	3.561	3.945	4.793	5.025	0.000	0.000
6	0.000	0.000	0.000	1.458	1.613	1.791	2.040	2.254	2.431	2.855	3.314	0.000	0.000	0.000	0.000	0.000
7	0.000	0.231	0.357	0.390	1.146	1.414	1.846	2.113	2.301	2.589	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.362	-0.344	0.411	0.543	0.644	1.509	1.958	2.148	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-1.043	-1.125	-1.097	-0.606	-0.468	1.026	1.742	2.022	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-3.150	-2.489	-2.460	-2.340	-2.059	-1.672	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-3.150	-3.150	-3.150	-3.150	-3.150	-3.150	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

8.5. D (ii)

BAY DATA ELEVTN AT 29.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	3.241	3.268	3.269	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	2.979	3.141	3.223	3.233	0.000
4	0.000	0.000	0.000	0.000	0.000	0.102	0.102	0.248	0.000	0.000	2.161	2.468	2.946	3.136	3.188	0.000
5	0.000	0.000	0.000	0.000	-0.120	-0.034	0.058	0.236	0.485	1.011	1.387	1.793	2.823	3.095	0.000	0.000
6	0.000	0.000	0.000	-0.892	-0.738	-0.545	-0.266	-0.030	0.162	0.634	1.132	0.000	0.000	0.000	0.000	0.000
7	0.000	-2.250	-2.098	-1.507	-1.230	-0.947	-0.467	-0.172	0.032	0.348	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-2.275	-2.950	-2.005	-1.804	-1.709	-0.321	-0.333	-0.122	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-3.604	-3.735	-3.753	-3.177	-3.014	-1.343	-0.568	-0.261	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-5.980	-5.325	-5.230	-5.120	-4.793	-4.310	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-5.980	-5.980	-5.980	-5.980	-5.980	-5.980	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 29.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.457	1.493	1.495	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.155	1.348	1.443	1.455	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.302	-1.302	-1.329	0.000	0.000	0.607	0.555	1.127	1.348	1.402	0.000
5	0.000	0.000	0.000	0.000	-2.285	-2.126	-2.034	-1.842	-1.582	-1.024	-0.652	-0.236	1.000	1.307	0.000	0.000
6	0.000	0.000	0.000	-3.022	-2.885	-2.686	-2.386	-2.137	-1.938	-1.430	-0.907	0.000	0.000	0.000	0.000	0.000
7	0.000	-4.430	-4.264	-3.669	-3.309	-3.006	-2.585	-2.276	-2.062	-1.728	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-5.305	-5.277	-4.154	-4.079	-3.909	-2.945	-2.435	-2.211	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-6.017	-6.116	-6.081	-5.439	-5.262	-3.485	-2.681	-2.357	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-8.320	-7.729	-7.675	-7.474	-7.103	-6.574	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-8.320	-8.320	-8.320	-8.320	-8.320	-8.320	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 30.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.169	-0.122	-0.121	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.511	-0.287	-0.179	-0.166	0.000
4	0.000	0.000	0.000	0.000	0.000	-3.770	-3.771	-3.701	0.000	0.000	-1.593	-1.206	-0.533	-0.283	-0.227	0.000
5	0.000	0.000	0.000	0.000	-4.160	-4.003	-3.918	-3.715	-3.455	-2.885	-2.531	-2.118	-0.661	-0.321	0.000	0.000
6	0.000	0.000	0.000	-4.914	-4.810	-4.611	-4.207	-4.041	-3.840	-3.310	-2.778	0.000	0.000	0.000	0.000	0.000
7	0.000	-6.206	-6.130	-5.567	-5.327	-5.022	-4.492	-4.174	-3.957	-3.614	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-7.363	-7.340	-6.947	-6.023	-5.960	-4.852	-4.331	-4.099	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-8.060	-8.173	-8.147	-7.447	-7.257	-5.402	-4.583	-4.245	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-10.400	-9.364	-9.799	-9.559	-9.160	-8.577	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-10.400	-10.400	-10.400	-10.400	-10.400	-10.400	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 30.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.636	-1.574	-1.572	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.013	-1.759	-1.636	-1.624	0.000
4	0.000	0.000	0.000	0.000	0.000	-5.454	-5.453	-5.381	0.000	0.000	-3.232	-2.807	-2.027	-1.747	-1.694	0.000
5	0.000	0.000	0.000	0.000	-5.831	-5.695	-5.610	-5.396	-5.141	-4.578	-4.250	-3.848	-2.150	-1.782	0.000	0.000
6	0.000	0.000	0.000	-6.596	-6.533	-6.336	-6.015	-5.758	-5.559	-5.016	-4.487	0.000	0.000	0.000	0.000	0.000
7	0.000	-7.379	-7.726	-7.249	-7.031	-6.734	-6.198	-5.882	-5.667	-5.322	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-9.171	-9.148	-7.688	-7.799	-7.666	-6.547	-6.032	-5.798	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-9.782	-9.386	-9.842	-9.107	-8.023	-7.938	-6.285	-5.941	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-11.820	-11.422	-11.363	-11.099	-10.714	-10.145	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-11.820	-11.820	-11.820	-11.820	-11.820	-11.820	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

8.5.D. (iii)

BAY DATA ELEVTN AT 31.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.944	-2.864	-2.861	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-3.355	-3.069	-2.933	-2.921	0.000
4	0.000	0.000	0.000	0.000	0.000	-6.947	-6.952	-6.979	0.000	0.000	-4.717	-4.255	-3.359	-3.052	-3.000	0.000
5	0.000	0.000	0.000	0.000	-7.284	-7.180	-7.104	-6.984	-6.646	-6.112	-5.817	-5.435	-3.474	-3.082	0.000	0.000
6	0.000	0.000	0.000	-8.026	-8.013	-7.829	-7.519	-7.274	-7.087	-6.549	-6.037	0.000	0.000	0.000	0.000	0.000
7	0.000	-9.133	-9.007	-8.649	-8.461	-8.190	-7.683	-7.385	-7.182	-6.848	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-10.604	-10.573	-9.021	-9.067	-9.048	-8.000	-7.521	-7.300	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-11.040	-11.106	-11.003	-10.295	-10.147	-8.501	-7.766	-7.432	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-12.400	-12.179	-12.144	-11.900	-11.591	-11.135	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-12.400	-12.400	-12.400	-12.400	-12.400	-12.400	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 31.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.104	-4.002	-3.998	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.541	-4.228	-4.078	-4.067	0.000
4	0.000	0.000	0.000	0.000	0.000	-8.233	-8.232	-8.157	0.000	0.000	-6.047	-5.552	-4.534	-4.203	-4.157	0.000
5	0.000	0.000	0.000	0.000	-8.505	-8.444	-8.391	-8.169	-7.956	-7.478	-7.223	-6.870	-4.637	-4.229	0.000	0.000
6	0.000	0.000	0.000	-9.178	-9.207	-9.050	-8.776	-8.562	-8.400	-7.896	-7.419	0.000	0.000	0.000	0.000	0.000
7	0.000	-10.061	-9.967	-9.719	-9.566	-9.344	-8.911	-8.655	-8.478	-8.173	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-11.506	-11.459	-9.925	-10.037	-10.041	-9.173	-8.771	-8.578	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-11.689	-11.699	-11.526	-10.944	-10.858	-9.597	-8.295	-8.697	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-12.230	-12.153	-12.147	-11.978	-11.784	-11.505	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-12.230	-12.230	-12.230	-12.230	-12.230	-12.230	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 32.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-5.126	-5.000	-4.995	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-5.578	-5.244	-5.082	-5.072	0.000
4	0.000	0.000	0.000	0.000	0.000	-9.292	-9.300	-9.225	0.000	0.000	-7.216	-6.696	-5.562	-5.213	-5.172	0.000
5	0.000	0.000	0.000	0.000	-9.473	-9.455	-9.415	-9.233	-9.058	-8.656	-8.447	-8.134	-5.650	-5.234	0.000	0.000
6	0.000	0.000	0.000	-10.012	-10.028	-9.920	-9.723	-9.576	-9.457	-9.026	-8.611	0.000	0.000	0.000	0.000	0.000
7	0.000	-10.636	-10.600	-10.349	-10.232	-10.096	-9.814	-9.640	-9.513	-9.262	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-11.506	-11.437	-10.465	-10.458	-10.406	-9.976	-9.720	-9.588	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-11.303	-11.230	-11.093	-10.813	-10.824	-10.248	-9.904	-9.695	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-11.070	-11.129	-11.143	-11.094	-11.042	-10.084	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-11.070	-11.070	-11.070	-11.070	-11.070	-11.070	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 32.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-6.019	-5.870	-5.865	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-6.475	-6.128	-5.958	-5.949	0.000
4	0.000	0.000	0.000	0.000	0.000	-10.039	-10.049	-10.013	0.000	0.000	-8.211	-7.680	-6.451	-6.093	-6.059	0.000
5	0.000	0.000	0.000	0.000	-10.088	-10.077	-10.081	-10.008	-9.893	-9.579	-9.431	-9.178	-6.525	-6.109	0.000	0.000
6	0.000	0.000	0.000	-10.286	-10.193	-10.175	-10.151	-10.129	-10.099	-9.836	-9.537	0.000	0.000	0.000	0.000	0.000
7	0.000	-10.500	-10.502	-10.152	-10.134	-10.152	-10.150	-10.136	-10.113	-9.978	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-9.970	-9.909	-10.000	-10.023	-10.099	-10.136	-10.148	-10.144	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-9.710	-9.686	-9.698	-9.821	-9.909	-10.133	-10.234	-10.246	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-9.400	-9.510	-9.513	-9.537	-9.578	-9.637	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-9.400	-9.400	-9.400	-9.400	-9.400	-9.400	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

8.5.D. (iv)

BAY DATA ELEVTN AT 33.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-6.793	-6.629	-6.623	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-7.234	-6.890	-6.718	-6.711	0.000
4	0.000	0.000	0.000	0.000	0.000	-10.028	-10.058	-10.165	0.000	0.000	-8.961	-8.458	-7.209	-6.854	-6.827	0.000
5	0.000	0.000	0.000	0.000	-9.748	-9.763	-9.850	-10.083	-10.135	-9.962	-9.926	-9.801	-7.270	-6.867	0.000	0.000
6	0.000	0.000	0.000	-9.305	-9.189	-9.326	-9.403	-9.657	-9.802	-9.928	-9.894	0.000	0.000	0.000	0.000	0.000
7	0.000	-9.123	-9.034	-8.920	-8.987	-9.129	-9.353	-9.546	-9.716	-9.835	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-8.443	-8.416	-8.734	-8.748	-8.857	-9.101	-9.423	-9.601	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-8.144	-8.090	-8.097	-8.314	-8.406	-9.050	-9.377	-9.590	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-7.400	-7.641	-7.651	-7.712	-7.818	-7.967	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-7.400	-7.400	-7.400	-7.400	-7.400	-7.400	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 33.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-7.447	-7.292	-7.285	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-7.818	-7.527	-7.371	-7.367	0.000
4	0.000	0.000	0.000	0.000	0.000	-8.624	-8.612	-8.650	0.000	0.000	-9.008	-8.713	-7.800	-7.496	-7.482	0.000
5	0.000	0.000	0.000	0.000	-8.436	-8.430	-8.465	-8.641	-8.705	-8.657	-8.830	-9.018	-7.858	-7.510	0.000	0.000
6	0.000	0.000	0.000	-7.812	-7.730	-7.836	-7.994	-8.146	-8.283	-8.489	-8.621	0.000	0.000	0.000	0.000	0.000
7	0.000	-7.128	-7.133	-7.299	-7.307	-7.587	-7.892	-8.049	-8.169	-8.285	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-6.418	-6.413	-7.007	-6.982	-7.071	-7.682	-7.943	-8.068	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-6.000	-5.927	-5.963	-6.295	-6.406	-7.422	-7.840	-8.041	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-4.820	-5.166	-5.135	-5.280	-5.446	-5.652	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-4.820	-4.820	-4.820	-4.820	-4.820	-4.820	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 34.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-7.918	-7.827	-7.813	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-7.787	-7.871	-7.866	-7.884	0.000
4	0.000	0.000	0.000	0.000	0.000	-6.665	-6.661	-6.711	0.000	0.000	-7.111	-7.395	-7.755	-7.872	-7.929	0.000
5	0.000	0.000	0.000	0.000	-6.329	-6.453	-6.514	-6.686	-6.761	-6.823	-6.969	-7.153	-7.785	-7.913	0.000	0.000
6	0.000	0.000	0.000	-5.520	-5.609	-5.781	-6.027	-6.219	-6.353	-6.612	-6.725	0.000	0.000	0.000	0.000	0.000
7	0.000	-4.458	-4.579	-4.937	-5.142	-5.442	-5.873	-6.009	-6.247	-6.391	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-3.798	-3.837	-4.573	-4.613	-4.760	-5.649	-5.979	-6.112	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-3.311	-3.247	-3.350	-3.763	-3.929	-5.318	-5.945	-6.077	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	-1.900	-2.335	-2.354	-2.462	-2.678	-2.911	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	-1.900	-1.900	-1.900	-1.900	-1.900	-1.900	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 35.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-6.655	-7.009	-7.047	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-5.752	-6.398	-6.803	-6.853	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.230	-1.250	-1.343	0.000	0.000	-3.787	-4.513	-5.808	-6.467	-6.575	0.000
5	0.000	0.000	0.000	0.000	-0.683	-0.986	-1.190	-1.353	-1.672	-2.266	-2.745	-3.269	-5.725	-6.414	0.000	0.000
6	0.000	0.000	0.000	0.283	-0.031	-0.307	-0.675	-0.997	-1.274	-1.848	-2.249	0.000	0.000	0.000	0.000	0.000
7	0.000	1.864	1.644	0.992	0.549	0.112	-0.470	-0.922	-1.073	-1.347	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	2.255	2.158	1.357	1.168	0.901	-0.215	-0.647	-0.838	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	2.719	2.738	2.537	2.070	1.812	0.166	-0.428	-0.671	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	4.100	3.623	3.608	3.492	3.256	3.035	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	4.100	4.100	4.100	4.100	4.100	4.100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

8.5.D (v)

BAY DATA ELEVTN AT 36.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.936	-3.431	-3.477	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.327	-2.447	-3.096	-3.168	0.000
4	0.000	0.000	0.000	0.000	0.000	5.128	5.103	5.017	0.000	0.000	1.570	0.317	-1.502	-2.542	-2.744	0.000
5	0.000	0.000	0.000	0.000	5.559	5.209	5.108	4.985	4.538	3.701	2.912	1.956	-1.278	-2.348	0.000	0.000
6	0.000	0.000	0.000	6.279	5.984	5.759	5.462	5.174	4.864	4.209	3.664	0.000	0.000	0.000	0.000	0.000
7	0.000	7.565	7.374	6.744	6.447	6.095	5.641	5.342	5.099	4.797	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	7.811	7.727	7.137	6.948	6.704	5.849	5.518	5.367	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	8.154	8.173	7.996	7.640	7.404	6.131	5.715	5.557	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	9.270	8.850	8.848	8.762	8.582	8.432	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	9.270	9.270	9.270	9.270	9.270	9.270	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 36.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.122	-0.116	-0.130	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.587	0.574	0.100	0.054	0.000
4	0.000	0.000	0.000	0.000	0.000	8.124	8.102	8.025	0.000	0.000	4.415	3.082	1.424	0.550	0.357	0.000
5	0.000	0.000	0.000	0.000	8.450	8.243	8.165	7.981	.515	6.649	5.766	4.672	1.675	0.782	0.000	0.000
6	0.000	0.000	0.000	8.931	8.744	8.575	8.344	8.104	7.808	7.167	6.600	0.000	0.000	0.000	0.000	0.000
7	0.000	9.966	9.814	9.314	9.002	8.827	8.484	8.244	8.033	7.752	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	10.126	10.065	9.622	9.474	9.289	8.643	8.396	8.282	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	10.363	10.387	10.248	9.985	9.303	8.857	8.564	8.458	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	11.180	10.369	10.874	10.810	10.681	10.585	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	11.180	11.180	11.130	11.180	11.180	11.180	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 37.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	4.007	3.933	3.940	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	4.958	4.276	4.035	4.026	0.000
4	0.000	0.000	0.000	0.000	0.000	10.572	10.556	10.490	0.000	0.000	7.238	6.066	4.856	4.308	4.187	0.000
5	0.000	0.000	0.000	0.000	10.703	10.649	10.596	10.453	10.031	9.256	8.421	7.372	5.066	4.506	0.000	0.000
6	0.000	0.000	0.000	11.135	10.976	10.868	10.710	10.531	10.282	9.728	9.214	0.000	0.000	0.000	0.000	0.000
7	0.000	11.791	11.639	11.352	11.213	11.042	10.813	10.641	10.478	10.244	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	11.900	11.865	11.570	11.476	11.360	10.925	10.761	10.681	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	12.053	12.075	11.983	11.815	11.706	11.076	10.893	10.828	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	12.600	12.386	12.398	12.357	12.277	12.230	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	12.600	12.600	12.600	12.600	12.600	12.600	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY DATA ELEVTN AT 37.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	8.182	8.180	8.198	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	8.605	8.281	8.208	8.227	0.000
4	0.000	0.000	0.000	0.000	0.000	12.351	12.349	12.307	0.000	0.000	9.986	9.215	8.571	8.332	8.292	0.000
5	0.000	0.000	0.000	0.000	12.429	12.366	12.356	12.281	11.973	11.419	10.811	10.041	8.699	8.452	0.000	0.000
6	0.000	0.000	0.000	12.541	12.483	12.455	12.392	12.305	12.150	11.774	11.299	0.000	0.000	0.000	0.000	0.000
7	0.000	12.803	12.767	12.612	12.580	12.526	12.438	12.363	12.281	12.134	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	12.852	12.852	12.718	12.702	12.680	12.481	12.421	12.395	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	12.911	12.936	12.903	12.846	12.827	12.556	12.500	12.490	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	13.100	13.036	13.052	13.038	13.013	13.018	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	13.100	13.100	13.100	13.100	13.100	13.100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

8.5.D. (vi)

BAY RIVER INTERFACE PARAMETERS SUMMARY

TIME	VELOCITY	FLOW RATE	HEIGHT	TIME	VELOCITY	FLOW RATE	HEIGHT	TIME	VELOCITY	FLOW RATE	HEIGHT
39.67	0.16617	0.45116E 05	33.729	39.77	0.17697	0.47420E 05	33.270	39.87	0.18726	0.49477E 05	32.790
39.97	0.19613	0.51061E 05	32.298	40.07	0.20295	0.52032E 05	31.802	40.17	0.20713	0.52284E 05	31.310
40.27	0.20891	0.51912E 05	30.821	40.37	0.20939	0.51214E 05	30.332	40.47	0.20955	0.50434E 05	29.842
40.57	0.20952	0.49697E 05	29.355	40.67	0.20906	0.48686E 05	28.871	40.77	0.20808	0.47657E 05	28.393
40.87	0.20669	0.46555E 05	27.920	40.97	0.20515	0.45435E 05	27.451	41.07	0.20357	0.44324E 05	26.988
41.17	0.20190	0.43218E 05	26.530	41.27	0.20012	0.42108E 05	26.078	41.37	0.19827	0.41007E 05	25.632
41.47	0.19619	0.39881E 05	25.192	41.57	0.19358	0.38675E 05	24.759	41.67	0.19078	0.37462E 05	24.335
41.77	0.18815	0.36313E 05	23.918	41.87	0.18565	0.35217E 05	23.509	41.97	0.18324	0.34166E 05	23.108
42.07	0.18091	0.33158E 05	22.715	42.17	0.17859	0.32178E 05	22.331	42.27	0.17626	0.31224E 05	21.956
42.37	0.17396	0.30299E 05	21.539	42.47	0.17175	0.29415E 05	21.230	42.57	0.16966	0.28574E 05	20.878
42.67	0.16768	0.27775E 05	20.533	42.77	0.16581	0.27013E 05	20.196	42.87	0.16402	0.26284E 05	19.867
42.97	0.16229	0.25583E 05	19.544	43.07	0.16063	0.24912E 05	19.229	43.17	0.15894	0.24255E 05	18.923
43.27	0.15720	0.23699E 05	18.624	43.37	0.15548	0.22984E 05	18.333	43.47	0.15380	0.22383E 05	18.051
43.57	0.15216	0.21806E 05	17.776	43.67	0.15054	0.21248E 05	17.510	43.77	0.14890	0.20704E 05	17.253
43.87	0.14720	0.20171E 05	17.004	43.97	0.14543	0.19644E 05	16.764	44.07	0.14361	0.19128E 05	16.533
44.17	0.14178	0.18627E 05	16.311	44.27	0.13992	0.18139E 05	16.098	44.37	0.13797	0.17657E 05	15.896
44.47	0.13591	0.17179E 05	15.794	44.57	0.13370	0.16700E 05	15.523	44.67	0.13126	0.16213E 05	15.356
44.77	0.12847	0.15765E 05	15.294	44.87	0.12518	0.15159E 05	15.070	44.97	0.12125	0.14564E 05	14.956
45.07	0.11658	0.13908E 05	14.866	45.17	0.11087	0.13161E 05	14.807	45.27	0.10361	0.12268E 05	14.789
45.37	0.09391	0.11129E 05	14.830	45.47	0.08025	0.95659E 04	14.957	45.57	0.06111	0.73703E 04	15.182
45.67	0.03730	0.45753E 04	15.468	45.77	0.01323	0.16532E 04	15.769	45.87	-0.00627	-0.79856E 03	16.069
45.97	-0.01976	-0.25644E 04	16.382	46.07	-0.02936	-0.38883E 04	16.724	46.17	-0.03768	-0.50967E 04	17.095
46.27	-0.04614	-0.63821E 04	17.490	46.37	-0.05483	-0.77686E 04	17.904	46.47	-0.06366	-0.92279E 04	18.337
46.57	-0.07263	-0.10787E 05	18.794	46.67	-0.08212	-0.12505E 05	19.277	46.77	-0.09202	-0.14378E 05	19.784
46.87	-0.10175	-0.16317E 05	20.397	46.97	-0.11077	-0.18232E 05	20.843	47.07	-0.11916	-0.20133E 05	21.396
47.17	-0.12752	-0.22119E 05	21.967	47.27	-0.13611	-0.24240E 05	22.553	47.37	-0.14463	-0.26443E 05	23.151
47.47	-0.15298	-0.28705E 05	23.760	47.57	-0.16151	-0.31101E 05	24.382	47.67	-0.17024	-0.33637E 05	25.015
47.77	-0.17858	-0.36191E 05	25.652	47.87	-0.18629	-0.38707E 05	26.292	47.97	-0.19306	-0.41102E 05	26.933
48.07	-0.19908	-0.43496E 05	27.576	48.17	-0.20412	-0.45552E 05	28.216	48.27	-0.20835	-0.47562E 05	28.854
48.37	-0.21170	-0.49403E 05	29.487	48.47	-0.21380	-0.50972E 05	30.115	48.57	-0.21458	-0.52230E 05	30.735
48.67	-0.21427	-0.53211E 05	31.348	48.77	-0.21321	-0.53985E 05	31.952	48.87	-0.21146	-0.54551E 05	32.543
48.97	-0.20848	-0.54751E 05	33.113	49.07	-0.20340	-0.54321E 05	33.654	49.17	-0.19592	-0.53148E 05	34.166
49.27	-0.18695	-0.51460E 05	34.952	49.37	-0.17788	-0.49643E 05	35.120	49.47	-0.16920	-0.47838E 05	35.564

FIG 8.5.E BAY-EST INTERFACE

(A)

BAY RIVER INTERFACE PARAMETERS SUMMARY

TIME	VELOCITY	FLOW RATE	HEIGHT	TIME	VELOCITY	FLOW RATE	HEIGHT	TIME	VELOCITY	FLOW RATE	HEIGHT
49.57	-0.16005	-0.45800E 05	35.979	49.67	-0.14957	-0.43278E 05	36.363	49.77	-0.13800	-0.40339E 05	36.721
49.87	-0.12598	-0.37175E 05	37.053	49.97	-0.11304	-0.33639E 05	37.350	50.07	-0.09753	-0.29233E 05	37.596
50.17	-0.07857	-0.23688E 05	37.789	50.27	-0.05785	-0.17521E 05	37.941	50.37	-0.03842	-0.11678E 05	38.065
50.47	-0.02130	-0.64935E 04	38.153	50.57	-0.00405	-0.12371E 04	38.184	50.67	0.01649	0.50322E 04	38.144
50.77	0.03970	0.12097E 05	38.048	50.87	0.06160	0.18712E 05	37.913	50.97	0.07967	0.24099E 05	37.739
51.07	0.09616	0.28925E 05	37.498	51.17	0.11536	0.34440E 05	37.176	51.27	0.13644	0.40358E 05	36.809
51.37	0.15164	0.44423E 05	36.461	51.47	0.15486	0.44964E 05	36.158	51.57	0.14906	0.42931E 05	35.875
51.67	0.14218	0.40622E 05	35.579	51.77	0.13953	0.39527E 05	35.262	51.87	0.14180	0.39801E 05	34.923
51.97	0.14774	0.41054E 05	34.557	52.07	0.15609	0.42896E 05	34.160	52.17	0.16616	0.45116E 05	33.729
52.27	0.17697	0.47420E 05	33.270	52.37	0.18725	0.49476E 05	32.790	52.47	0.19614	0.51062E 05	32.298
52.57	0.20295	0.52031E 05	31.802	52.67	0.20713	0.52283E 05	31.310	52.77	0.20890	0.51910E 05	30.821
52.87	0.20939	0.51214E 05	30.332	52.97	0.20955	0.50435E 05	29.842	53.07	0.20952	0.49608E 05	29.355
53.17	0.20906	0.48687E 05	28.871	53.27	0.20807	0.47657E 05	28.393	53.37	0.20669	0.46555E 05	27.920
53.47	0.20515	0.45434E 05	27.451	53.57	0.20357	0.44324E 05	26.988	53.67	0.20190	0.43217E 05	26.530
53.77	0.20012	0.42108E 05	26.078	53.87	0.19827	0.41007E 05	25.632	53.97	0.19619	0.39881E 05	25.192
54.07	0.19357	0.38675E 05	24.759	54.17	0.19077	0.37461E 05	24.335	54.27	0.18815	0.36314E 05	23.918
54.37	0.18565	0.35217E 05	23.509	54.47	0.18324	0.34167E 05	23.108	54.57	0.18091	0.33158E 05	22.715
54.67	0.17859	0.32178E 05	22.331	54.77	0.17626	0.31224E 05	21.956	54.87	0.17396	0.30300E 05	21.589
54.97	0.17175	0.29415E 05	21.230	55.07	0.16966	0.28574E 05	20.878	55.17	0.16768	0.27775E 05	20.533
55.27	0.16581	0.27013E 05	20.196	55.37	0.16402	0.26284E 05	19.867	55.47	0.16229	0.25583E 05	19.544
55.57	0.16063	0.24912E 05	19.229	55.67	0.15894	0.24255E 05	18.923	55.77	0.15720	0.23609E 05	18.624
55.87	0.15548	0.22984E 05	18.333	55.97	0.15380	0.22383E 05	18.051	56.07	0.15216	0.21806E 05	17.776
56.17	0.15054	0.21248E 05	17.510	56.27	0.14890	0.20704E 05	17.253	56.37	0.14720	0.20171E 05	17.004
56.47	0.14543	0.19644E 05	16.764	56.57	0.14361	0.19128E 05	16.533	56.67	0.14179	0.18627E 05	16.311
56.77	0.13992	0.18139E 05	16.098	56.87	0.13797	0.17657E 05	15.896	56.97	0.13591	0.17179E 05	15.704
57.07	0.13370	0.16700E 05	15.523	57.17	0.13126	0.16213E 05	15.356	57.27	0.12847	0.15705E 05	15.204
57.37	0.12518	0.15159E 05	15.070	57.47	0.12125	0.14564E 05	14.956	57.57	0.11653	0.13908E 05	14.866
57.67	0.11087	0.13161E 05	14.897	57.77	0.10361	0.12268E 05	14.789	57.87	0.09391	0.11129E 05	14.830
57.97	0.08025	0.95658E 04	14.957	58.07	0.06111	0.73703E 04	15.182	58.17	0.03730	0.45753E 04	15.468
58.27	0.01323	0.16533E 04	15.769	58.37	-0.00627	-0.79866E 03	16.069	58.47	-0.01976	-0.25644E 04	16.382
58.57	-0.02936	-0.38883E 04	16.724	58.67	-0.03768	-0.50967E 04	17.095	58.77	-0.04614	-0.63821E 04	17.490
58.87	-0.05488	-0.77686E 04	17.994	58.97	-0.06366	-0.92274E 04	18.337	59.07	-0.07263	-0.10786E 05	18.794
59.17	-0.08212	-0.12505E 05	19.277	59.27	-0.09202	-0.14377E 05	19.784	59.37	-0.10175	-0.16318E 05	20.307

FIG 8.5.E (B)

BAY DATA VEL IN X AT 50.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.123	-0.785	-0.643	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.113	-0.113	0.000	0.000	0.000	0.000	-2.490	-1.341	-1.038	-0.758	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.245	-0.642	0.674	0.000	0.000	-3.049	-2.717	-0.713	-0.721	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.165	-1.260	-0.798	-0.230	-0.562	-1.379	-2.907	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.134	-1.387	-1.310	-0.506	-0.605	-0.782	-2.386	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.898	-0.798	-1.095	-1.607	-1.734	-0.962	-0.616	-1.516	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-1.224	-1.393	-1.580	-1.993	-2.310	-0.823	-0.467	-0.913	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-1.607	-1.750	-1.756	-2.507	-2.480	0.000	0.000	0.000	0.000	0.800	0.000	0.000	0.000	0.000	0.000
A	0.000	-1.676	-1.549	-2.084	-2.105	-1.753	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 50.000 HRS,DATE 1975* 11* 22* 15* 5*
CL*RW

BAY DATA VEL IN Y AT 50.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.429	0.277	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.788	1.232	0.574	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.467	0.954	0.000	0.000	0.000	2.653	2.973	2.096	1.183	0.000	0.000
5	0.000	0.000	0.000	0.000	0.995	1.205	1.107	2.006	2.564	2.998	3.823	3.700	1.913	0.000	0.000	0.000
6	0.000	0.000	0.000	1.030	1.201	1.544	1.567	1.857	2.548	3.421	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.387	1.645	1.367	1.513	1.840	1.926	1.779	2.494	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.446	1.423	1.058	1.097	2.342	1.793	1.367	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.277	0.655	1.099	1.272	2.365	1.523	1.021	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.421	0.631	1.260	1.102	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 50.000 HRS,DATE 1975* 11* 22* 15* 5*
CL*RW

BAY DATA VEL IN X AT 50.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.739	-0.529	-0.483	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.021	-0.021	0.000	0.000	0.000	0.000	-1.760	-0.899	-0.709	-0.553	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.467	-0.402	0.456	0.000	0.000	-2.274	-1.988	-0.498	-0.550	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.612	-0.692	-0.406	0.030	-0.295	-0.969	-2.340	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.535	-0.722	-0.717	-0.247	-0.340	-0.473	-1.922	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.418	-0.278	-0.475	-0.814	-0.882	-0.565	-0.329	-1.140	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.543	-0.635	-0.592	-0.909	-1.260	-0.461	-0.243	-0.652	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.785	-0.798	-0.656	-1.307	-1.127	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.863	-0.660	-1.090	-0.926	-0.311	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 50.500 HRS,DATE 1975* 11* 22* 15* 35*
CL*RW

BAY DATA VEL IN Y AT 50.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.233	0.153	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.240	0.851	0.409	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.343	0.732	0.000	0.000	0.000	1.937	2.106	1.459	0.868	0.000	0.000
5	0.000	0.000	0.000	0.000	0.636	0.677	0.735	1.471	1.828	2.154	2.915	2.609	1.343	0.000	0.000	0.000
6	0.000	0.000	0.000	0.627	0.759	1.007	1.027	1.322	1.817	2.705	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.605	1.025	0.350	0.238	1.117	1.293	1.226	1.941	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.285	0.720	0.463	0.609	1.393	1.107	0.951	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.203	0.456	0.660	0.784	1.105	0.766	0.592	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.345	0.500	0.270	0.307	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 50.500 HRS,DATE 1975* 11* 22* 15* 35*
CL*RW

8.5.F PREDICTED VELOCITIES

BAY DATA VEL IN X AT 51.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.507	0.242	0.039	0.000
3	0.000	0.000	0.000	0.000	0.000	0.155	0.155	0.000	0.000	0.000	0.000	0.739	0.724	0.388	0.054	0.000
4	0.000	0.000	0.000	0.000	0.000	0.608	0.137	0.145	0.000	0.000	1.385	0.526	0.522	0.347	0.000	0.000
5	0.000	0.000	0.000	0.000	1.020	1.008	0.631	0.747	0.947	1.044	0.555	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.138	1.173	0.995	0.348	0.652	0.923	0.254	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	[illegible]	1.071	1.159	1.374	1.503	0.703	0.766	0.563	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	1.302	1.412	1.987	2.023	1.529	0.726	0.659	0.550	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	1.361	1.848	2.287	2.270	2.286	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	1.561	1.789	1.613	2.488	2.707	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 51.500 HRS,DATE 1975* 11* 22* 16* 35*
CL*RW

BAY DATA VEL IN Y AT 51.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.309	-0.202	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.456	-0.383	-0.224	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.224	-0.047	0.000	0.000	0.000	-1.203	-0.831	-0.551	-0.288	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.686	-0.676	-1.179	-1.453	-2.242	-2.423	-1.523	-1.812	-0.753	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.785	-0.305	-0.933	-1.238	-1.253	-1.786	-0.763	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.413	-1.105	-0.921	-0.996	-1.334	-0.974	-0.933	-0.407	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.301	-0.990	-0.842	-1.008	-1.567	-1.171	-0.545	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.215	-0.543	-1.124	-0.917	-2.505	-1.677	-0.856	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.076	-0.247	-0.177	-0.366	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 51.500 HRS,DATE 1975* 11* 22* 16* 35*
CL*RW

BAY DATA VEL IN X AT 52.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.696	0.364	0.202	0.000
3	0.000	0.000	0.000	0.000	0.000	0.148	0.148	0.000	0.000	0.000	0.000	1.434	0.848	0.532	0.235	0.000
4	0.000	0.000	0.000	0.000	0.000	0.897	0.316	-0.066	0.000	0.000	2.232	1.439	0.518	0.418	0.000	0.000
5	0.000	0.000	0.000	0.000	1.374	1.405	0.843	0.650	0.848	1.418	1.674	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.491	1.637	1.303	0.928	0.783	1.103	1.206	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.300	1.307	1.585	1.976	2.070	1.006	0.976	1.042	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	1.034	2.089	2.595	2.863	2.428	0.973	0.759	0.819	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	2.741	2.759	3.026	3.473	3.164	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	2.474	2.571	2.689	3.489	3.244	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 52.000 HRS,DATE 1975* 11* 22* 17* 5*
CL*RW

BAY DATA VEL IN Y AT 52.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.345	-0.205	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.955	-0.640	-0.327	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.392	-0.376	0.000	0.000	0.000	-1.800	-1.597	-1.030	-0.547	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.951	-0.908	-1.398	-1.765	-2.419	-2.789	-2.532	-2.450	-1.110	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.045	-1.151	-1.369	-1.628	-1.779	-2.650	-2.120	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.756	-1.616	-1.357	-1.506	-1.045	-1.665	-1.631	-1.414	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.545	-1.606	-1.134	-1.453	-2.496	-1.860	-1.022	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.254	-0.717	-1.482	-1.416	-3.306	-2.153	-1.182	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.317	-0.526	-0.886	-0.741	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 52.000 HRS,DATE 1975* 11* 22* 17* 5*
CL*RW

8.5.F (ii)

BAY DATA VEL IN X AT 52.500 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.679	0.379	0.280	0.000
3	0.000	0.000	0.000	0.000	0.000	0.106	0.106	0.000	0.000	0.000	0.000	1.480	0.755	0.545	0.323	0.000
4	0.000	0.000	0.000	0.000	0.000	1.097	0.534	-0.052	0.000	0.000	2.076	1.648	0.396	0.411	0.000	0.000
5	0.000	0.000	0.000	0.000	1.558	1.662	1.043	0.697	0.801	1.314	1.818	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.667	1.904	1.618	0.944	0.806	1.030	1.505	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.564	1.546	1.836	2.323	2.346	1.147	1.000	1.141	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	2.239	2.415	2.959	3.348	2.990	1.055	0.766	0.872	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	3.157	3.267	3.337	4.237	3.628	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	2.894	2.941	3.390	4.039	3.646	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 52.500 HRS.DATE 1975* 11* 22* 17* 35*
CL*RW

BAY DATA VEL IN Y AT 52.500 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.324	-0.174	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.016	-0.639	-0.318	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.416	-0.472	0.000	0.000	0.000	-1.639	-1.665	-1.079	-0.595	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.005	-0.901	-1.262	-1.538	-2.039	-2.347	-2.487	-2.272	-1.102	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.088	-1.254	-1.488	-1.553	-1.672	-2.458	-2.319	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.911	-1.805	-1.537	-1.729	-2.143	-1.394	-1.807	-1.747	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.591	-1.920	-1.176	-1.524	-2.985	-2.145	-1.211	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.197	-0.736	-1.516	-1.639	-3.653	-2.344	-1.310	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.446	-0.625	-1.401	-1.291	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 52.500 HRS.DATE 1975* 11* 22* 17* 35*
CL*RW

BAY DATA VEL IN X AT 53.000 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.658	0.377	0.321	0.000
3	0.000	0.000	0.000	0.000	0.000	0.210	0.210	0.000	0.000	0.000	0.000	1.486	0.721	0.550	0.377	0.000
4	0.000	0.000	0.000	0.000	0.000	1.148	0.634	-0.078	0.000	0.000	2.025	1.756	0.341	0.406	0.000	0.000
5	0.000	0.000	0.000	0.000	1.598	1.757	1.154	0.729	0.824	1.289	1.855	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.690	1.935	1.707	0.925	0.818	0.986	1.642	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.672	1.622	1.926	2.488	2.459	1.211	0.993	1.176	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	2.331	2.534	3.132	3.502	3.282	1.070	0.767	0.895	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	3.293	3.433	3.506	4.642	3.939	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	2.991	3.042	3.734	4.292	3.857	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 53.000 HRS.DATE 1975* 11* 22* 18* 5*
CL*RW

BAY DATA VEL IN Y AT 53.000 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.314	-0.160	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.021	-0.631	-0.310	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.411	-0.552	0.000	0.000	0.000	-1.561	-1.713	-1.105	-0.616	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.990	-0.383	-1.227	-1.500	-1.967	-2.191	-2.464	-2.252	-1.121	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.040	-1.281	-1.565	-1.530	-1.646	-2.387	-2.398	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.240	-1.820	-1.583	-1.833	-2.237	-2.013	-1.886	-1.904	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.582	-2.040	-1.192	-1.490	-3.236	-2.267	-1.308	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.125	-0.686	-1.457	-1.714	-3.784	-2.413	-1.364	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.499	-0.645	-1.705	-1.489	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 53.000 HRS.DATE 1975* 11* 22* 18* 5*
CL*RW

8.5.F (iii)

BAY DATA VEL IN X AT 53.500 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.666	0.390	0.351	0.000
3	0.000	0.000	0.000	0.000	0.000	0.265	0.265	0.000	0.000	0.000	0.000	1.547	0.733	0.564	0.414	0.000
4	0.000	0.000	0.000	0.000	0.000	1.164	0.607	-0.096	0.000	0.000	2.062	1.893	0.313	0.396	0.000	0.000
5	0.000	0.000	0.000	0.000	1.622	1.329	1.234	0.756	0.847	1.319	1.900	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.483	2.046	1.770	0.905	0.926	0.957	1.734	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.755	1.705	1.972	2.615	2.559	1.249	0.977	1.180	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	2.366	2.605	3.263	3.901	3.501	1.032	0.767	0.897	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	3.390	3.628	3.603	4.043	4.103	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	3.012	3.091	3.958	4.518	4.036	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 53.500 HRS,DATE 1975* 11* 22* 18* 35*
CL*RW

BAY DATA VEL IN Y AT 53.500 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.327	-0.157	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.058	-0.653	-0.313	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.398	-0.610	0.000	0.000	0.000	-1.567	-1.818	-1.164	-0.642	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.950	-0.860	-1.212	-1.488	-1.950	-2.137	-2.493	-2.338	-1.167	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.254	-1.280	-1.620	-1.522	-1.657	-2.401	-2.472	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.224	-1.776	-1.375	-1.892	-2.310	-2.103	-1.241	-2.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.553	-2.113	-0.987	-1.434	-3.408	-2.344	-1.373	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.059	-0.616	-1.415	-1.746	-3.859	-2.449	-1.386	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.527	-0.655	-1.209	-1.651	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 53.500 HRS,DATE 1975* 11* 22* 18* 35*
CL*RW

BAY DATA VEL IN X AT 54.000 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.678	0.407	0.373	0.000
3	0.000	0.000	0.000	0.000	0.000	0.196	0.196	0.000	0.000	0.000	0.000	1.612	0.752	0.582	0.441	0.000
4	0.000	0.000	0.000	0.000	0.000	1.167	0.749	-0.104	0.000	0.000	2.100	2.031	0.284	0.375	0.000	0.000
5	0.000	0.000	0.000	0.000	1.627	1.886	1.292	0.773	0.858	1.358	1.927	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.436	2.072	1.800	0.872	0.819	0.924	1.799	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.801	1.754	1.966	2.691	2.626	1.259	0.950	1.155	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	2.307	2.587	3.298	3.938	3.648	1.078	0.749	0.869	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	3.328	3.638	3.580	5.106	4.159	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	2.882	2.992	4.015	4.605	4.064	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 54.000 HRS,DATE 1975* 11* 22* 19* 5*
CL*RW

BAY DATA VEL IN Y AT 54.000 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.342	-0.160	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.094	-0.676	-0.314	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.379	-0.643	0.000	0.000	0.000	-1.571	-1.925	-1.220	-0.652	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.894	-0.832	-1.200	-1.459	-1.906	-2.073	-2.500	-2.412	-1.195	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.846	-1.269	-1.659	-1.530	-1.664	-2.424	-2.522	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.383	-1.687	-1.539	-1.924	-2.360	-2.176	-1.982	-2.066	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.514	-2.164	-0.842	-1.358	-3.527	-2.394	-1.419	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.002	-0.549	-1.383	-1.740	-3.856	-2.434	-1.387	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.549	-0.679	-2.060	-1.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 54.000 HRS,DATE 1975* 11* 22* 19* 5*
CL*RW

8.5.F (IV)

BAY DATA VEL IN X AT 54.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.686	0.428	0.389	0.000
3	0.000	0.000	0.000	0.000	0.000	0.183	0.183	0.000	0.000	0.000	0.000	1.667	0.767	0.600	0.462	0.000
4	0.000	0.000	0.000	0.000	0.000	1.141	0.761	-0.105	0.000	0.000	2.119	2.160	0.247	0.342	0.000	0.000
5	0.000	0.000	0.000	0.000	1.593	1.005	1.320	0.775	0.850	1.381	1.923	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.549	2.048	1.785	0.817	0.791	0.877	1.832	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.305	1.760	1.910	2.703	2.635	1.239	0.708	1.105	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	2.174	2.480	3.239	3.977	3.702	1.052	0.716	0.818	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	3.158	3.521	3.431	5.114	4.005	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	2.637	2.772	3.922	4.531	3.046	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 54.500 HRS,DATE 1975* 11* 22* 19* 35*
CL*RW

BAY DATA VEL IN Y AT 54.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.356	-0.165	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.123	-0.694	-0.311	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.342	-0.667	0.000	0.000	0.000	-1.559	-2.023	-1.267	-0.648	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.205	-0.734	-1.167	-1.398	-1.818	-1.975	-2.470	-2.457	-1.204	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.697	-1.228	-1.661	-1.503	-1.641	-2.415	-2.533	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.302	-1.526	-1.450	-1.900	-2.370	-2.108	-1.090	-2.092	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.458	-2.161	-0.662	-1.247	-3.560	-2.392	-1.433	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.057	-0.482	-1.337	-1.620	-3.752	-2.359	-1.355	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.562	-0.708	-2.146	-1.364	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 54.500 HRS,DATE 1975* 11* 22* 19* 35*
CL*RW

BAY DATA VEL IN X AT 55.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.680	0.446	0.396	0.000
3	0.000	0.000	0.000	0.000	0.000	0.172	0.172	0.000	0.000	0.000	0.000	1.705	0.768	0.613	0.472	0.000
4	0.000	0.000	0.000	0.000	0.000	1.099	0.769	-0.098	0.000	0.000	2.117	2.271	0.199	0.299	0.000	0.000
5	0.000	0.000	0.000	0.000	1.538	1.899	1.330	0.769	0.828	1.384	1.888	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.443	1.994	1.743	0.753	0.750	0.819	1.832	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.784	1.747	1.330	2.682	2.613	1.201	0.856	1.036	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	2.017	2.346	3.144	3.977	3.711	1.016	0.672	0.748	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	2.980	3.393	3.273	5.074	3.997	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	2.412	2.567	3.817	4.448	3.820	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 55.000 HRS,DATE 1975* 11* 22* 20* 5*
CL*RW

BAY DATA VEL IN Y AT 55.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.367	-0.171	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.139	-0.705	-0.305	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.312	-0.668	0.000	0.000	0.000	-1.529	-2.107	-1.302	-0.630	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.604	-0.726	-1.119	-1.310	-1.692	-1.845	-2.401	-2.464	-1.192	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.524	-1.172	-1.638	-1.456	-1.590	-2.370	-2.503	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.689	-1.301	-1.345	-1.861	-2.356	-2.183	-1.969	-2.083	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.383	-2.110	-0.457	-1.119	-3.539	-2.356	-1.423	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.117	-0.400	-1.281	-1.626	-3.614	-2.254	-1.303	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.567	-0.724	-2.196	-1.923	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 55.000 HRS,DATE 1975* 11* 22* 20* 5*
CL*RW

BAY DATA VEL IN X AT 55.500 HRS					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.661	0.465	0.398	0.000
3	0.000	0.000	0.000	0.000	0.000	0.162	0.162	0.000	0.000	0.000	0.000	1.727	0.757	0.622	0.475	0.000
4	0.000	0.000	0.000	0.000	0.000	1.053	0.776	-0.084	0.000	0.000	2.101	2.365	0.143	0.250	0.000	0.000
5	0.000	0.000	0.000	0.000	1.473	1.083	1.338	0.766	0.805	1.376	1.831	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.336	1.913	1.620	0.638	0.704	0.758	1.813	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.738	1.701	1.725	2.613	2.548	1.152	0.200	0.959	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	1.704	2.124	2.065	3.883	3.645	0.971	0.422	0.667	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	2.642	3.096	2.055	4.872	3.795	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	2.032	2.191	3.532	4.148	3.518	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 55.500 HRS,DATE 1975* 11* 22* 20* 35*
CL*RW

BAY DATA VEL IN Y AT 55.500 HRS					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.371	-0.176	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.142	-0.710	-0.295	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.272	-0.661	0.000	0.000	0.000	-1.485	-2.178	-1.325	-0.603	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.567	-0.664	-1.063	-1.205	-1.538	-1.692	-2.303	-2.436	-1.162	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.341	-1.113	-1.601	-1.397	-1.525	-2.303	-2.445	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.551	-1.028	-1.728	-1.800	-2.319	-2.145	-1.030	-2.052	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.317	-2.043	-0.261	-0.932	-3.469	-2.204	-1.397	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.147	-0.365	-1.238	-1.535	-3.403	-2.113	-1.237	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.560	-0.756	-2.181	-1.201	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 55.500 HRS,DATE 1975* 11* 22* 20* 35*
CL*RW

BAY DATA VEL IN X AT 56.000 HRS					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.633	0.483	0.396	0.000
3	0.000	0.000	0.000	0.000	0.000	0.154	0.154	0.000	0.000	0.000	0.000	1.739	0.739	0.629	0.473	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000	0.766	-0.072	0.000	0.000	2.078	2.449	0.081	0.199	0.000	0.000
5	0.000	0.000	0.000	0.000	1.375	1.828	1.320	0.742	0.767	1.351	1.758	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.182	1.777	1.571	0.628	0.646	0.689	1.770	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.628	1.584	1.574	2.456	2.395	1.077	0.734	0.873	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	1.481	1.769	2.674	3.618	3.432	0.000	0.564	0.575	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	2.110	2.553	2.420	4.416	3.420	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	1.527	1.656	3.020	3.570	3.005	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 56.000 HRS,DATE 1975* 11* 22* 21* 5*
CL*RW

BAY DATA VEL IN Y AT 56.000 HRS					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.366	-0.180	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.135	-0.711	-0.283	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.230	-0.645	0.000	0.000	0.000	-1.434	-2.242	-1.337	-0.569	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.423	-0.506	-1.007	-1.087	-1.362	-1.541	-2.187	-2.376	-1.118	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.164	-1.042	-1.534	-1.314	-1.439	-2.207	-2.362	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.414	-0.750	-1.109	-1.706	-2.213	-2.066	-1.961	-1.993	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.278	-1.945	-0.115	-0.842	-3.292	-2.130	-1.341	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.111	-0.428	-1.204	-1.396	-3.081	-1.232	-1.147	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.523	-0.781	-2.050	-1.744	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 56.000 HRS,DATE 1975* 11* 22* 21* 5*
CL*RW

8.5.F (vi)

BAY DATA VEL IN X AT 56.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.595	0.502	0.389	0.000
3	0.000	0.000	0.000	0.000	0.000	0.145	0.145	0.000	0.000	0.000	0.000	1.739	0.711	0.634	0.466	0.000
4	0.000	0.000	0.000	0.000	0.000	0.004	0.735	-0.065	0.000	0.000	2.041	2.518	0.014	0.147	0.000	0.000
5	0.000	0.000	0.000	0.000	1.434	1.713	1.255	0.686	0.704	1.299	1.664	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	1.006	1.571	1.402	0.500	0.573	0.609	1.697	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.441	1.385	1.363	2.179	2.123	0.970	0.657	0.778	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	1.054	1.253	2.254	3.142	3.026	0.825	0.504	0.482	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	1.385	1.756	1.722	3.694	2.854	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.288	1.046	2.347	2.765	2.309	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 56.500 HRS,DATE 1975* 11* 22* 21* 35*
CL*RW

BAY DATA VEL IN Y AT 56.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.347	-0.184	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.119	-0.708	-0.268	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.189	-0.617	0.000	0.000	0.000	-1.974	-2.297	-1.339	-0.528	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.272	-0.522	-0.931	-0.958	-1.166	-1.373	-2.050	-2.276	-1.061	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.026	-0.954	-1.423	-1.199	-1.325	-2.073	-2.246	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.300	-0.522	-0.993	-1.567	-2.043	-1.927	-1.748	-1.895	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.286	-1.792	-0.054	-0.708	-2.092	-1.994	-1.245	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.004	-0.601	-1.145	-1.202	-2.627	-1.683	-1.025	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.451	-0.771	-1.799	-1.456	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 56.500 HRS,DATE 1975* 11* 22* 21* 35*
CL*RW

BAY DATA VEL IN X AT 57.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.548	0.519	0.376	0.000
3	0.000	0.000	0.000	0.000	0.000	0.136	0.136	0.000	0.000	0.000	0.000	1.724	0.677	0.636	0.452	0.000
4	0.000	0.000	0.000	0.000	0.000	0.779	0.665	-0.067	0.000	0.000	1.984	2.560	-0.054	0.098	0.000	0.000
5	0.000	0.000	0.000	0.000	1.019	1.489	1.117	0.583	0.606	1.209	1.544	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.749	1.240	1.128	0.374	0.478	0.516	1.581	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.086	1.003	1.041	1.679	1.627	0.803	0.559	0.672	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.272	0.345	1.507	2.264	2.272	0.688	0.442	0.407	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.017	0.373	0.615	2.500	1.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.220	0.140	1.362	1.519	1.160	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 57.000 HRS,DATE 1975* 11* 22* 22* 5*
CL*RW

BAY DATA VEL IN Y AT 57.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.307	-0.187	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.091	-0.699	-0.252	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.150	-0.570	0.000	0.000	0.000	-1.306	-2.334	-1.328	-0.483	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.139	-0.439	-0.828	-0.222	-0.962	-1.199	-1.891	-2.127	-0.993	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	-0.333	-1.237	-1.031	-1.169	-1.883	-2.085	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.262	-0.490	-0.992	-1.351	-1.745	-1.693	-1.566	-1.740	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.380	-1.506	-0.125	-0.567	-2.439	-1.693	-1.082	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.222	-0.872	-0.275	-0.993	-1.880	-1.301	-0.842	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.339	-0.697	-1.401	-0.995	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 57.000 HRS,DATE 1975* 11* 22* 22* 5*
CL*RW

BAY DATA VEL IN X AT 57.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.497	0.532	0.356	0.000
3	0.000	0.000	0.000	0.000	0.000	0.121	0.121	0.000	0.000	0.000	0.000	1.690	0.637	0.636	0.431	0.000
4	0.000	0.000	0.000	0.000	0.000	0.515	0.478	-0.098	0.000	0.000	1.885	2.555	-0.114	0.053	0.000	0.000
5	0.000	0.000	0.000	0.000	0.522	0.928	0.721	0.321	0.382	1.026	1.276	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.031	0.584	0.585	0.116	0.313	0.379	1.371	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.342	-0.672	0.342	0.725	0.649	0.469	0.396	0.529	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.643	-0.660	0.034	0.764	0.953	0.356	0.354	0.396	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-1.084	-0.912	-0.577	0.837	0.385	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.579	-0.731	0.334	-0.004	-0.548	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 57.500 HRS,DATE 1975* 11* 22* 22* 35*
CL*RW

BAY DATA VEL IN Y AT 57.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.234	-0.190	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.051	-0.683	-0.234	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.109	-0.460	0.000	0.000	0.000	-1.230	-2.346	-1.305	-0.434	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.087	-0.313	-0.620	-0.666	-0.776	-1.010	-1.694	-1.919	-0.916	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.262	-0.601	-0.855	-0.711	-0.892	-1.554	-1.829	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.346	-0.761	-0.699	-0.929	-1.116	-1.213	-1.207	-1.442	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-0.244	-0.511	-0.169	-0.229	-1.359	-1.045	-0.736	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.231	-0.537	-0.335	-0.430	-0.522	-0.533	-0.438	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-0.137	-0.381	-0.374	-0.445	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 57.500 HRS,DATE 1975* 11* 22* 22* 35*
CL*RW

BAY DATA VEL IN X AT 58.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.448	0.539	0.327	0.000
3	0.000	0.000	0.000	0.000	0.000	0.080	0.080	0.000	0.000	0.000	0.000	1.615	0.596	0.631	0.402	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.455	-0.214	-0.293	0.000	0.000	1.648	2.412	-0.155	0.014	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.032	-0.854	-0.648	-0.711	-0.392	0.536	1.060	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.262	-0.760	-0.609	-0.599	-0.108	0.064	0.879	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.205	-1.225	-0.647	-0.872	-1.087	-0.304	-0.035	0.183	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-1.103	-1.265	-1.585	-1.403	-0.956	-0.436	-0.109	0.246	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-1.606	-1.705	-1.731	-1.309	-1.626	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-1.244	-1.399	-0.906	-1.941	-2.164	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 58.000 HRS,DATE 1975* 11* 22* 23* 5*
CL*RW

BAY DATA VEL IN Y AT 58.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.100	-0.193	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.995	-0.658	-0.216	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.006	-0.070	0.000	0.000	0.000	-1.145	-2.298	-1.264	-0.383	0.000	0.000
5	0.000	0.000	0.000	0.000	-0.027	0.082	0.167	-0.294	-0.663	-0.733	-1.270	-1.617	-0.831	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.343	0.025	0.090	0.125	-0.195	-0.746	-1.267	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.288	-0.458	0.007	0.107	0.322	-0.022	-0.341	-0.711	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.003	0.729	0.188	0.453	0.705	0.418	0.169	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.102	-0.018	0.611	0.314	1.719	1.085	0.746	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.160	0.122	-0.027	0.452	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 58.000 HRS,DATE 1975* 11* 22* 23* 5*
CL*RW

8.5.F(viii)

BAY DATA VEL IN X AT 58.500 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.406	0.527	0.281	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.020	-0.020	0.000	0.000	0.000	0.000	1.367	0.544	0.612	0.362	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.012	-0.706	-0.212	0.000	0.000	0.543	1.606	-0.133	-0.012	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.358	-1.759	-1.509	-1.065	-1.042	-0.773	0.421	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.162	-1.487	-1.302	-0.913	-0.514	-0.431	-0.156	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.407	-1.539	-1.290	-1.938	-2.153	-0.809	-0.404	-0.361	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-1.504	-1.856	-2.308	-2.911	-2.559	-0.920	-0.262	-0.058	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-2.287	-2.448	-2.489	-3.190	-2.023	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-1.905	-2.016	-2.223	-3.241	-2.857	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 58.500 HRS.DATE 1975* 11* 22* 23* 35*
CL*RW

BAY DATA VEL IN Y AT 58.500 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.179	-0.199	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.893	-0.605	-0.194	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.091	0.469	0.000	0.000	0.000	-0.991	-1.992	-1.167	-0.323	0.000	0.000
5	0.000	0.000	0.000	0.000	0.095	0.413	0.814	0.258	-0.185	0.200	-0.420	-0.989	-0.719	0.000	0.000	0.000
6	0.000	0.000	0.000	-0.120	0.644	0.853	0.766	0.735	0.905	0.059	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.109	0.232	0.522	0.935	1.435	0.979	0.645	0.373	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.122	1.277	0.203	0.776	2.230	1.485	0.877	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-0.062	0.160	1.031	0.930	2.492	1.400	0.866	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.357	0.504	0.955	1.210	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 58.500 HRS.DATE 1975* 11* 22* 23* 35*
CL*RW

BAY DATA VEL IN X AT 59.000 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.212	0.418	0.258	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.064	-0.064	0.000	0.000	0.000	0.000	-0.163	0.154	0.469	0.260	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.047	-0.954	-0.301	0.000	0.000	-0.782	-0.179	0.040	0.031	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.617	-2.016	-1.624	-1.114	-1.013	-1.132	-0.431	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.466	-1.933	-1.683	-0.920	-0.661	-0.600	-0.768	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.746	-1.765	-1.676	-2.403	-2.667	-1.101	-0.559	-0.595	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-1.945	-2.253	-2.734	-3.401	-3.396	-1.074	-0.358	-0.149	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-2.724	-2.908	-2.958	-4.057	-3.626	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-2.330	-2.406	-3.004	-3.786	-3.308	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 59.000 HRS.DATE 1975* 11* 23* 0* 5*
CL*RW

BAY DATA VEL IN Y AT 59.000 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.792	-0.203	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	[illegible]	0.000	0.000	0.000	0.000	-0.388	-0.423	-0.132	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	[illegible]	0.400	0.000	0.000	0.000	0.166	-0.255	-0.539	-0.130	0.000	0.000
5	0.000	0.000	0.000	0.000	0.294	0.544	0.726	0.306	0.168	0.766	0.517	0.865	-0.211	0.000	0.000	0.000
6	0.000	0.000	0.000	0.132	0.955	1.327	1.088	1.036	1.422	0.849	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.307	0.854	0.955	1.431	2.049	[illegible]	1.259	1.053	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.229	1.602	0.398	0.903	2.954	[illegible]	1.214	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.007	0.328	1.291	1.357	3.118	1.878	1.083	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.439	0.596	1.453	1.409	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 59.000 HRS.DATE 1975* 11* 23* 0* 5*
CL*RW

8.5F (ix)

CL*RW

BAY DATA VEL IN X AT 59.500 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.640	-0.427	-0.392	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.111	-0.111	0.000	0.000	0.000	0.000	-1.258	-0.632	-0.447	-0.432	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.193	-1.041	-0.224	0.000	0.000	-1.696	-1.670	-0.117	0.004	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.341	-2.209	-1.624	-1.165	-1.055	-1.366	-1.178	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.738	-2.256	-1.991	-0.991	-0.786	-0.762	-1.363	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.856	-1.854	-1.934	-2.838	-3.012	-1.343	-0.723	-0.894	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-2.212	-2.548	-3.066	-3.828	-3.905	-1.277	-0.518	-0.438	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-3.056	-3.242	-3.331	-4.589	-4.143	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-2.674	-2.732	-3.481	-4.131	-3.710	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 59.500 HRS,DATE 1975* 11* 23* 0* 35*

CL*RW

BAY DATA VEL IN Y AT 59.500 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.552	0.136	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.962	0.828	0.339	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.135	0.626	0.000	0.000	0.000	1.028	1.750	1.205	0.511	0.000	0.000
5	0.000	0.000	0.000	0.000	0.653	0.763	1.028	0.796	1.000	1.379	1.494	1.768	0.811	0.000	0.000	0.000
6	0.000	0.000	0.000	0.595	1.243	1.720	1.454	1.398	1.935	1.711	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.723	1.437	1.349	1.803	2.463	2.193	1.751	1.689	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.356	1.873	0.709	1.130	3.436	2.373	1.487	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.092	0.491	1.487	1.651	3.629	2.213	1.285	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.483	0.654	1.683	1.629	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 59.500 HRS,DATE 1975* 11* 23* 0* 35*

CL*RW

BAY DATA VEL IN X AT 60.000 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.668	-0.955	-0.465	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.153	-0.153	0.000	0.000	0.000	0.000	-2.000	-0.966	-0.942	-0.604	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.335	-1.027	-0.039	0.000	0.000	-2.266	-2.429	-0.221	0.004	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.276	-2.315	-1.623	-1.101	-1.017	-1.512	-1.773	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.281	-2.411	-2.166	-1.043	-0.876	-0.887	-1.884	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.792	-1.771	-2.021	-2.955	-3.155	-1.478	-0.842	-1.191	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-2.257	-2.597	-3.142	-3.397	-4.078	-1.377	-0.628	-0.706	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-3.100	-3.279	-3.416	-4.746	-4.374	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-2.765	-2.802	-3.613	-4.261	-3.815	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 60.000 HRS,DATE 1975* 11* 23* 1* 5*

CL*RW

BAY DATA VEL IN Y AT 60.000 HRS

BAY					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.155	0.363	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.328	1.039	0.317	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.313	0.727	0.000	0.000	0.000	1.567	2.551	1.838	0.561	0.000	0.000
5	0.000	0.000	0.000	0.000	1.092	1.000	1.276	1.337	1.651	1.867	2.247	2.660	1.247	0.000	0.000	0.000
6	0.000	0.000	0.000	1.001	1.506	2.024	1.792	1.739	2.370	2.449	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.004	2.000	1.703	2.005	2.755	2.545	2.113	2.229	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.400	2.089	1.053	1.374	3.732	2.646	1.709	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.190	0.663	1.624	1.826	3.925	2.447	1.458	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.509	0.718	1.772	1.665	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 60.000 HRS,DATE 1975* 11* 23* 1* 5*

CL*RW

8.5.F(x)

BAY DATA VEL IN X AT 60.500 HRS				5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.996	-1.072	-0.672	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.193	-0.193	0.000	0.000	0.000	0.000	-2.530	-1.321	-1.201	-0.819	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.361	-1.085	0.136	0.000	0.000	-2.718	-2.872	-0.434	-0.169	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.991	-2.264	-1.600	-0.965	-0.941	-1.583	-2.232	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.043	-2.432	-2.293	-1.059	-0.907	-0.948	-2.193	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.768	-1.717	-2.043	-2.943	-3.136	-1.514	-0.891	-1.407	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-2.293	-2.611	-3.139	-3.933	-4.059	-1.382	-0.668	-0.871	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-3.118	-3.275	-3.422	-4.722	-4.428	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-2.829	-2.846	-3.649	-4.233	-3.312	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 60.500 HRS,DATE 1975* 11* 23* 1* 35*
CL*RW

BAY DATA VEL IN Y AT 60.500 HRS				5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.481	0.397	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.827	1.405	0.481	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.395	0.834	0.000	0.000	0.000	2.113	3.177	2.419	0.909	0.000	0.000
5	0.000	0.000	0.000	0.000	1.160	1.118	1.364	1.478	2.112	2.349	2.899	3.443	1.781	0.000	0.000	0.000
6	0.000	0.000	0.000	1.199	1.602	2.115	1.925	1.927	2.615	2.943	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.113	2.208	1.344	2.182	2.803	2.641	2.238	2.519	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.551	2.128	1.241	1.591	3.757	2.699	1.783	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.247	0.747	1.651	1.887	3.984	2.498	1.506	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.517	0.744	1.775	1.646	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 60.500 HRS,DATE 1975* 11* 23* 1* 35*
CL*RW

BAY DATA VEL IN X AT 61.000 HRS				5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.299	-1.280	-0.889	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.214	-0.214	0.000	0.000	0.000	0.000	-2.883	-1.594	-1.451	-1.041	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.335	-1.034	0.309	0.000	0.000	-3.070	-3.136	-0.639	-0.439	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.929	-2.144	-1.470	-0.808	-0.875	-1.627	-2.616	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.923	-2.354	-2.147	-1.038	-0.904	-0.987	-2.404	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.695	-1.634	-1.988	-2.825	-3.908	-1.486	-0.900	-1.567	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-2.248	-2.546	-3.024	-3.657	-3.910	-1.334	-0.674	-0.977	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-3.051	-3.187	-3.332	-4.558	-4.340	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-2.819	-2.813	-3.566	-4.094	-3.694	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 61.000 HRS,DATE 1975* 11* 23* 2* 5*
CL*RW

BAY DATA VEL IN Y AT 61.000 HRS				5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.795	0.501	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	2.205	1.708	0.639	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.462	0.957	0.000	0.000	0.000	2.576	3.576	2.783	1.218	0.000	0.000
5	0.000	0.000	0.000	0.000	1.254	1.200	1.433	1.954	2.487	2.796	3.477	4.027	2.190	0.000	0.000	0.000
6	0.000	0.000	0.000	1.313	1.636	2.125	1.992	2.068	2.806	3.319	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.157	2.275	1.389	2.131	2.757	2.636	2.275	2.698	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.575	2.078	1.338	1.557	3.654	2.660	1.798	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.281	0.782	1.624	1.882	3.904	2.461	1.506	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.514	0.749	1.730	1.593	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 61.000 HRS,DATE 1975* 11* 23* 2* 5*
CL*RW

BAY DATA VEL IN X AT 61.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.492	-1.224	-0.901	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.203	-0.203	0.000	0.000	0.000	0.000	-3.054	-1.732	-1.481	-1.078	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.250	-0.249	0.471	0.000	0.000	-3.320	-3.251	-0.780	-0.667	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.775	-1.045	-1.302	-0.436	-0.809	-1.642	-2.923	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.771	-2.144	-1.075	-0.957	-0.859	-0.993	-2.545	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.406	-1.416	-1.790	-2.541	-2.717	-1.385	-0.965	-1.667	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-2.011	-2.278	-2.687	-3.257	-3.546	-1.221	-0.647	-1.035	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-2.739	-2.853	-2.980	-4.110	-3.060	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-2.577	-2.537	-3.240	-3.662	-3.268	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 61.500 HRS,DATE 1975* 11* 23* 2* 35*
CL*RW

BAY DATA VEL IN Y AT 61.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.794	0.484	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	2.357	1.735	0.715	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.504	1.008	0.000	0.000	0.000	2.903	3.732	2.846	1.401	0.000	0.000
5	0.000	0.000	0.000	0.000	1.270	1.224	1.453	2.143	2.751	3.150	3.936	4.361	2.383	0.000	0.000	0.000
6	0.000	0.000	0.000	1.341	1.397	2.047	1.976	2.138	2.917	3.593	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.148	2.227	1.845	2.082	2.594	2.529	2.232	2.787	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.577	1.960	1.358	1.520	3.334	2.500	1.742	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.307	0.793	1.531	1.777	3.594	2.281	1.425	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.409	0.741	1.628	1.431	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 61.500 HRS,DATE 1975* 11* 23* 2* 35*
CL*RW

BAY DATA VEL IN X AT 62.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.414	-1.027	-0.792	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.169	-0.169	0.000	0.000	0.000	0.000	-2.945	-1.645	-1.315	-0.950	0.000
4	0.000	0.000	0.000	0.000	0.000	-1.077	-0.800	0.604	0.000	0.000	-3.349	-3.137	-0.810	-0.768	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.519	-1.649	-1.070	-0.432	-0.703	-1.567	-3.056	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.523	-1.833	-1.702	-0.812	-0.758	-0.927	-2.553	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-1.270	-1.186	-1.523	-2.162	-2.314	-1.215	-0.774	-1.656	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-1.729	-1.951	-2.264	-2.744	-3.042	-1.059	-0.584	-1.018	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-2.363	-2.445	-2.522	-3.509	-3.394	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-2.267	-2.189	-2.805	-3.096	-2.700	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN X AT TIME 62.000 HRS,DATE 1975* 11* 23* 3* 5*
CL*RW

BAY DATA VEL IN Y AT 62.000 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.627	0.394	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	2.202	1.548	0.686	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.507	1.011	0.000	0.000	0.000	2.968	3.545	2.595	1.384	0.000	0.000
5	0.000	0.000	0.000	0.000	1.172	1.157	1.377	2.171	2.796	3.255	4.107	4.285	2.282	0.000	0.000	0.000
6	0.000	0.000	0.000	1.223	1.442	1.348	1.834	2.075	2.847	3.655	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	1.041	1.939	1.652	1.849	2.288	2.291	2.069	2.728	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.522	1.724	1.249	1.363	2.051	2.214	1.601	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.292	0.739	1.353	1.580	3.107	1.922	1.270	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.467	0.696	1.471	1.321	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 62.000 HRS,DATE 1975* 11* 23* 3* 5*
CL*RW

8.5.F (xii)

BAY DATA VEL IN X AT 62.500 HRS

				5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.123	-0.785	-0.643	0.000
3	0.000	0.000	0.000	0.000	0.000	-0.113	-0.113	0.000	0.000	0.000	0.000	-2.490	-1.341	-1.038	-0.758	0.000
4	0.000	0.000	0.000	0.000	0.000	-0.845	-0.642	0.674	0.000	0.000	-3.049	-2.717	-0.713	-0.721	0.000	0.000
5	0.000	0.000	0.000	0.000	-1.165	-1.260	-0.793	-0.230	-0.562	-1.379	-2.907	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	-1.134	-1.307	-1.310	-0.506	-0.605	-0.782	-2.386	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	-0.898	-0.798	-1.095	-1.607	-1.734	-0.962	-0.616	-1.516	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	-1.224	-1.393	-1.580	-1.983	-2.310	-0.823	-0.467	-0.913	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	-1.697	-1.750	-1.756	-2.596	-2.480	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	-1.676	-1.549	-2.084	-2.195	-1.753	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

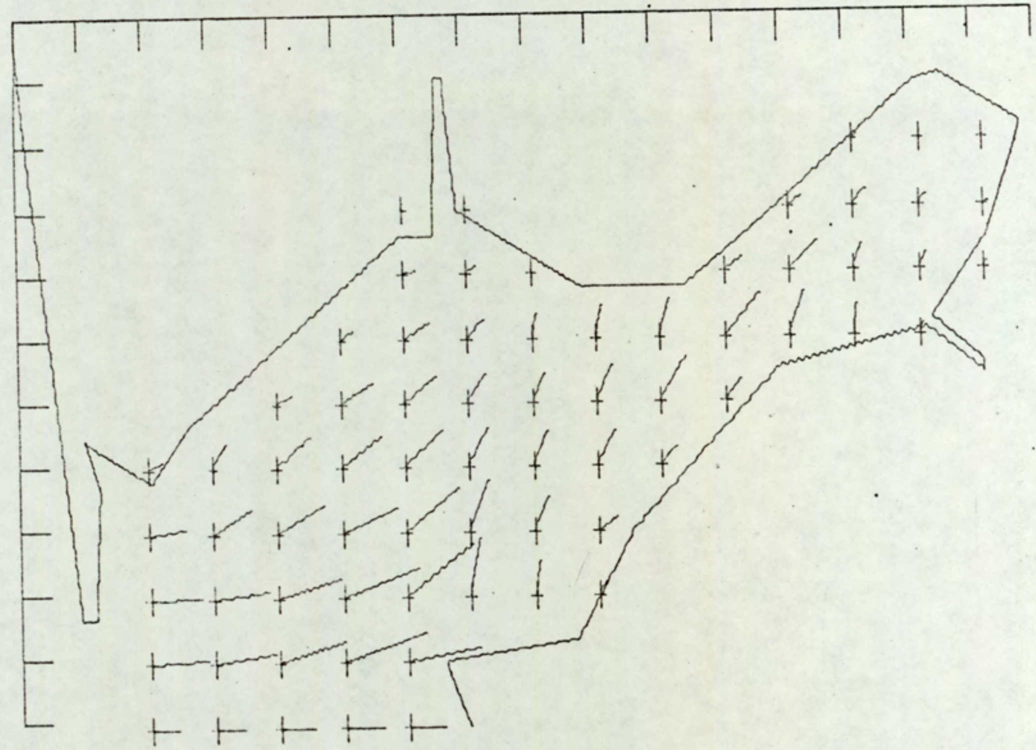
BAY OUTPUT OF VEL IN X AT TIME 62.500 HRS.DATE 1975* 11* 23* 3* 35*
CL*RW

BAY DATA VEL IN Y AT 62.500 HRS

					5	6	7	8	9	10	11	12	13	14	15	16
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.429	0.277	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.467	0.954	0.000	0.000	0.000	0.000	1.788	1.232	0.574	0.000	0.000
5	0.000	0.000	0.000	0.000	0.905	1.005	1.197	2.006	2.564	2.998	2.653	2.973	2.096	1.183	0.000	0.000
6	0.000	0.000	0.000	1.030	1.201	1.544	1.567	1.857	2.548	3.421	3.823	3.700	1.913	0.000	0.000	0.000
7	0.000	0.887	1.645	1.367	1.513	1.849	1.926	1.779	2.494	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.446	1.423	1.058	1.007	2.342	1.793	1.367	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.279	0.655	1.099	1.272	2.365	1.523	1.021	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.000	0.421	0.631	1.260	1.102	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

BAY OUTPUT OF VEL IN Y AT TIME 62.500 HRS.DATE 1975* 11* 23* 3* 35*
CL*RW

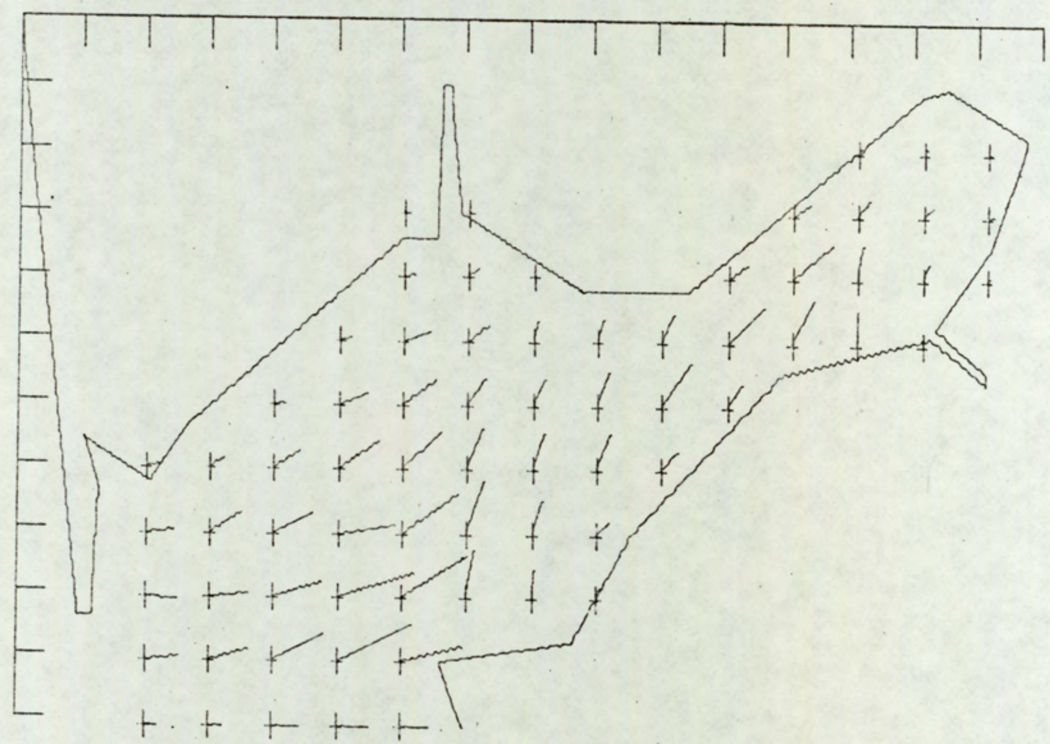
NOTE: cf 62.5 & 50



URD - MWR 2-D BAY MODEL

FIG 8.5.9 (i)

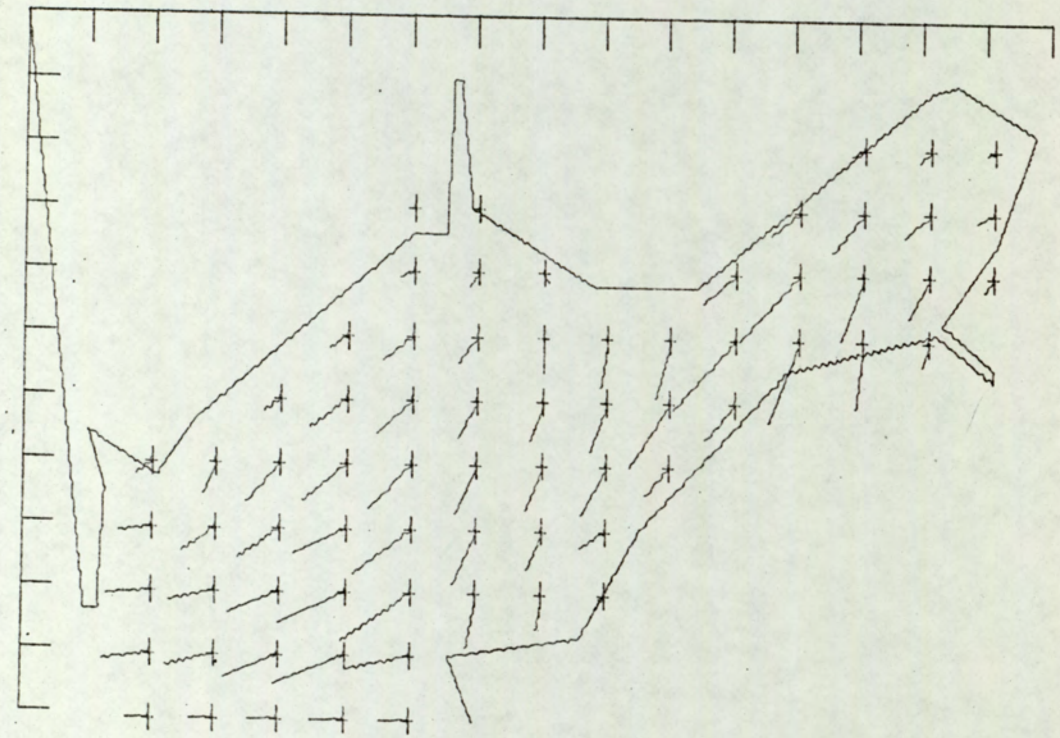
BAY VEL TIME = 6.00



URD - MWR 2-D BAY MODEL

FIG 8.5 G (ii)

BAY VEL TIME = 112

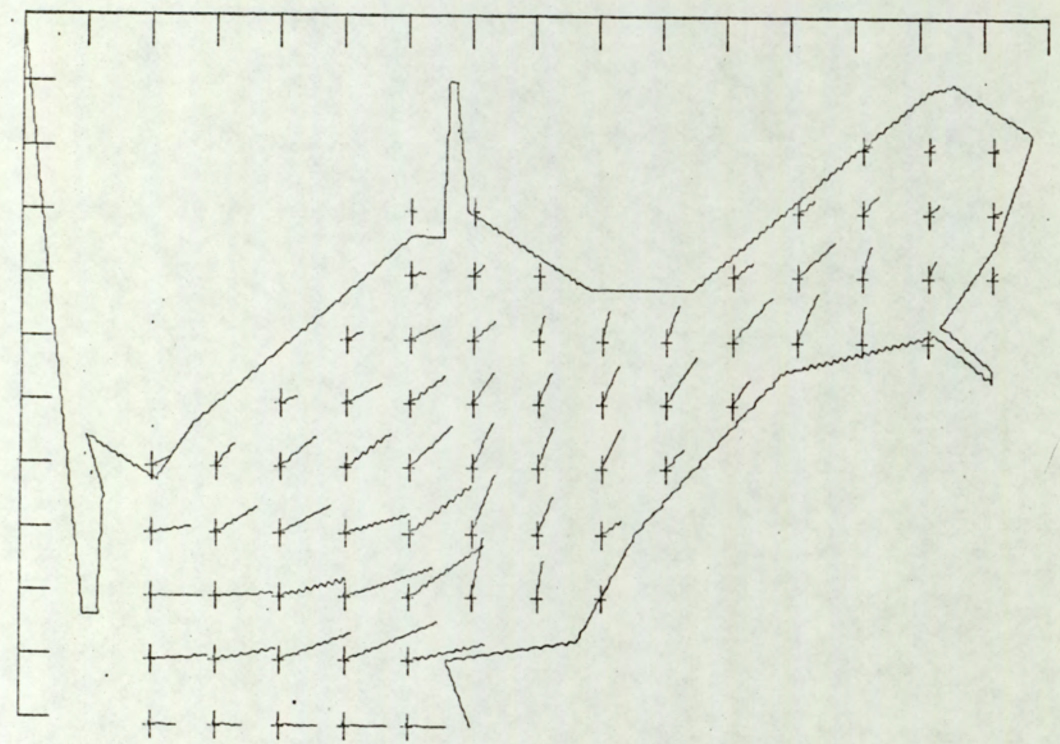


BRISTOL CHANNEL

URD - MWR 2-D BAY MODEL

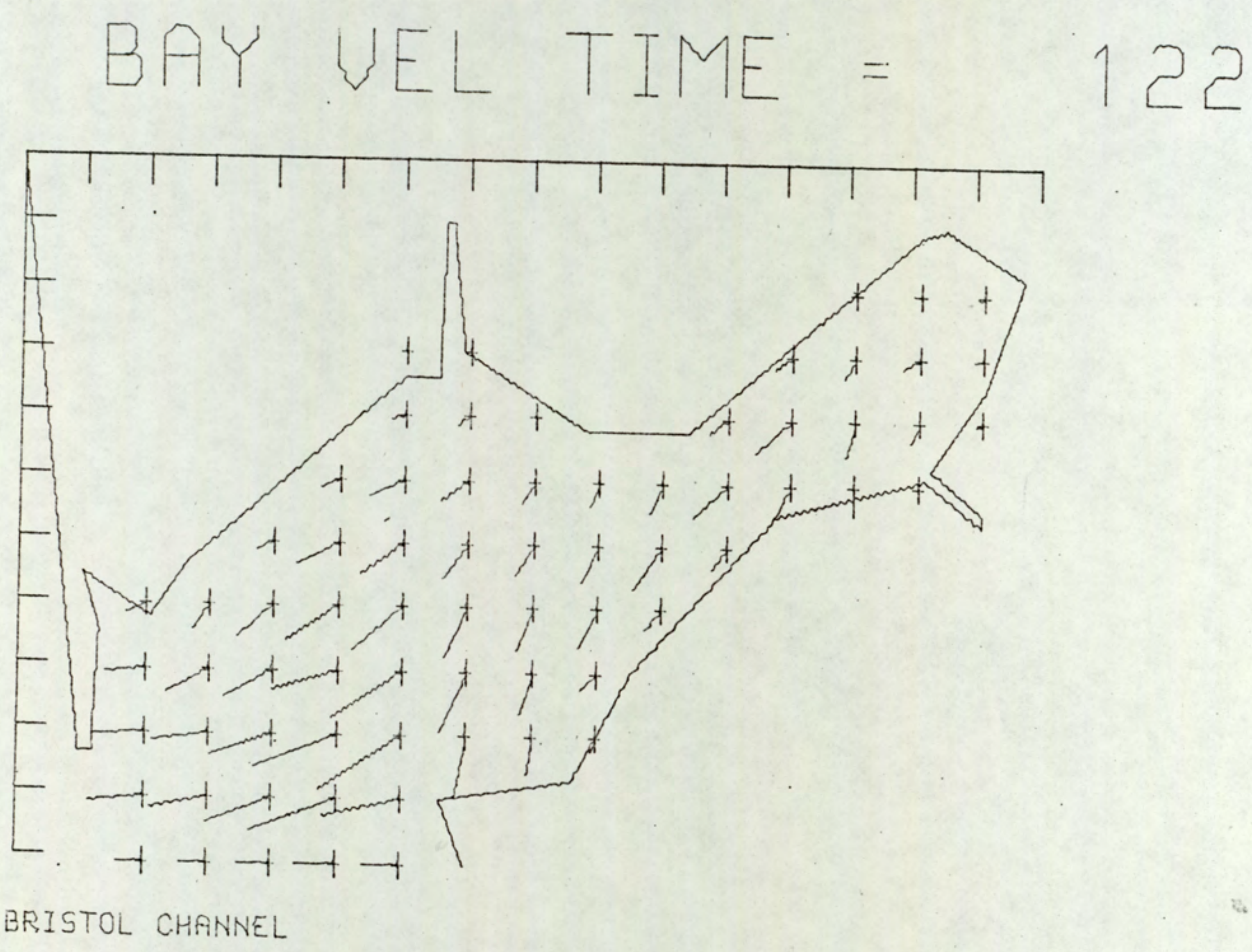
FIG 8.5.6 (iii)

BAY VEL TIME = 117



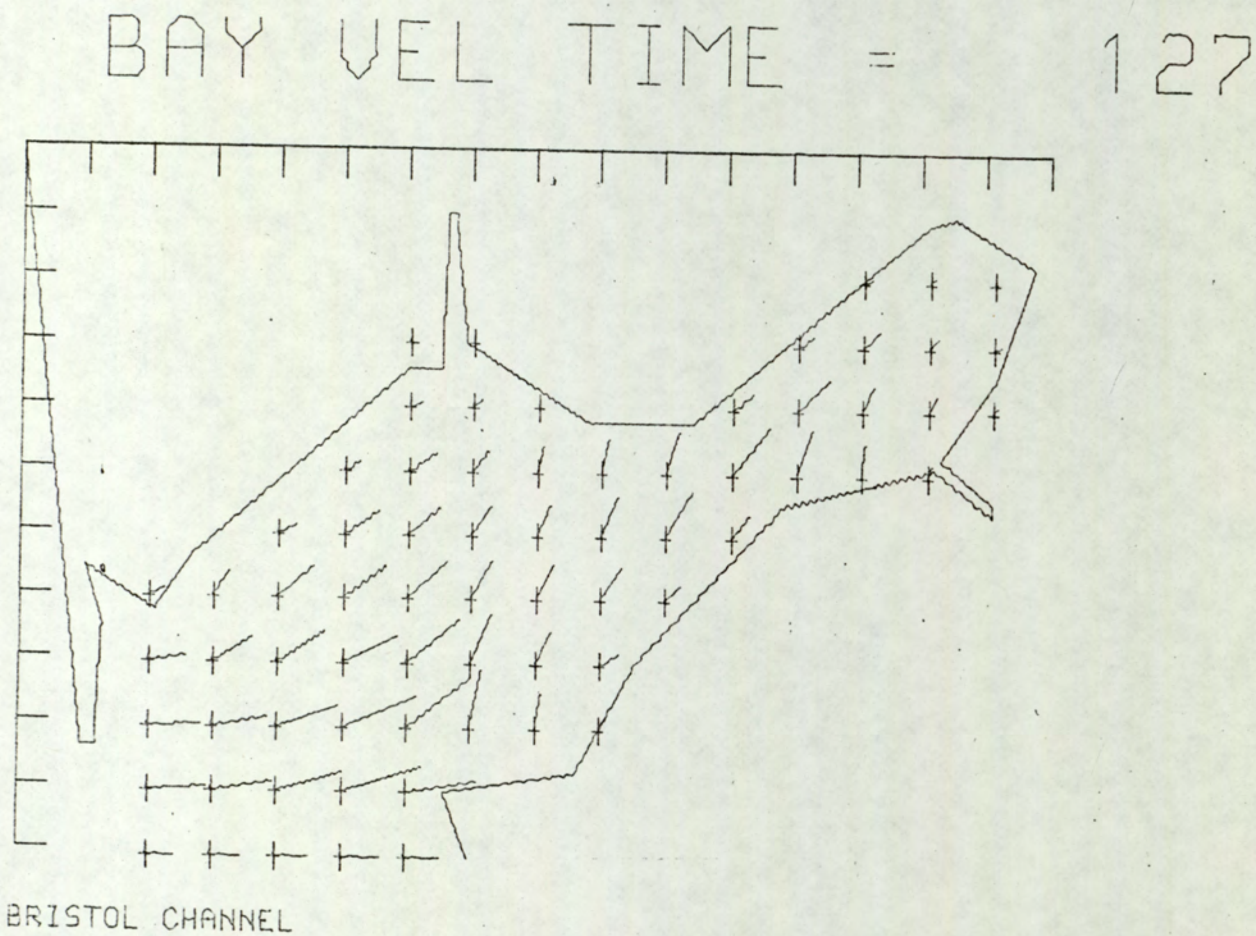
URD - MWR 2-D BAY MODEL

FIG 8.5G (iv)

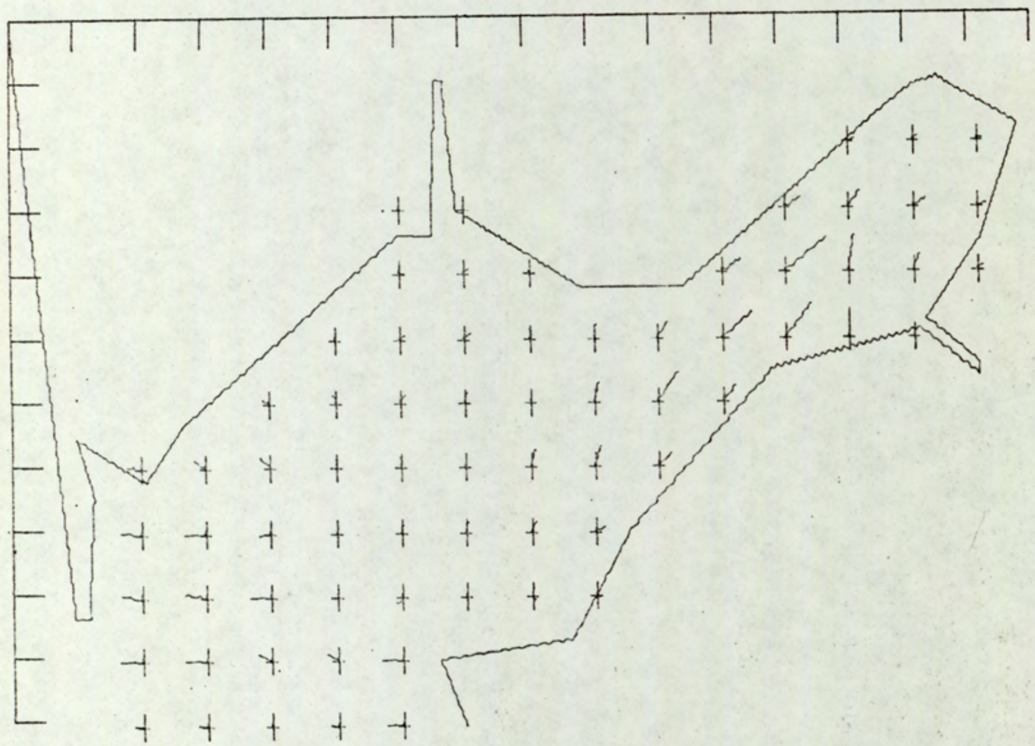


URD - MWR 2-D BAY MODEL

FIG 25 G (V)


BAY VEL TIME = 127
BRISTOL CHANNEL

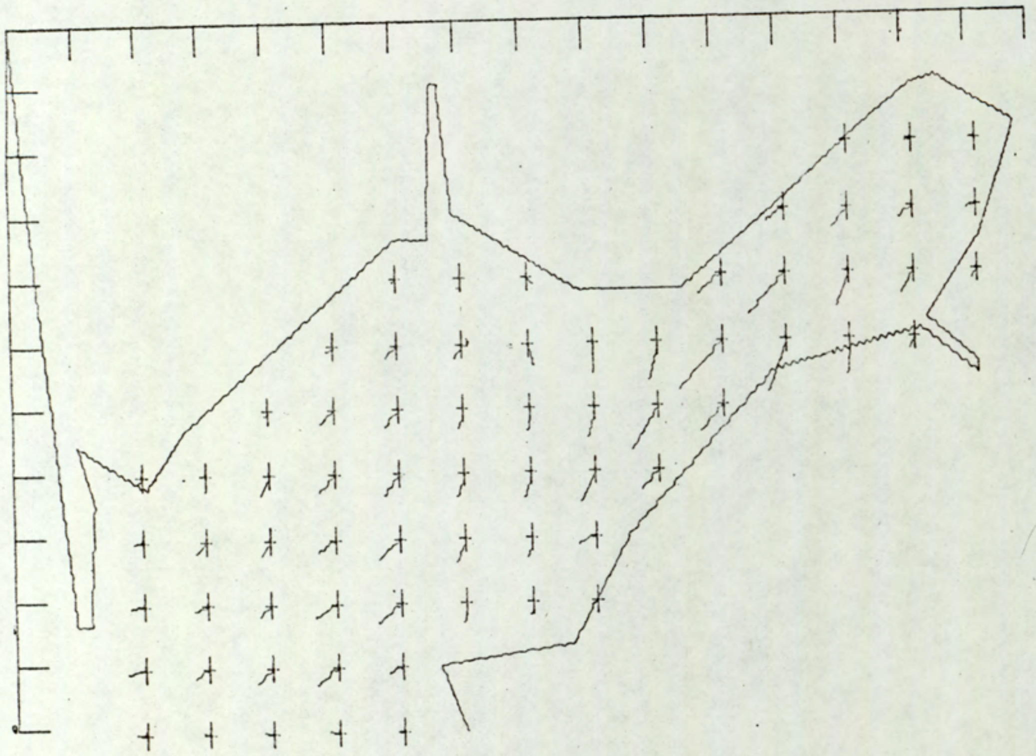
BAY VEL TIME = 133



URD - MWR 2-D BAY MODEL

FIG. 8.59 (vii)

BAY VEL TIME -



URD - MWR 2-D BAY MODEL

FIG 8.59(viii)

8.6 <u>The 1-D Phase Simulation</u>

Initial conditions in the estuary phase are set to a horizontal profile as a continuation of the bay phase (this is recommended by not essential). Only branches containing head water inflows are fitted with a modified elevation and velocity (elsewhere zero) calculated from the discharge propagated throughout the segment. Fig. 8.6.A shows the initial conditions for the phase. The number of sub-grid points are the minimum required to maintain the stability conditions discussed in chapter 4 . Because of the relatively high wave velocity in the reaches, and the requirement to have at least two points per segment, with a very short segment in the middle of Newport of about 1km , the time step cannot be safely extended beyond 1 min.

Fig. 8.6.B (9 parts) is the one dimensional simulation for a complete tidal cycle. From inspection, it is seen that the tide does not penetrate to the upper reaches, in agreement with observation. The tidal progress up the R. Ebbw is less pronounced due to its slope being 6-10*more than that of the main river.

Fig. 8.6.C shows the through-cycle elevations at points along the estuary, and fig. 8.6.D the associated velocities. Notable are the relative lengths of ebb and flood , and the rate at which the tide arrives when it turns.
Fig. 8.6.E is a conglomeration of more and less relevant elevation observations available for the estuary. Better agreement in noted for more recently available data, gathered cohesively[19]. Again, friction could be adjusted within limits without infringing the best values policy. Friction is more

important in this phase than in the bay[12][20]. Table 8.6.A shows some values for Mannings 'n', the friction term used in the model for fully turbulent flow.

Table 8.6.A Mannings Roughness Coefficient 'n'[16][20]

Clean Channel, very straight	0.025	(all ± 0.005)
Clean channel,some meandering	0.030	
Winding with a clear channel	0.033	
Winding with pools and weeds	0.040	
Winding,overgrown,weedy	>0.075	

(note : Mannings Equation and Coefficient are sometimes known as Strickler's , notably on the continent)

Fig. 8.6.F illustrates the best calibration achieved with available data for elevations. Fig. 8.6.G shows the correlation between predicted velocities for the two simulations for similar conditions (tide height predicted to ± 8%)against velocities measured through the depth for a 42.1 ft tide at St. Julians monitor site.

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS
1	4.86	4.58	143.6	3
2	12.80	11.11	195.5	3
3	19.10	15.72	294.4	3
4	23.00	19.30	360.4	2
5	25.10	20.02	395.2	2
6	27.85	20.60	524.8	2
7	33.45	19.78	1305.1	4
8	18.84	12.55	191.4	4
9	37.45	15.53	4052.3	2

SEG	X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	X	1	12.6	0.00	0.	2	14.9	0.88	542.	3	17.5	1.77	111.
2	X	1	12.6	0.00	0.	2	12.6	0.00	0.	3	12.6	0.00	0.
3	X	1	12.6	0.00	0.	2	12.6	0.00	0.	3	12.6	0.00	0.
4	X	1	12.6	0.00	0.	2	12.6	0.00	0.				
5	X	1	12.6	0.00	0.	2	12.6	0.00	0.				
6	X	1	12.6	0.00	0.	2	12.6	0.00	0.				
7	X	1	12.6	0.00	0.	2	12.6	0.00	0.	3	12.6	0.00	0.
	X	4	12.6	0.00	0.								
8	X	1	12.6	0.00	0.	2	14.3	0.26	589.	3	16.0	0.53	479.
	X	4	18.0	0.79	28.								
9	X	1	12.6	0.00	0.	2	12.6	0.00	0.				

FIG 8.6.A INITIAL CONDITIONS
1-D PHASE

RIVER AT TIME 50.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	5.29	4.91	147.6	3	X 1	13.9	1.42	2016.	2	14.5	2.21	1235.	3	17.8	1.06	111.
2	14.09	11.98	292.4	3	X 1	13.8	0.45	1310.	2	13.9	0.75	1660.	3	13.9	1.25	2016.
3	20.22	16.45	300.3	3	X 1	13.6	0.07	386.	2	13.7	0.17	808.	3	13.8	0.33	1310.
4	23.94	19.96	365.0	2	X 1	13.5	0.01	98.	2	13.6	0.06	386.				
5	25.95	20.64	399.3	2	X 1	13.4	-0.01	-76.	2	13.5	0.01	98.				
6	28.61	21.12	528.3	2	X 1	13.3	-0.05	-648.	2	13.4	-0.01	-76.				
7	34.98	20.11	1320.9	4	X 1	13.2	-0.12	-3459.	2	13.2	-0.09	-2439.	3	13.3	-0.07	-1495.
					X 4	13.3	-0.03	-649.								
8	17.11	11.32	196.4	4	X 1	13.2	0.38	1613.	2	13.0	0.94	1927.	3	10.3	5.85	2813.
					X 4	17.7	0.79	28.								
9	38.00	15.83	4115.8	2	X 1	13.1	-0.10	-6465.	2	13.2	-0.03	-1846.				

TIDE REVERSES AT 50.586 PHASE DURATION 4.752

RIVER AT TIME 51.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	5.40	5.04	147.8	3	X 1	13.9	1.95	2786.	2	14.9	2.27	1375.	3	17.9	1.03	111.
2	13.94	11.92	200.9	3	X 1	13.5	1.38	3916.	2	13.7	1.51	3318.	3	13.9	1.72	2786.
3	19.90	16.27	298.5	3	X 1	13.2	1.09	5862.	2	13.4	1.05	4813.	3	13.5	1.01	3915.
4	23.60	19.75	363.4	2	X 1	13.2	0.93	6904.	2	13.2	0.88	5862.				
5	25.62	20.43	397.9	2	X 1	13.1	0.91	7691.	2	13.2	0.90	6904.				
6	28.27	20.92	526.8	2	X 1	13.0	0.84	9849.	2	13.1	0.77	7690.				
7	33.73	19.95	1311.6	4	X 1	12.8	0.65	18738.	2	12.9	0.61	15616.	3	12.9	0.56	12658.
					X 4	13.0	0.50	9848.								
8	16.93	11.30	193.5	4	X 1	12.8	0.70	2958.	2	12.6	1.35	2723.	3	10.3	5.93	2889.
					X 4	17.7	0.79	28.								
9	37.63	15.63	4071.6	2	X 1	12.8	0.48	30732.	2	12.8	0.34	21648.				

RIVER AT TIME 51.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	5.02	4.73	145.2	3	X 1	13.1	2.48	3254.	2	14.6	2.42	1376.	3	17.8	1.05	111.
2	12.91	11.24	194.4	3	X 1	12.3	2.08	5404.	2	12.7	2.11	4255.	3	13.1	2.19	3254.
3	18.58	15.42	290.1	3	X 1	11.8	1.87	9278.	2	12.1	1.69	7188.	3	12.3	1.52	5403.
4	22.15	18.74	355.9	2	X 1	11.7	1.64	11342.	2	11.8	1.49	9278.				
5	24.12	19.34	390.4	2	X 1	11.6	1.65	12877.	2	11.7	1.60	11342.				
6	26.73	19.35	519.1	2	X 1	11.4	1.57	17104.	2	11.6	1.40	12877.				
7	32.15	19.10	1266.9	4	X 1	11.2	1.22	32651.	2	11.3	1.16	27584.	3	11.3	1.08	22344.
					X 4	11.4	0.95	17103.								
8	15.80	10.32	179.4	4	X 1	11.2	1.02	4000.	2	10.9	1.96	3551.	3	9.1	6.51	2624.
					X 4	17.6	0.79	28.								
9	36.06	14.77	3887.0	2	X 1	11.2	0.89	51401.	2	11.2	0.64	36518.				

FIG 8.6.B 1-D PHASE

(i)

RIVER AT TIME 52.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	4.41	4.21	141.3	3	X 1	11.8	2.96	3344.	2	14.0	2.53	1261.	3	17.8	1.08	111.
					X											
2	11.37	10.13	195.2	3	X 1	10.6	2.60	5895.	2	11.1	2.63	4573.	3	11.8	2.61	3344.
					X											
3	16.80	14.22	279.3	3	X 1	10.0	2.12	9423.	2	10.3	2.04	7660.	3	10.6	1.89	5895.
					X											
4	20.40	17.44	346.8	2	X 1	10.0	1.73	10917.	2	10.0	1.68	9422.				
5	22.43	18.12	391.9	2	X 1	9.9	1.67	11991.	2	10.0	1.69	10917.				
6	25.12	18.70	510.7	2	X 1	9.8	1.49	15067.	2	9.9	1.43	11991.				
7	30.60	18.26	1222.7	4	X 1	9.7	1.17	29057.	2	9.7	1.09	23865.	3	9.8	1.01	19217.
					X 4	9.8	0.92	15066.								
8	14.77	10.42	166.9	4	X 1	9.7	1.27	4625.	2	9.2	2.54	4106.	3	8.3	6.65	2282.
					X 4	17.6	0.79	28.								
9	34.49	14.04	3672.3	2	X 1	9.6	0.97	50139.	2	9.7	0.66	33536.				

RIVER AT TIME 52.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	3.84	3.71	137.6	3	X 1	10.7	3.15	3048.	2	13.5	2.53	1086.	3	17.8	1.11	111.
					X											
2	10.05	9.14	177.6	3	X 1	9.1	2.69	5397.	2	9.8	2.76	4173.	3	10.7	2.78	3048.
					X											
3	15.24	13.13	269.3	3	X 1	9.4	2.31	9230.	2	8.7	2.13	7147.	3	9.1	1.96	5396.
					X											
4	18.66	16.16	337.8	2	X 1	8.2	1.99	11347.	2	8.4	1.83	9230.				
5	20.60	16.89	370.4	2	X 1	8.0	1.99	12930.	2	8.2	1.96	11347.				
6	23.17	17.52	490.7	2	X 1	7.8	1.91	17380.	2	8.0	1.71	12929.				
7	28.49	17.09	1163.3	4	X 1	7.5	1.63	36208.	2	7.6	1.51	29441.	3	7.7	1.37	23160.
					X 4	7.8	1.20	17380.								
8	13.43	9.68	159.0	4	X 1	7.5	1.76	5814.	2	6.7	3.76	5097.	3	7.6	6.66	1967.
					X 4	17.6	0.79	28.								
9	32.28	13.50	3251.9	2	X 1	7.4	1.40	61462.	2	7.5	0.96	41765.				

RIVER AT TIME 53.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	3.35	3.27	134.9	3	X 1	9.6	3.31	2735.	2	13.1	2.48	933.	3	17.8	1.13	111.
					X											
2	8.63	8.02	170.0	3	X 1	7.3	3.15	5332.	2	8.3	3.06	3923.	3	9.6	2.93	2735.
					X											
3	13.21	11.67	255.8	3	X 1	6.1	2.85	9705.	2	6.7	2.58	7372.	3	7.3	2.29	5332.
					X											
4	16.37	14.40	326.2	2	X 1	5.9	2.43	11999.	2	6.1	2.24	9704.				
5	18.24	15.33	352.7	2	X 1	5.7	2.42	13630.	2	5.8	2.40	11999.				
6	20.76	16.05	465.9	2	X 1	5.4	2.28	18052.	2	5.7	2.10	13630.				
7	26.05	15.77	1096.4	4	X 1	5.0	1.83	35619.	2	5.1	1.73	29299.	3	5.3	1.62	23431.
					X 4	5.4	1.48	18052.								
8	11.94	8.87	150.0	4	X 1	5.0	2.17	6335.	2	3.8	5.28	5672.	3	7.0	6.52	1660.
					X 4	17.5	0.79	28.								
9	29.83	13.08	2790.3	2	X 1	4.9	1.60	58347.	2	5.0	1.16	41695.				

FIG. 8.6.B. (ii)

RIVER AT TIME SEGMENT	AV.DEPTH	53.50 AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	2.38	2.32	133.1	3	X 1	9.6	3.34	2297.	2	12.7	2.40	785.	3	17.8	1.15	111.
2	7.18	6.32	162.7	3	X 1	5.5	3.39	4693.	2	6.9	3.21	3377.	3	8.6	2.96	2297.
3	11.19	10.12	243.8	3	X 1	3.9	3.93	8638.	2	4.7	2.76	6535.	3	5.5	2.46	4693.
4	14.14	12.59	315.3	2	X 1	3.6	2.55	10740.	2	3.9	2.37	8638.				
5	15.07	13.68	336.4	2	X 1	3.4	2.53	12239.	2	3.6	2.54	10740.				
6	18.45	14.57	440.0	2	X 1	3.0	2.37	16277.	2	3.4	2.21	12239.				
7	23.09	14.53	1031.1	4	X 1	2.7	1.89	32006.	2	2.8	1.81	26279.	3	2.9	1.71	21026.
					X 4	3.0	1.59	16277.								
8	10.51	8.12	141.2	4	X 1	2.7	2.49	6418.	2	0.9	6.84	5681.	3	6.6	6.31	1415.
					X 4	17.5	0.79	28.								
9	27.44	12.99	2340.3	2	X 1	2.5	1.70	51550.	2	2.7	1.27	38193.				

RIVER AT TIME SEGMENT	AV.DEPTH	54.00 AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	2.48	2.44	131.7	3	X 1	7.7	3.25	1856.	2	12.4	2.29	654.	3	17.8	1.17	111.
2	5.94	5.74	156.9	3	X 1	3.9	3.45	3887.	2	5.6	3.19	2750.	3	7.7	2.87	1856.
3	9.37	8.65	233.9	3	X 1	1.9	3.11	7364.	2	2.9	2.82	5497.	3	3.8	2.51	3887.
4	12.07	10.35	306.7	2	X 1	1.4	2.61	9297.	2	1.9	2.44	7364.				
5	13.33	12.04	323.8	2	X 1	1.2	2.59	10682.	2	1.4	2.61	9297.				
6	16.26	13.10	417.1	2	X 1	0.8	2.43	14386.	2	1.2	2.29	10682.				
7	21.44	13.53	969.0	4	X 1	0.4	1.91	28094.	2	0.5	1.85	23016.	3	0.7	1.77	18412.
					X 4	0.8	1.69	14385.								
8	9.22	7.35	132.9	4	X 1	0.4	2.75	6212.	2	-1.7	8.11	5184.	3	6.3	6.09	1226.
					X 4	17.5	0.79	28.								
9	25.17	13.39	1914.8	2	X 1	0.3	1.74	44445.	2	0.4	1.35	34117.				

RIVER AT TIME SEGMENT	AV.DEPTH	54.50 AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	2.16	2.13	130.5	3	X 1	7.0	3.10	1486.	2	12.1	2.18	549.	3	17.7	1.18	111.
2	4.92	4.81	152.5	3	X 1	2.5	3.42	3112.	2	4.6	3.08	2187.	3	7.0	2.73	1486.
3	7.74	7.30	224.9	3	X 1	0.0	3.15	6116.	2	1.2	2.83	4490.	3	2.5	2.49	3112.
4	10.15	9.29	297.5	2	X 1	-0.5	2.65	7851.	2	0.0	2.47	6116.				
5	11.83	10.51	311.4	2	X 1	-0.8	2.62	9119.	2	-0.5	2.66	7851.				
6	14.20	11.69	397.3	2	X 1	-1.3	2.46	12464.	2	-0.8	2.34	9119.				
7	19.35	12.82	992.1	4	X 1	-1.7	1.86	23734.	2	-1.6	1.83	19463.	3	-1.4	1.79	15663.
					X 4	-1.3	1.74	12464.								
8	8.21	6.60	124.7	4	X 1	-1.7	2.99	5956.	2	-3.4	8.42	4342.	3	6.0	5.90	1092.
					X 4	17.5	0.79	28.								
9	23.09	12.90	1689.2	2	X 1	-1.8	1.73	37781.	2	-1.7	1.37	29536.				

FIG 8.6.B (iii)

RIVER AT TIME 55.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1.90	1.88	129.6	3	1	6.5	2.92	1192.	2	11.9	2.08	467.	3	17.7	1.19	111.
2	4.07	4.02	149.6	3	1	1.3	3.29	2409.	2	3.8	2.93	1722.	3	6.5	2.57	1192.
3	6.32	6.08	217.1	3	1	-1.7	3.12	4901.	2	-0.2	2.76	3547.	3	1.3	2.39	2409.
4	8.42	7.90	235.5	2	1	-2.3	2.64	6434.	2	-1.7	2.45	4901.				
5	10.02	9.09	298.9	2	1	-2.7	2.60	7585.	2	-2.3	2.65	6434.				
6	12.33	10.39	379.0	2	1	-3.2	2.44	10541.	2	-2.7	2.34	7585.				
7	17.46	12.24	839.3	4	1	-3.6	1.77	19626.	2	-3.5	1.76	16086.	3	-3.3	1.75	13075.
					4	-3.2	1.73	10541.								
8	7.47	6.09	117.0	4	1	-3.6	3.51	6197.	2	-4.3	8.42	3762.	3	5.9	5.77	1017.
					4	17.5	0.79	28.								
9	21.20	12.47	1507.6	2	1	-3.7	1.72	32281.	2	-3.6	1.38	25698.				

LL*RW
RIVER AT TIME 56.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1.54	1.53	128.4	3	1	5.7	2.57	793.	2	11.6	1.90	354.	3	17.7	1.21	111.
2	2.85	2.84	146.0	3	1	-0.5	2.84	1340.	2	2.7	2.55	1063.	3	5.7	2.26	793.
3	4.11	4.09	204.2	3	1	-4.4	2.85	2844.	2	-2.4	2.46	2053.	3	-0.5	2.06	1340.
4	5.59	5.46	266.5	2	1	-5.3	2.49	4022.	2	-4.4	2.23	2844.				
5	6.90	6.62	278.0	2	1	-5.7	2.47	4978.	2	-5.3	2.51	4022.				
6	9.17	8.15	342.3	2	1	-6.4	2.33	7333.	2	-5.7	2.24	4978.				
7	14.24	11.12	706.5	4	1	-6.8	1.56	13399.	2	-6.7	1.59	11156.	3	-6.6	1.62	9196.
					4	-6.4	1.64	7333.								
8	6.37	5.41	104.1	4	1	-6.8	4.74	6686.	2	-5.4	8.21	3053.	3	5.8	5.63	934.
					4	17.5	0.79	28.								
9	18.01	11.87	1218.5	2	1	-6.9	1.67	24182.	2	-6.8	1.40	20005.				

RIVER AT TIME 55.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1.70	1.69	128.9	3	1	6.1	2.74	966.	2	11.8	1.98	404.	3	17.7	1.20	111.
2	3.39	3.37	147.2	3	1	0.3	3.09	1814.	2	3.2	2.74	1352.	3	6.1	2.41	966.
3	5.11	5.01	210.2	3	1	-3.1	3.02	3791.	2	-1.4	2.63	2724.	3	0.3	2.25	1814.
4	6.90	6.63	274.9	2	1	-3.9	2.58	5134.	2	-3.1	2.36	3791.				
5	8.41	7.79	287.8	2	1	-4.3	2.55	6183.	2	-3.9	2.59	5134.				
6	10.65	9.24	359.6	2	1	-4.9	2.39	8814.	2	-4.3	2.30	6183.				
7	15.76	11.70	779.6	4	1	-5.3	1.67	16222.	2	-5.2	1.68	13376.	3	-5.0	1.70	11001.
					4	-4.9	1.69	8814.								
8	6.87	5.72	110.2	4	1	-5.3	4.13	6488.	2	-5.0	8.33	3357.	3	5.8	5.69	969.
					4	17.5	0.79	28.								
9	19.51	12.15	1348.6	2	1	-5.4	1.70	27881.	2	-5.3	1.40	22609.				

FIG 8.6.B (iv)

RIVER AT TIME 57.50 SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1,21	1.21	127.4	3	X	1	5.0	2.15	469,	2	11,3	1,69	255,	3	17,7	1,22	111,
2	1,75	1.75	144.6	3	X	1	-2.0	2.02	494,	2	1,6	2,02	536,	3	5,0	1,89	469,
3	2,07	2.08	199.8	3	X	1	-6.9	2.06	979,	2	-4,3	1,80	783,	3	-2,0	1,46	494,
4	2,82	2.80	257.5	2	X	1	-8.2	1.99	1660,	2	-6,9	1,63	979,				
5	3,92	3.85	259.6	2	X	1	-8.9	2.08	2415,	2	-8,2	2,01	1660,				
6	5,91	5.55	303.8	2	X	1	-9.8	2.00	4109,	2	-8,9	1,91	2415,				
7	10,97	9.50	551.3	4	X	1	-10.0	1.04	6864,	2	-9,9	1,14	6015,	3	-9,9	1,26	5118,
					X	4	-9.8	1.39	4109,								
8	5,36	4.82	91.2	4	X	1	-10.0	6.12	6721,	2	-6,1	7,88	2507,	3	5,7	5,51	873,
					X	4	17.5	0.79	28,								
9	14,36	11.64	948.9	2	X	1	-10.0	1.35	14949,	2	-10,0	1,25	13562,				

RIVER AT TIME 57.00 SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1,30	1.30	127.7	3	X	1	5.2	2.28	553,	2	11,4	1,75	281,	3	17,7	1,22	111,
2	2,05	2.05	145.0	3	X	1	-1.6	2.29	700,	2	1,9	2,18	671,	3	5,2	2,00	553,
3	2,62	2.63	200.2	3	X	1	-6.2	2.36	1448,	2	-3,8	2,02	1096,	3	-1,6	1,66	700,
4	3,56	3.53	259.8	2	X	1	-7.4	2.19	2282,	2	-6,2	1,85	1448,				
5	4,75	4.64	262.4	2	X	1	-8.1	2.24	3107,	2	-7,4	2,21	2282,				
6	6,78	6.28	316.6	2	X	1	-8.9	2.15	5027,	2	-8,1	2,05	3107,				
7	11,82	9.96	590.3	4	X	1	-9.2	1.28	9015,	2	-9,1	1,35	7669,	3	-9,0	1,42	6347,
					X	4	-8.9	1.50	5027,								
8	5,61	4.96	94.5	4	X	1	-9.2	5,78	6779,	2	-6,0	7.97	2639,	3	5,7	5,54	888,
					X	4	17.5	0,79	28,								
9	15,64	11.69	1013.6	2	X	1	-9,3	1,53	18137,	2	-9,2	1,35	15751,				

RIVER AT TIME 56.50 SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1,41	1.40	128.0	3	X	1	5.4	2.42	659,	2	11.5	1.82	314,	3	17.7	1,21	111,
2	2,41	2.40	145.4	3	X	1	-1.1	2.56	975,	2	2.3	2.36	842,	3	5,4	2,13	659,
3	3,29	3.30	200.7	3	X	1	-5.4	2.63	2071,	2	-3.2	2.25	1511,	3	-1,1	1,86	975,
4	4,43	4.41	262.9	2	X	1	-6.4	2.36	3075,	2	-5.4	2.06	2071,				
5	5,78	5.57	269.5	2	X	1	-7.0	2.37	3957,	2	-6.4	2.39	3075,				
6	7,37	7.16	327.2	2	X	1	-7.7	2.25	6090,	2	-7.0	2.16	3957,				
7	12,92	10.56	643.1	4	X	1	-8.1	1.43	11056,	2	-8.0	1.49	9315,	3	-7.9	1.53	7665,
					X	4	-7.7	1.58	6090,								
8	5,95	5.16	98.9	4	X	1	-8.1	5.30	6775,	2	-5.7	8.09	2819,	3	5.7	5.58	908,
					X	4	17.5	0.79	28,								
9	16,71	11.73	1106.3	2	X	1	-8.2	1.62	21002,	2	-8.1	1.39	17770,				

FIG 8.6.B (v)

RIVER AT TIME 58.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1.13	1.13	127.1	3	1	4.9	2.03	400.	2	11.3	1.64	232.	3	17.7	1.22	111.
2	1.50	1.50	144.2	3	1	-2.4	1.75	339.	2	1.4	1.86	428.	3	4.9	1.78	400.
3	1.63	1.64	199.5	3	1	-7.5	1.76	642.	2	-4.8	1.58	549.	3	-2.4	1.27	339.
4	2.23	2.23	256.1	2	1	-9.8	1.76	1193.	2	-7.5	1.39	642.				
5	3.31	3.25	257.6	2	1	-9.5	1.86	1852.	2	-8.8	1.79	1193.				
6	5.36	5.07	304.4	2	1	-10.2	1.64	3133.	2	-9.5	1.70	1852.				
7	10.66	9.25	545.7	4	1	-10.1	0.41	2638.	2	-10.2	0.62	3180.	3	-10.2	0.87	3347.
					4	-10.2	1.14	3133.								
8	5.30	4.77	99.5	4	1	-10.1	6.10	6585.	2	-6.2	7.82	2449.	3	5.7	5.49	864.
					4	17.5	0.79	28.								
9	14.77	11.58	947.6	2	1	-10.1	0.72	7951.	2	-10.1	0.86	9238.				

TIDE REVERSES AT 58.334 PHASE DURATION 7.748

RIVER AT TIME 58.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1.07	1.06	126.9	3	1	4.7	1.92	342.	2	11.2	1.59	213.	3	17.7	1.23	111.
2	1.28	1.28	144.0	3	1	-2.7	1.50	225.	2	1.2	1.71	341.	3	4.7	1.68	342.
3	1.28	1.29	199.3	3	1	-7.9	1.49	426.	2	-5.1	1.37	380.	3	-2.7	1.08	225.
4	1.92	1.90	255.4	2	1	-9.1	1.44	873.	2	-7.9	1.18	426.				
5	3.24	3.15	257.8	2	1	-9.4	1.12	1133.	2	-9.1	1.46	873.				
6	5.95	5.50	313.1	2	1	-9.2	-0.06	-127.	2	-9.4	1.03	1133.				
7	11.93	9.90	697.4	4	1	-8.7	-1.02	-7423.	2	-8.8	-0.87	-5007.	3	-9.0	-0.59	-2616.
					4	-9.2	-0.04	-127.								
8	5.71	4.99	95.9	4	1	-8.7	5.30	6394.	2	-6.0	7.90	2589.	3	5.7	5.51	874.
					4	17.5	0.79	28.								
9	16.16	11.70	1065.1	2	1	-8.7	-0.44	-5466.	2	-8.7	-0.09	-1035.				

RIVER AT TIME 59.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1.00	1.00	126.7	3	1	4.6	1.31	293.	2	11.2	1.54	196.	3	17.7	1.23	111.
2	1.09	1.10	143.7	3	1	-2.9	1.26	146.	2	1.0	1.57	270.	3	4.6	1.59	293.
3	1.20	1.19	199.4	3	1	-7.7	1.27	403.	2	-5.3	1.19	278.	3	-2.9	0.91	146.
4	2.61	2.52	258.1	2	1	-7.9	0.20	178.	2	-7.7	1.00	403.				
5	4.75	4.52	264.6	2	1	-7.6	-0.75	-1107.	2	-7.9	0.20	178.				
6	7.37	7.03	332.6	2	1	-7.2	-1.40	-4009.	2	-7.6	-0.68	-1107.				
7	13.38	10.88	700.2	4	1	-6.8	-1.48	-12638.	2	-6.9	-1.37	-9323.	3	-7.0	-1.21	-6522.
					4	-7.2	-0.99	-4009.								
8	6.31	5.33	103.6	4	1	-6.8	4.36	6086.	2	-5.6	8.05	2870.	3	5.7	5.56	902.
					4	17.5	0.79	28.								
9	18.09	11.39	1233.0	2	1	-6.7	-0.91	-13376.	2	-6.8	-0.46	-6593.				

FIG 8.6.B (vi)

RIVER AT TIME 59.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	0.95	0.95	126.5	3	X 1	4.5	1.71	252.	2	11.1	1.49	181.	3	17.7	1.23	111.
2	0.97	0.97	143.6	3	X 1	-3.0	1.11	111.	2	0.9	1.44	217.	3	4.5	1.50	252.
3	1.84	1.80	209.4	3	X 1	-5.0	0.49	324.	2	-5.1	1.08	300.	3	-3.0	0.80	111.
4	4.74	4.53	265.6	2	X 1	-5.4	-1.20	-1845.	2	-5.9	0.39	324.				
5	7.10	6.66	290.7	2	X 1	-5.2	-1.56	-3313.	2	-5.4	-1.21	-1845.				
6	10.26	8.83	359.4	2	X 1	-4.8	-1.92	-7078.	2	-5.2	-1.41	-3313.				
7	16.30	11.31	810.2	4	X 1	-4.3	-1.88	-19516.	2	-4.5	-1.75	-14575.	3	-4.6	-1.59	-10565.
					X 4	-4.8	-1.36	-7078.								
8	7.11	5.80	113.5	4	X 1	-4.3	3.22	5363.	2	-4.9	8.19	3290.	3	5.8	5.64	944.
					X 4	17.5	0.79	28.								
9	20.57	12.35	1456.1	2	X 1	-4.2	-1.34	-24081.	2	-4.3	-0.81	-14258.				

RIVER AT TIME 60.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	0.91	0.90	126.4	3	X 1	4.4	1.63	223.	2	11.1	1.45	168.	3	17.7	1.23	111.
2	1.02	1.01	143.9	3	X 1	-2.7	1.23	167.	2	0.8	1.36	195.	3	4.4	1.43	223.
3	3.34	3.19	210.0	3	X 1	-3.0	-1.45	-1803.	2	-3.7	0.56	302.	3	-2.7	0.89	167.
4	7.58	7.06	282.4	2	X 1	-2.6	-1.35	-4269.	2	-3.0	-1.13	-1803.				
5	10.00	8.94	299.7	2	X 1	-2.4	-2.06	-6059.	2	-2.6	-1.86	-4269.				
6	13.10	10.82	389.7	2	X 1	-1.9	-2.26	-10701.	2	-2.4	-1.85	-6059.				
7	19.17	12.78	905.8	4	X 1	-1.4	-2.12	-27255.	2	-1.6	-1.98	-20775.	3	-1.8	-1.81	-15208.
					X 4	-1.9	-1.60	-10701.								
8	8.13	6.44	125.1	4	X 1	-1.4	2.09	4192.	2	-3.8	8.19	3913.	3	5.9	5.75	1011.
					X 4	17.5	0.79	28.								
9	23.47	13.01	1735.8	2	X 1	-1.3	-1.61	-36517.	2	-1.4	-1.06	-23246.				

RIVER AT TIME 60.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	0.90	0.89	126.4	3	X 1	4.4	1.61	220.	2	11.0	1.42	159.	3	17.7	1.23	111.
2	1.59	1.55	145.1	3	X 1	-1.3	1.37	462.	2	1.1	1.43	254.	3	4.4	1.41	220.
3	5.37	5.46	224.5	3	X 1	0.1	-2.40	-4613.	2	-0.7	-1.36	-1550.	3	-1.3	0.99	462.
4	10.72	9.62	302.2	2	X 1	0.5	-2.38	-7658.	2	0.1	-1.88	-4613.				
5	13.15	11.39	329.3	2	X 1	0.8	-2.50	-9801.	2	0.5	-2.38	-7658.				
6	16.26	12.97	419.5	2	X 1	1.2	-2.56	-15416.	2	0.8	-2.21	-9801.				
7	22.34	13.88	1001.3	4	X 1	1.7	-2.29	-36305.	2	1.6	-2.16	-28610.	3	1.4	-1.99	-21589.
					X 4	1.2	-1.77	-15416.								
8	9.53	7.43	137.3	4	X 1	1.7	1.27	3068.	2	-1.7	7.60	4775.	3	6.1	5.93	1135.
					X 4	17.5	0.79	28.								
9	26.64	13.07	2208.2	2	X 1	1.8	-1.77	-51033.	2	1.7	-1.20	-33512.				

FIG 8.6.B (vii)

RIVER AT TIME 61.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1.01	1.00	126.8	3	X	1	4.7	1.77	306.	2	11.1	1.44	165.	3	17.7	1.23	111.
					X												
2	3.27	3.12	151.5	3	X	1	2.2	-0.31	-265.	2	2.3	1.53	529.	3	4.7	1.55	306.
					X												
3	9.48	8.54	241.8	3	X	1	3.7	-2.78	-7659.	2	3.0	-2.04	-3960.	3	2.2	-0.23	-265.
					X												
4	14.25	12.53	316.4	2	X	1	4.0	-2.58	-11113.	2	3.7	-2.18	-7659.				
5	16.62	14.03	340.5	2	X	1	4.2	-2.66	-13464.	2	4.0	-2.57	-11113.				
6	19.65	15.21	455.1	2	X	1	4.6	-2.62	-19614.	2	4.2	-2.32	-13464.				
7	25.61	15.45	1089.1	4	X	1	5.0	-2.23	-42906.	2	4.8	-2.11	-34635.	3	4.7	-1.95	-26825.
					X	4	4.6	-1.73	-19615.								
8	11.46	8.53	149.0	4	X	1	5.0	0.21	2622.	2	2.4	5.43	5050.	3	6.6	6.22	1389.
					X	4	17.5	0.79	28.								
9	29.33	13.09	2806.7	2	X	1	5.0	-1.70	-62686.	2	5.0	-1.13	-40612.				

RIVER AT TIME 61.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	1.52	1.47	128.7	3	X	1	6.0	2.12	709.	2	11.3	1.57	219.	3	17.7	1.23	111.
					X												
2	6.20	5.73	165.4	3	X	1	6.4	-1.71	-2566.	2	5.6	0.26	214.	3	6.0	1.86	709.
					X												
3	13.30	11.55	261.5	3	X	1	7.2	-2.71	-9879.	2	6.8	-2.19	-6180.	3	6.4	-1.24	-2566.
					X												
4	17.74	15.31	333.3	2	X	1	7.4	-2.44	-13189.	2	7.2	-2.14	-9879.				
5	20.00	16.38	365.5	2	X	1	7.6	-2.48	-15454.	2	7.4	-2.41	-13189.				
6	22.92	17.26	488.8	2	X	1	7.8	-2.39	-21474.	2	7.6	-2.13	-15454.				
7	28.74	17.13	1172.5	4	X	1	8.0	-1.96	-44512.	2	7.9	-1.85	-36481.	3	7.9	-1.71	-28782.
					X	4	7.8	-1.51	-21475.								
8	13.47	9.61	160.2	4	X	1	8.0	0.69	2323.	2	6.6	3.19	4229.	3	7.3	6.48	1767.
					X	4	17.5	0.79	28.								
9	32.85	13.63	3372.0	2	X	1	8.0	-1.45	-66857.	2	8.0	-0.94	-42487.				

RIVER AT TIME 62.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	2.90	2.77	134.2	3	X	1	9.4	1.74	1341.	2	12.0	1.92	448.	3	17.7	1.19	111.
					X												
2	9.93	8.85	181.4	3	X	1	10.0	-1.53	-3267.	2	9.8	-0.65	-968.	3	9.4	1.54	1341.
					X												
3	16.65	14.00	280.1	3	X	1	10.3	-2.16	-9696.	2	10.1	-1.77	-6525.	3	10.0	-1.11	-3268.
					X												
4	20.73	17.59	348.2	2	X	1	10.4	-1.93	-12376.	2	10.3	-1.72	-9696.				
5	22.87	18.33	383.6	2	X	1	10.4	-1.94	-14188.	2	10.4	-1.89	-12376.				
6	25.67	19.00	513.8	2	X	1	10.5	-1.83	-18983.	2	10.4	-1.66	-14188.				
7	31.32	18.58	1242.1	4	X	1	10.5	-1.46	-37506.	2	10.5	-1.37	-31043.	3	10.5	-1.26	-24863.
					X	4	10.5	-1.12	-18984.								
8	15.11	10.46	171.7	4	X	1	10.5	0.42	1593.	2	9.8	1.74	2913.	3	8.2	6.52	2171.
					X	4	17.6	0.79	28.								
9	35.34	14.37	3802.5	2	X	1	10.5	-1.05	-57756.	2	10.5	-0.67	-36101.				

FIG 8.6.B (viii)

RIVER AT TIME 62.50

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	4.43	4.15	143.1	3	X	1	12.5	1.23	1494.	2	13.4	2.19	906.	3	17.8	1.12	111.
					X												
2	12.31	11.06	105.4	3	X	1	12.6	-0.53	-1393.	2	12.7	0.02	46.	3	12.5	1.09	1494.
					X												
3	19.09	15.67	203.6	3	X	1	12.5	-1.11	-5685.	2	12.6	-0.82	-3553.	3	12.6	-0.39	-1393.
					X												
4	22.90	19.19	359.5	2	X	1	12.5	-1.04	-7497.	2	12.5	-0.89	-5685.				
5	24.96	19.88	394.2	2	X	1	12.4	-1.07	-3722.	2	12.5	-1.02	-7497.				
6	27.67	20.43	523.6	2	X	1	12.4	-1.06	-12038.	2	12.4	-0.91	-8722.				
7	33.21	19.62	1295.6	4	X	1	12.3	-0.89	-24970.	2	12.3	-0.83	-20546.				
					X	4	12.4	-0.63	-12040.					3	12.4	-0.74	-16218.
8	16.39	10.94	188.4	4	X	1	12.3	0.25	1020.	2	12.0	0.97	1877.	3	9.2	6.27	2550.
					X	4	17.6	0.79	28.								
9	37.16	15.37	4016.7	2	X	1	12.3	-0.63	-39400.	2	12.3	-0.39	-24027.				

RIVER AT TIME 63.00

SEGMENT	AV.DEPTH	AV.MEAN DEPTH	DOWNSTR WIDTH	NO.OF SUBSEGS	X X	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.	N	ELEV.	VELOC.	DISCH.
1	5.29	4.91	147.6	3	X	1	13.9	1.42	2016.	2	14.5	2.21	1235.	3	17.8	1.06	111.
					X												
2	14.09	11.98	292.4	3	X	1	13.8	0.45	1310.	2	13.9	0.75	1660.	3	13.9	1.25	2016.
					X												
3	20.22	16.45	300.3	3	X	1	13.6	0.07	386.	2	13.7	0.17	808.	3	13.8	0.33	1310.
					X												
4	23.94	19.96	365.0	2	X	1	13.5	0.01	98.	2	13.6	0.06	386.				
5	25.95	20.64	399.3	2	X	1	13.4	-0.01	-76.	2	13.5	0.01	98.				
6	28.61	21.12	528.3	2	X	1	13.3	-0.05	-648.	2	13.4	-0.01	-76.				
7	34.08	20.11	1320.9	4	X	1	13.2	-0.12	-3459.	2	13.2	-0.09	-2439.				
					X	4	13.3	-0.03	-649.					3	13.3	-0.07	-1495.
8	17.11	11.32	196.4	4	X	1	13.2	0.38	1613.	2	13.0	0.94	1927.	3	10.3	5.85	2813.
					X	4	17.7	0.79	28.								
9	38.00	15.83	4115.8	2	X	1	13.1	-0.10	-6465.	2	13.2	-0.03	-1846.				

NOTE cf 63.0 to 50.5

FIG. 8.6.B (ix)

FIG 8.6.C ELEVATIONS AT POINTS THROUGH A TIDE

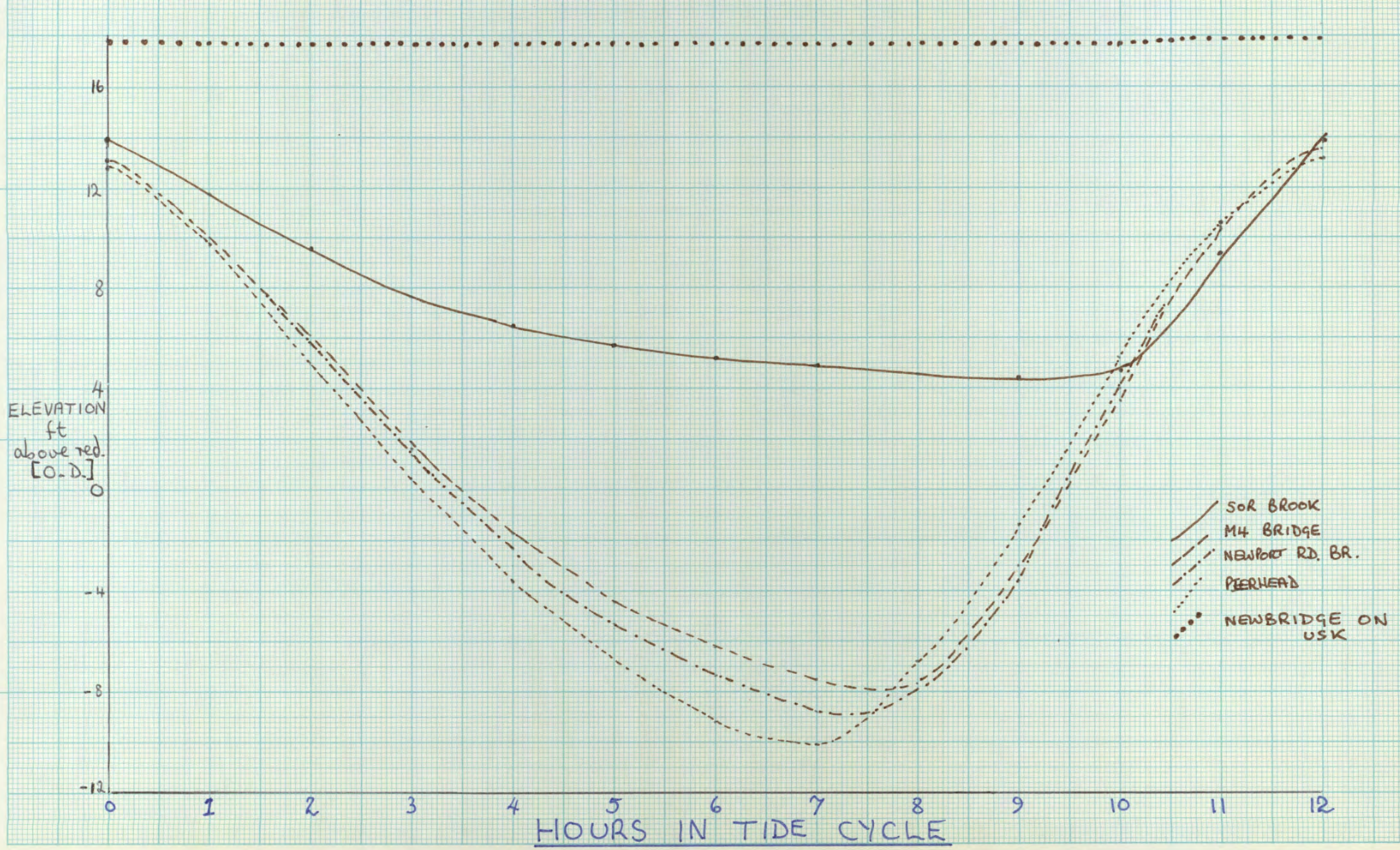
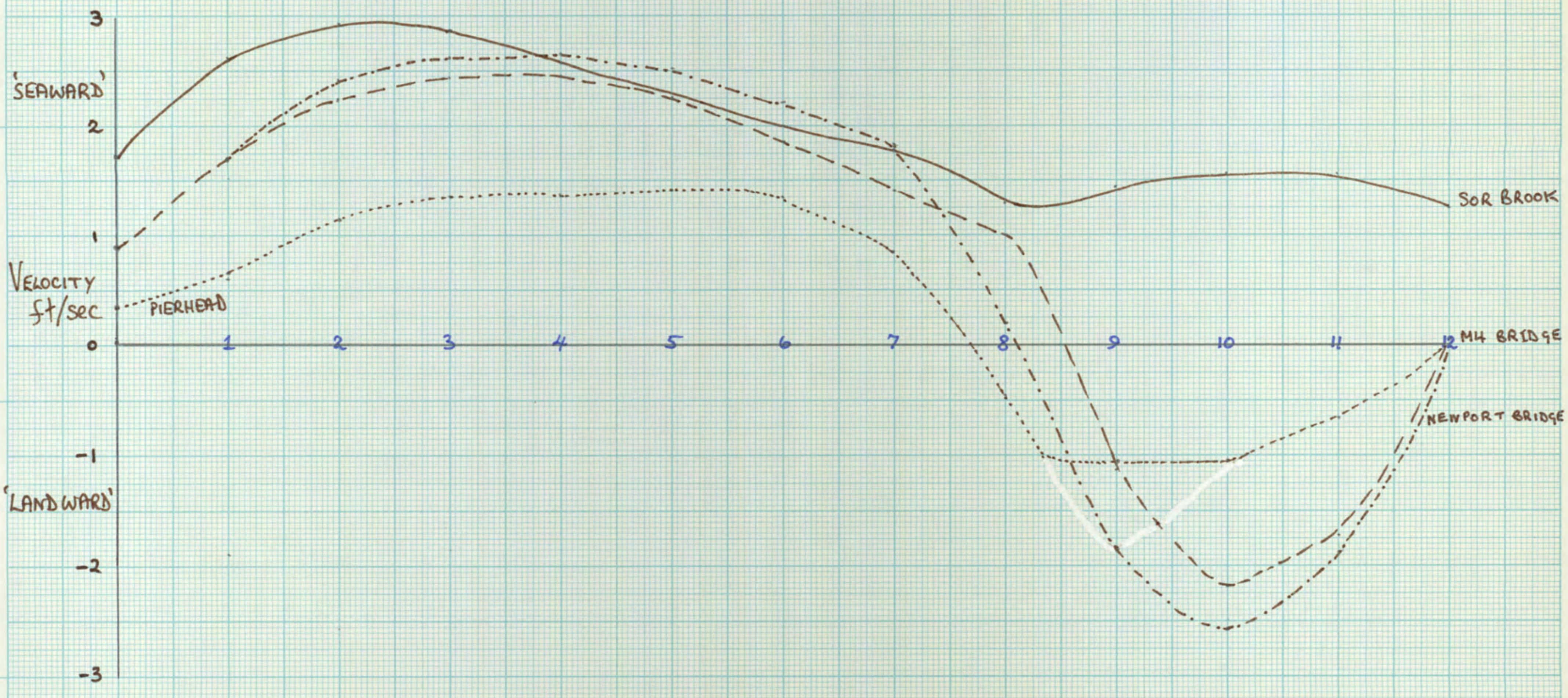


FIG 8.6.D VELOCITIES AT POINTS THROUGH A TIDE



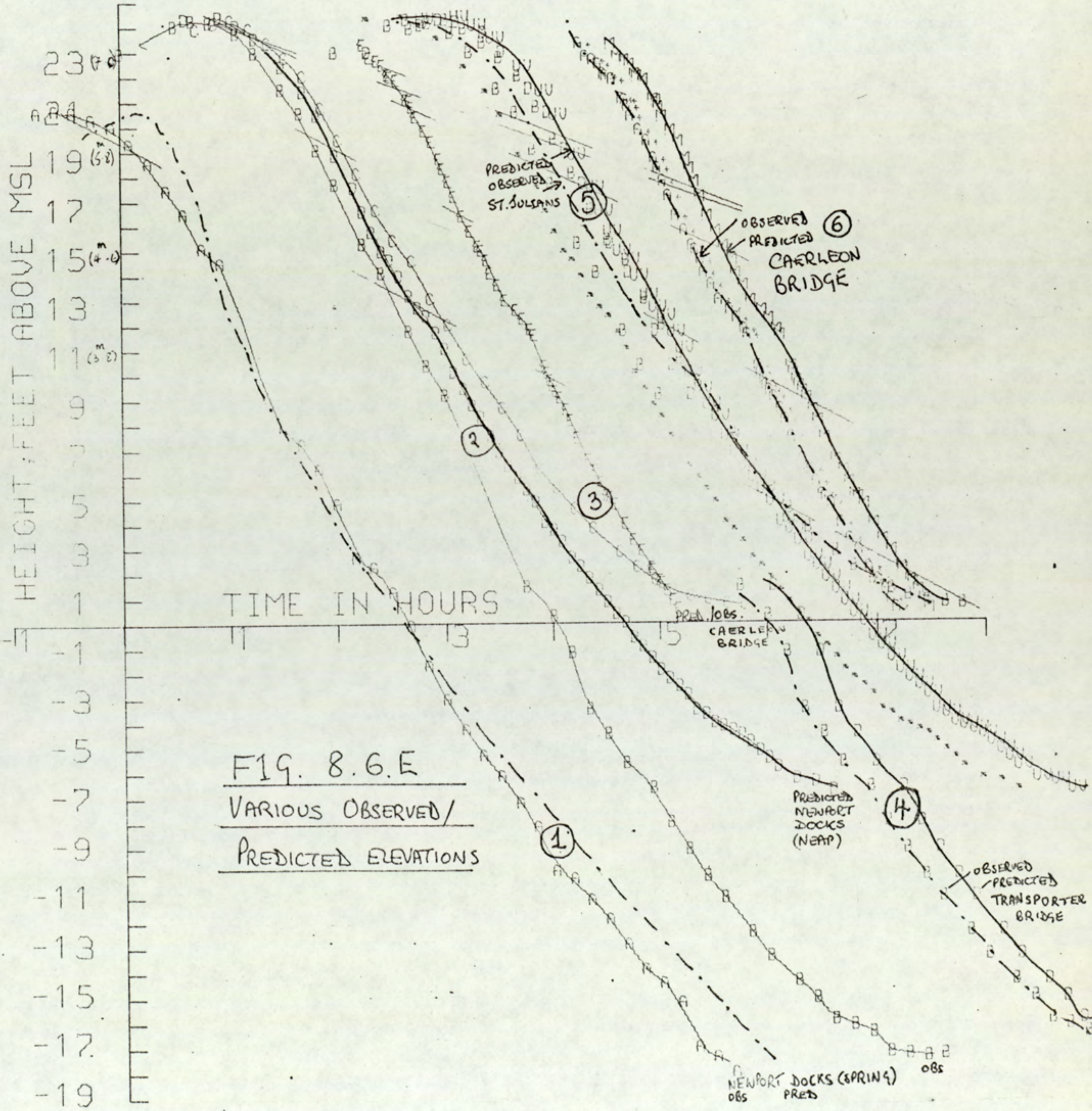
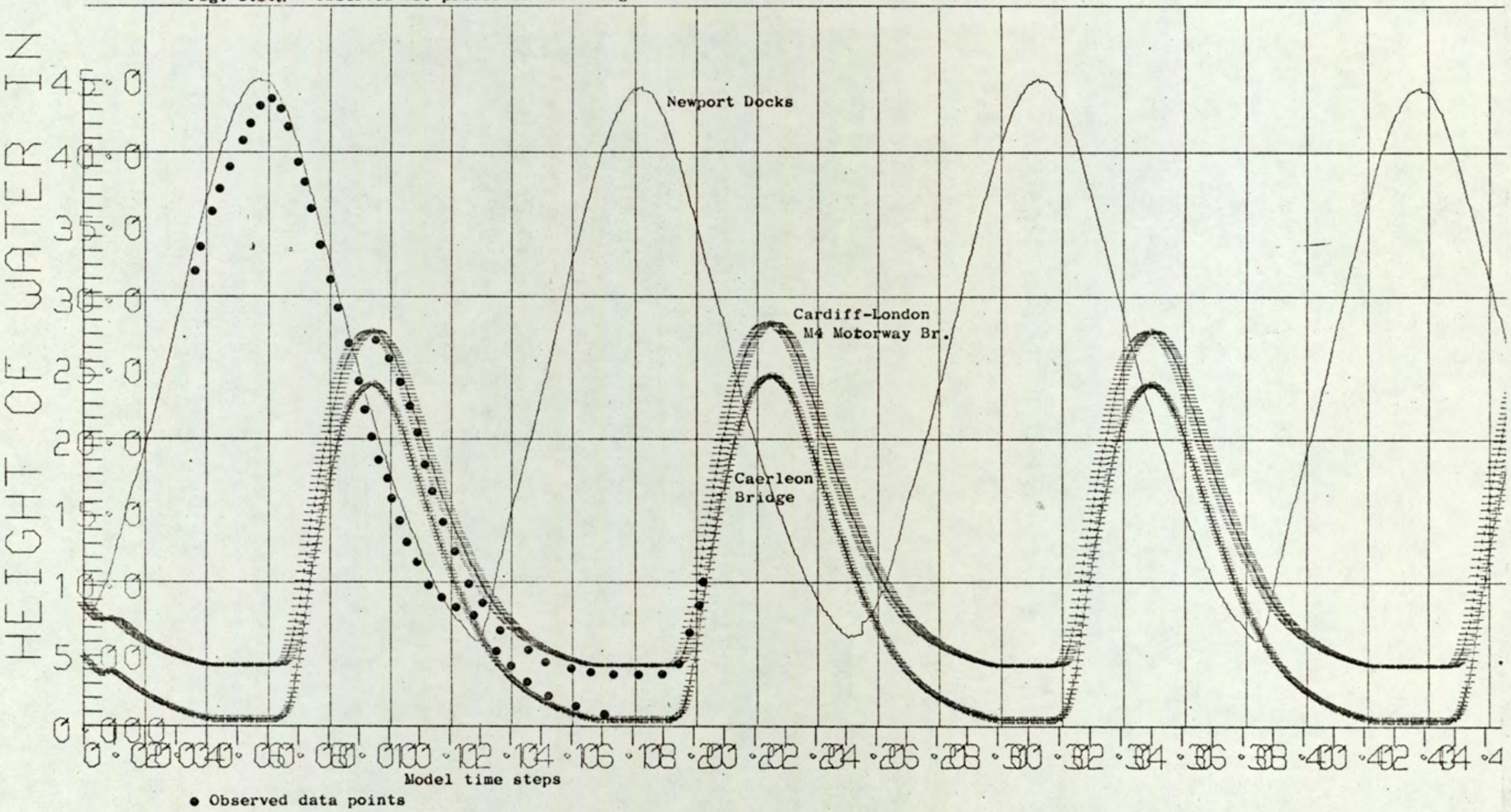


FIG. 8.6.E
VARIOUS OBSERVED/ PREDICTED ELEVATIONS

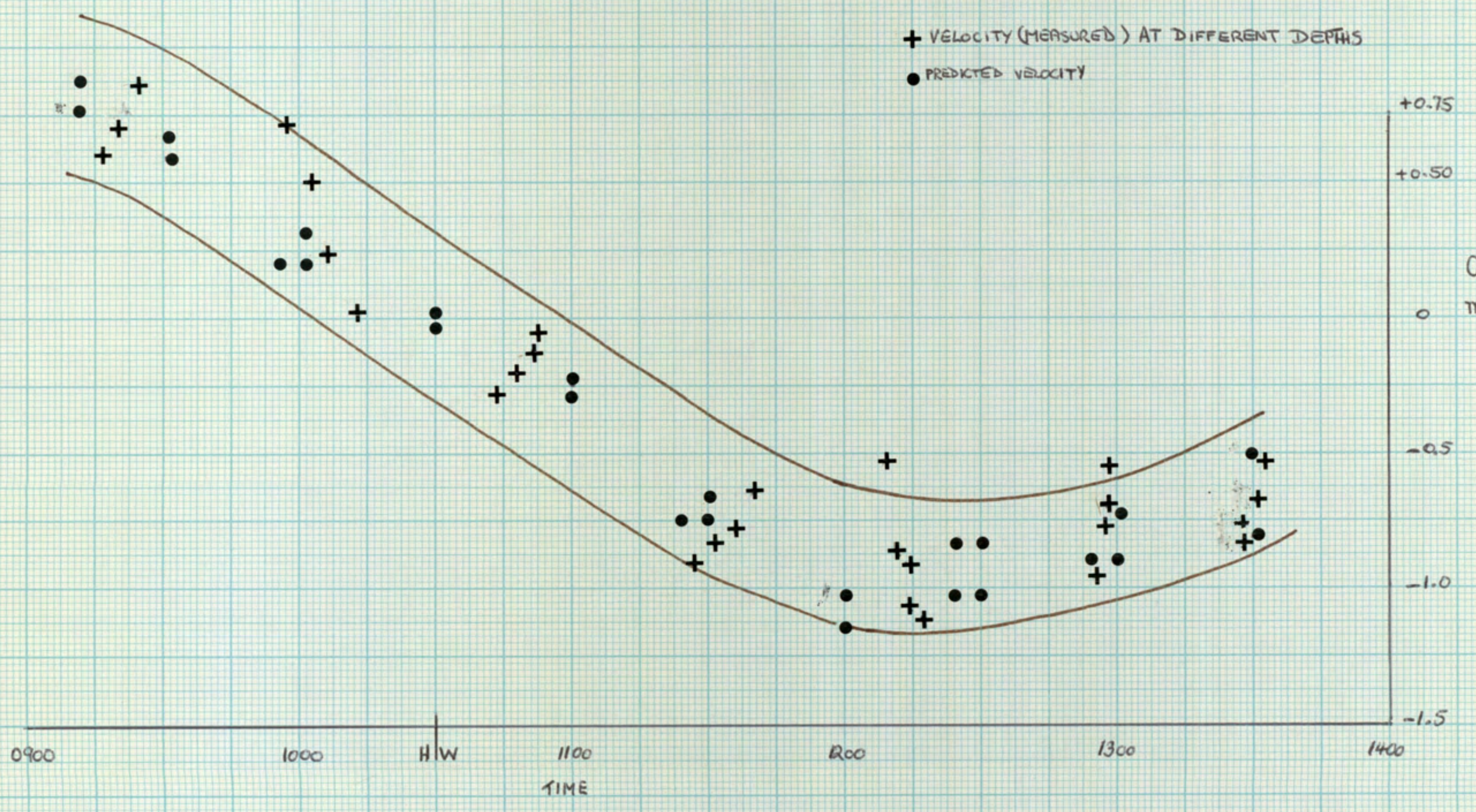
Fig. 8.5.F Observed vs. predicted Tide heights for the F1/F2 Model



URD 1-D MODEL PLOT HEIGHTS

FIG 8.6.9 VELOCITY CORRELATION AT

ST JULIANS

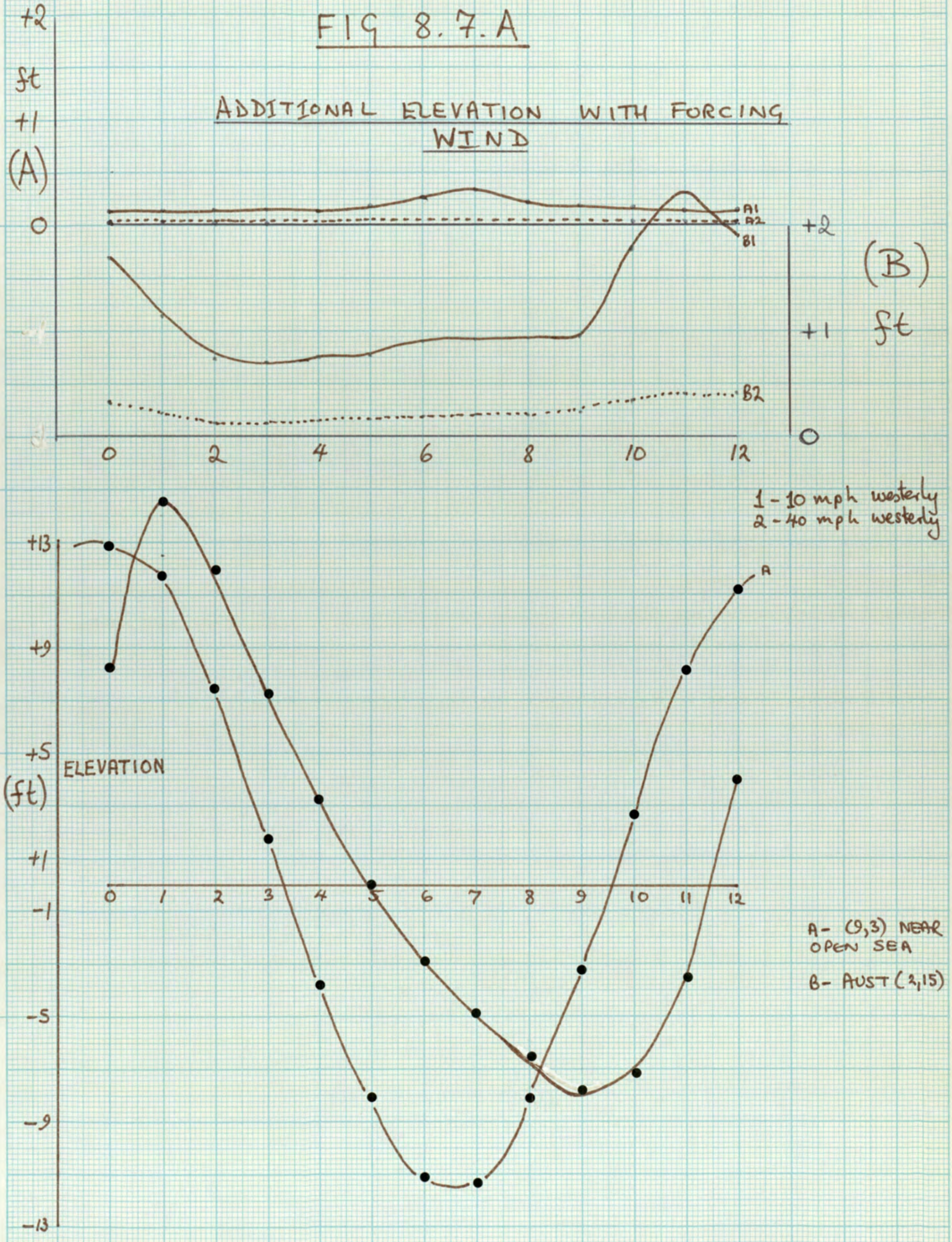


8.7 Wind Effects on the Bay

The geographical siting of the estuary causes it to be subjected to prevailing rain bearing westerlies. The model allows the effect of wind to be included in the bay by modifying the surface stress. This facility is of use in flood prevention modelling. The Usk flood plain suffers from flooding and housing tends to be built on lower and lower levels. Extensive protection work has been completed in the upper reaches, but large parts of the plain near Caerleon is still in danger when high tides combine with strong forcing winds. Fig. 8.7.A shows the effect of a very light breeze of about 10 knots westerly, and a 40 knot westerly gale. It is seem that the breeze has little but some effect (line A1/A2) but the gale adds up to 2.5 ft to a tide at high water. During the ebb phase the now opposing prevailing wind prevents a complete ebb and about 1ft additional depth is retained at the upper end of the system. By making the wind stress time dependant whole storm cycles can be imposed on the tide to predict water levels in the system.

FIG 8.7.A

ADDITIONAL ELEVATION WITH FORCING WIND



8.8 Pollutant Transport Inputs

A small data file is required to steer the two models from the first phase. Much of it represents restating the parameters used in the first phase, such data should be consistent with that used to generate the background file. The B.O.D. reaction assumed is the simple 1st order decay :

$$B(t) = B(0) \cdot e^{-0.23 * t} \qquad (8.8.A)$$

where 0.23 is the BOD decay rate in per day, t is in days. The line OMEAS in fig. 8.8.A requires restating of the head flows. This should be consistent with those used by F1/F2 although small variations are acceptable in practice (±10% of main DWF) , otherwise water transfer volumes will exceed space and the routine fails. Flows are in f.p.s. (ft.per sec) and 1 mgd = 1.11 cfs.

The line CMEAS BOD sets the initial BOD's in each segment. That for the bay and estuary are set globally to 0.5 . Similarly for D.O.

Brief data for each segment is also required to be restated (length, connection).

For each discharge, the following is required :

Segment and position withing segment of discharge point

Height above local datum of outfall for tidelocking (set to 9999. else)

Decay Constant for tidelocked load -proportion let out from 'reservoir' upon opening after tidelocking

Loads for each of the polluting components being simulated (up to 4 at one time)

Re-aeration is assumed to be 1st order process like 8.8.A with a variable rate. BOD decays is assumed to require DO on a 1 to 1 basis. No other sources and sinks of oxygen are modelled. This simplistic approach is sufficient at this stage, but is capable of expansion to justify the cost of computing background data. Re-aeration is automatically adjusted for height within the routine.

```
                      96 40 41 2
MMAX1 NMAX2 MMAX2       9  16 11
IDO IBOD                2   1
                      4 7 5 8 5
                      -15
                      15 0.5 0.1  10. 0.
 PRINTING               10  1 2 3 4 5 11 12 13 14 15
               TITLE  MOD B/R I/F BND FISCHER PHASE 2  TEST DATA - A
NAMES OF PARAMETERS    B.O.D. DIS.O.D.
                      14  3
DECAY RATES             .23     .50           0.0        0.0
COL 2 - 4 OF BAY      2     7 10       3  7 10          4  6 10
COL 5 - 7 OF BAY      5     5 10       6  4 10          7  4  9
COL 8 -10 OF BAY      8     4  9       9  5  9          10 5  7
COL11 -13 OF BAY      11    4  6       12 3  5          13 2  5
  COL    14 OF BAY    14    2  4    15 2 4
                      8000. 1 1  .TRUE.
RIVER-BAY I/F         2   6 4 7 4
BAY-OCEAN I/F VERT.   2   1 10  1 11
BAY-OCEAN I/F HORIZ   5   2 11  3 11  4 11  5 11  6 11
                      0.5 5 0 1 6
                      0 15 .TRUE.
OMEAS                 111. 0. 0. 0. 0. 0. 0. 28. 0.
CMEAS  BOD            .5 .5 .5 .5 .5 .5 .5 .5 .5
CMEAS  OD             .1 .1 .1 .1 .1 .1 .1 .1 .1
                      0. 0. 0. 0. 0. 0. 0. 0. 0.
SEGMENT DATA  SEG1    19812.0   10 10 10  2
SEGMENT DATA  SEG2    13593.0    1 10 10  3
SEGMENT DATA  SEG3    14916.0    2 10 10  4
SEGMENT DATA  SEG4     5406.0    3 10 10  5
SEGMENT DATA  SEG5     3458.0    4 10 10  6
SEGMENT DATA  SEG6     7310.0    5 10 10  7
SEGMENT DATA  SEG7    13079.0    6 10 10  9
SEGMENT DATA  SEG8    13343.0   10 10 10  9
SEGMENT DATA  SEG9     4677.0    7  8 10 10
PILL SOUTH              6   6674.   31.8   0.9     6.688E+05  0.0 0.0
TREDEGAR DRY DOCK       6   6394.   30.7   0.9     9.000E+05  0.0 0.0
RINGLAND                6   3207.   28.7   0.9     1.021E+06  0.0 0.0
MAINDEE                 5   3198.   25.5   0.9     8.000E+05  0.0 0.0
TOWN                    5    573.   21.5   0.9     3.600E+05  0.0 0.0
CIVIC                   4   4979.   11.1   0.9     4.208E+05  0.0 0.0
BARRACK                 4   4530.   23.0   0.9     2.000E+05  0.0 0.0
CENOTAPH SOUTH          4   4180.   22.3   0.9     5.328E+05  0.0 0.0
CENOTAPH NORTH          4   2812.   23.9   0.9     2.336E+05  0.0 0.0
BRYNGLAS & BETTWS       4   2229.  9999.   0.0     1.467E+06  0.0 0.0
ST JULIANS              3   9646.   22.0   0.9     3.400E+05  0.0 0.0
CAERLEON                3   6958.  9999.   0.0     1.333E+06  0.0 0.0
EASTERN VALLEY          2  11093.  9999.   0.0     3.000E+06  0.0 0.0
COUNTY                  6   2435.   24.1   0.9     1.600E+06  0.0 0.0
MAESGLAS                8   1750.  9999.   0.0     7.200E+05  0.0 0.0
```

FIG 8.8.A INPUT FOR POLLUTANT TRANSPORT

FIG 8.8 B PARAMETER OUTPUT PT/PT1

```
USK=SEVERN.13.1 FT AVERAGE TIDE AT BARRY=40.7 FT AT NEWPORT
MOD B/R I/F BNDRFISCHER PHASE 2  TEST DATA = ACTUAL (USK)

PROBLEM IDENTIFIERS DON'T MATCH     USK=    MOD
PROBLEM IDENTIFIERS DON'T MATCH     SEVE    B/R
PROBLEM IDENTIFIERS DON'T MATCH     RN.1    I/F
PROBLEM IDENTIFIERS DON'T MATCH     3.1     BNDR
PROBLEM IDENTIFIERS DON'T MATCH     FT A    FISC    CHECK TITLES
PROBLEM IDENTIFIERS DON'T MATCH     VERA    HER     FOR
PROBLEM IDENTIFIERS DON'T MATCH     GE T    PHAS    CONSISTENCY
PROBLEM IDENTIFIERS DON'T MATCH     IDE     E 2
PROBLEM IDENTIFIERS DON'T MATCH     AT B     TES
PROBLEM IDENTIFIERS DON'T MATCH     ARRY    T DA
PROBLEM IDENTIFIERS DON'T MATCH     =40.    TA =
PROBLEM IDENTIFIERS DON'T MATCH     7 FT     ACT
PROBLEM IDENTIFIERS DON'T MATCH      AT     UAL
PROBLEM IDENTIFIERS DON'T MATCH     NEWP    (USK
PROBLEM IDENTIFIERS DON'T MATCH     ORT     )
PROBLEM IDENTIFIERS DON'T MATCH
PROBLEM IDENTIFIERS DON'T MATCH
PROBLEM IDENTIFIERS DON'T MATCH
PROBLEM IDENTIFIERS DON'T MATCH                T
PROBLEM IDENTIFIERS DON'T MATCH             ITLE

                                            PRINT FLAGS
CYCLES     15  PRINTING AT TIMES
   1     2     3     4     5     11    12    13    14    15    0    0


     PARAMETER  RATE CNST.
                               DECAY &
     B.O.D.        0.230       REAERATION
   DIS.O.D.        0.500       RATE
                   0.000

BOUNDARY CONDITIONS IN OPEN SEA
        0.50000          0.10000          10.00000
        B.O.D.           O.D.             TEST COMPONENT
                                          (CONSERVATIVE)
   N    MF   ML

   2     7   10
   3     7   10
   4     6   10
   5     5   10    BAY OUTLINE
   6     4   10
   7     4    9
   8     4    9
   9     5    9
  10     5    7
  11     4    6
  12     3    5
  13     2    5
  14     2    4
  15     2    4
 RIVER-BAY INTERFACE POINTS   2
  NO  1  POINT    6   4
  NO  2  POINT    7   4

 I/F POINTS VERT HORIZ      2     5
     1    10             1   11            2   11              3   11
     6    11
 GRID SIZE= 8000.0 FEET.  DIFFUSION 1 STEPS PER HR.   MULTIPLIER=  1200.
 BAY DIFFUSSION  PARAMETERS
                                                 PHASE  0 OF CYCLE  1
```

8.9 Pollutant Transport Simulation

The routine operates in complete phases only. Convective transport operates in blocks of steps (6 currently), followed then by one diffusive step.At each time step a block of water is moved using the velocities generated by F1/F2. At each step, the net travel in the current direction, together with the grossinflow at the receiving node and the gro ss outflow from the 'leaving' node is computed for each segment.Subsegments are created as required, by an inflow block reaching a node and splitting into tributaries or just being length adjusted for the new channel dimesnions.To avoid new blocks swamping the system , after each scan they are cleaned, smaller ones being compounded into larger ones. Fig. 8.9.A shows the record of nodal flow over a flood phase of 5 hours (time steps of 10 minutes). Care should be taken to see that the headwater inflows (-111. and -28. in this case) reside in the segments gross flow for the whole of the simulation. Should they not, it indicates that the tide has exceeded the limit and the remaining projections are invalid. This should not occur if precisely the same data as that used for F1/F2 is used on this phase.

Reports are issued if requested on the behaviour of the tidelocked discharges. Fig. 8.9.B is a list of discharges and the weighted discharge over the long terms mean that they are issuing. The ADDNOW units refer to the multiple of the mean load, that has built up through tidelocking's reservoir effect. All discharges have a coefficient of 0.9, ie. 90% of reservoir is discharged in the first cycle after opening. Up to 25 times normal loads suddenly enter thesystem.

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-63703.	-111.	-231.25
3	-614900.	-219411.	-1469.84
5	-934542.	-826844.	-1044.77
7	761434.	-923406.	-13.77
9	1950788.	847566.	114.46

2

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	-219411.	-63703.	-723.06
4	-826844.	-614900.	-1372.48
6	-923406.	-934542.	-539.84
8	86131.	-28.	91.28

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-63703.	-111.	-236.76
3	-614900.	-219412.	-1621.31
5	-934542.	-826843.	-1084.85
7	761487.	-923404.	-13.47
9	1950864.	847622.	112.29

3

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	-219412.	-63703.	-768.00
4	-826843.	-614900.	-1483.19
6	-923404.	-934542.	-539.36
8	86135.	-28.	90.05

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-49800.	-111.	-188.69
3	105029.	-151584.	-84.80
5	1361048.	751652.	1069.14
7	10882730.	3127701.	1060.79
9	15586844.	11283164.	1004.06

4

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	-151584.	-49800.	-569.42
4	751652.	105029.	707.25
6	3127701.	1361048.	1142.73
8	400435.	-28.	394.01

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-49800.	-111.	-192.33
3	105029.	-151583.	-79.80
5	1361049.	751653.	907.31
7	10882733.	3127704.	973.39
9	15586868.	11283170.	939.46

5

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	-151583.	-49800.	-594.60
4	751653.	105029.	590.61
6	3127704.	1361049.	1017.53
8	400438.	-28.	372.04

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-49800.	-111.	-196.12
3	105029.	-151583.	-75.35
5	1361048.	751652.	788.03
7	10882730.	3127701.	899.30
9	15586865.	11283167.	882.66

6

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	-151583.	-49800.	-622.11
4	751652.	105029.	506.99
6	3127701.	1361048.	917.05
8	400438.	-28.	352.38

FIG 8.9 A WATER MOVEMENTS DURING FLOOD TIDE (i)

7

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-49800.	-111.	-200.07	2	-151583.	-49800.	-652.28
3	105029.	-151583.	-71.38	4	751653.	105029.	444.12
5	1361049.	751653.	696.47	6	3127702.	1361049.	834.63
7	10882782.	3127702.	835.69	8	400438.	-28.	334.70
9	15586917.	11283219.	832.34				

8

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-49800.	-111.	-204.17	2	-151583.	-49800.	-685.53
3	105030.	-151583.	-67.80	4	751652.	105030.	395.12
5	1361048.	751652.	623.97	6	3127703.	1361048.	765.80
7	10882732.	3127703.	780.48	8	400438.	-28.	318.71
9	15586867.	11283169.	787.45				

9

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-49800.	-111.	-208.45	2	-151584.	-49800.	-722.36
3	105033.	-151584.	-64.56	4	751658.	105033.	355.86
5	1361058.	751658.	565.14	6	3127718.	1361058.	707.47
7	10882798.	3127718.	732.12	8	400438.	-28.	304.18
9	15586951.	11283235.	747.16				

10

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-22127.	-111.	-93.75	2	169958.	-22127.	481.46
3	2319292.	169958.	2466.48	4	3852750.	2319292.	2074.83
5	4878067.	3852750.	2015.72	6	7641844.	4878067.	1763.12
7	17070512.	7641844.	1200.87	8	804891.	-28.	560.09
9	26353968.	17875392.	1117.22				

11

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-22127.	-111.	-94.63	2	169958.	-22127.	440.88
3	2319296.	169958.	1918.61	4	3852754.	2319296.	1742.52
5	4878072.	3852754.	1772.98	6	7641846.	4878072.	1593.44
7	17070512.	7641846.	1122.24	8	804894.	-28.	516.71
9	26353984.	17875392.	1023.48				

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-22127.	-111.	-95.53
3	2319296.	169958.	1569.90
5	4878069.	3852752.	1582.42
7	17070496.	7641821.	1053.28
9	26353952.	17875376.	944.26

12

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	169958.	-22127.	406.61
4	3852752.	2319296.	1501.95
6	7641821.	4878069.	1453.55
8	804894.	-28.	479.57

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-22127.	-111.	-96.46
3	2319293.	169958.	1328.44
5	4878068.	3852751.	1428.84
7	17070528.	7641847.	992.30
9	26354000.	17875408.	876.42

13

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	169958.	-22127.	377.28
4	3852751.	2319293.	1319.75
6	7641847.	4878068.	1336.24
8	804894.	-28.	447.40

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-22127.	-111.	-97.39
3	2319292.	169958.	1151.36
5	4878069.	3852752.	1302.44
7	17070528.	7641848.	937.99
9	26353984.	17875408.	817.68

14

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	169958.	-22127.	351.90
4	3852752.	2319292.	1176.98
6	7641848.	4878069.	1236.45
8	804894.	-28.	419.28

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	-22127.	-111.	-98.35
3	2319304.	169959.	1015.95
5	4878089.	3852768.	1196.59
7	17070624.	7641897.	889.33
9	26354128.	17875520.	766.32

15

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	169959.	-22127.	329.72
4	3852768.	2319304.	1062.08
6	7641897.	4878089.	1150.54
8	804898.	-28.	394.49

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	252992.	-111.	1004.87
3	6354031.	1891096.	2704.56
5	9681531.	8333677.	2230.71
7	26845440.	13331362.	1345.76
9	41330544.	27999712.	1093.23

16

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1891096.	252992.	3110.16
4	8333677.	6354031.	2244.56
6	13331362.	9681531.	1937.04
8	1154278.	-28.	521.51

17

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	252993.	-111.	912.25
3	6354032.	1891097.	2260.79
5	9681529.	8333679.	2034.34
7	26845488.	13331360.	1258.64
9	41330608.	27999769.	1003.07

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1891097.	252993.	2304.47
4	8333679.	6354032.	2018.63
6	13331360.	9681529.	1786.85
8	1154281.	-28.	483.70

18

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	252993.	-111.	835.27
3	6354026.	1891094.	1942.12
5	9681523.	8333673.	1869.74
7	26845504.	13331383.	1132.11
9	41330608.	27999776.	926.64

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1891094.	252993.	1830.32
4	8333673.	6354026.	1834.02
6	13331383.	9681523.	1658.28
8	1154278.	-28.	451.00

19

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	252993.	-111.	770.26
3	6354039.	1891097.	1702.19
5	9681545.	8333682.	1729.79
7	26845504.	13331376.	1114.36
9	41330608.	27999776.	861.04

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1891097.	252993.	1518.00
4	8333682.	6354039.	1680.35
6	13331376.	9681545.	1546.96
8	1154281.	-28.	422.44

20

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	252993.	-111.	714.64
3	6354029.	1891097.	1515.02
5	9681530.	8333680.	1609.33
7	26845456.	13331390.	1053.94
9	41330592.	27999728.	804.11

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1891097.	252993.	1296.72
4	8333680.	6354029.	1550.44
6	13331390.	9681530.	1449.65
8	1154278.	-28.	397.28

21

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	252993.	-111.	666.52
3	6354055.	1891105.	1364.95
5	9681569.	8333706.	1504.56
7	26845600.	13331429.	999.75
9	41330736.	27999872.	754.24

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1891105.	252993.	1131.75
4	8333706.	6354055.	1439.18
6	13331429.	9681569.	1363.86
8	1154284.	-28.	374.95

8.9.A (iv)

22

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	1332537.	-111.	2592.59	2	4082574.	1332537.	2355.30
3	8011410.	4082574.	1641.51	4	9629577.	8011410.	1632.77
5	10695667.	9629577.	1614.35	6	13449652.	10695667.	1369.80
7	25076880.	13449652.	918.06	8	1160575.	-28.	356.83
9	38538656.	26237440.	666.55				

23

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	1332536.	-111.	2054.73	2	4082576.	1332536.	2002.82
3	8011416.	4082576.	1704.76	4	9629604.	8011416.	1547.05
5	10695708.	9629604.	1538.93	6	13449693.	10695708.	1313.65
7	25076960.	13449693.	880.75	8	1160578.	-28.	338.71
9	38538768.	26237536.	632.33				

24

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	1332536.	-111.	1701.70	2	4082576.	1332536.	1742.11
3	8011408.	4082576.	1586.92	4	9629575.	8011408.	1469.88
5	10695679.	9629575.	1479.33	6	13449664.	10695679.	1261.92
7	25076944.	13449664.	846.35	8	1160578.	-28.	322.35
9	38538736.	26237520.	601.44				

25

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	1332536.	-111.	1452.19	2	4082576.	1332536.	1541.45
3	8011416.	4082576.	1484.32	4	9629604.	8011416.	1400.05
5	10695708.	9629604.	1407.55	6	13449721.	10695708.	1214.11
7	25076992.	13449721.	814.53	8	1160575.	-28.	307.49
9	38538784.	26237552.	573.43				

26

BRANCH	INFLOW	OUTFLOW	TRAVEL	BRANCH	INFLOW	OUTFLOW	TRAVEL
1	1332541.	-111.	1266.50	2	4082581.	1332541.	1382.25
3	8011417.	4082581.	1394.18	4	9629605.	8011417.	1336.55
5	10695709.	9629605.	1349.91	6	13449694.	10695709.	1169.79
7	25076912.	13449694.	785.02	8	1160578.	-28.	293.94
9	38538688.	26237488.	547.92				

27

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	1332545.	-111.	1122.91
3	8011443.	4082592.	1314.37
5	10695748.	9629631.	1296.81
7	25077088.	13449761.	757.58
9	38538928.	26237664.	524.58

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	4082592.	1332545.	1252.85
4	9629631.	8011443.	1278.56
6	13449761.	10695748.	1128.60
8	1160581.	-28.	281.54

28

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	501511.	-111.	405.26
3	1666824.	1030738.	290.48
5	2086099.	1917590.	253.87
7	4108238.	2480210.	128.92
9	5871413.	4387197.	82.65

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1030738.	501511.	348.23
4	1917590.	1666824.	258.05
6	2480210.	2086099.	212.37
8	278960.	-28.	66.99

29

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	501511.	-111.	339.33
3	1666885.	1030742.	287.84
5	2086181.	1917672.	252.32
7	4108401.	2480321.	128.30
9	5871576.	4387360.	82.23

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1030742.	501511.	342.17
4	1917672.	1666885.	256.35
6	2480321.	2086181.	211.31
8	278960.	-28.	66.32

30

BRANCH	INFLOW	OUTFLOW	TRAVEL
1	501511.	-111.	374.60
3	1666824.	1030738.	285.24
5	2086134.	1917611.	250.77
7	4108354.	2480274.	127.68
9	5871529.	4387313.	81.82

BRANCH	INFLOW	OUTFLOW	TRAVEL
2	1030738.	501511.	336.33
4	1917611.	1666824.	254.65
6	2480274.	2086134.	210.26
8	278960.	-28.	65.67

FIG 8.9.B TIDELOCKED DISCHARGES HISTORY

```
CYCLE,PHASE & HRS. IN IT ARE =          1          1          7  PHASE= EBB
OUTFALL  1  OPEN.ADDNOW=  13.00000 TO PT.     6     5
OUTFALL  1  OPEN.ADDNOW=   1.30000 TO PT.     6     5
OUTFALL  2  OPEN.ADDNOW=  14.00000 TO PT.     6     5
OUTFALL  1  OPEN.ADDNOW=   0.13000 TO PT.     6     5
OUTFALL  2  OPEN.ADDNOW=   1.40000 TO PT.     6     5
OUTFALL  1  OPEN.ADDNOW=   0.01300 TO PT.     6     5
OUTFALL  2  OPEN.ADDNOW=   0.14000 TO PT.     6     5
OUTFALL  3  OPEN.ADDNOW=  16.00000 TO PT.     6     3
OUTFALL  2  OPEN.ADDNOW=   0.01400 TO PT.     6     5
OUTFALL  3  OPEN.ADDNOW=   1.60000 TO PT.     6     3
OUTFALL  3  OPEN.ADDNOW=   0.16000 TO PT.     6     2
OUTFALL  3  OPEN.ADDNOW=   0.01600 TO PT.     6     2
OUTFALL  4  OPEN.ADDNOW=  19.00000 TO PT.     5     2
OUTFALL 11  OPEN.ADDNOW=  19.00000 TO PT.     3     7
OUTFALL  4  OPEN.ADDNOW=   1.90001 TO PT.     5     2
OUTFALL  9  OPEN.ADDNOW=  20.00000 TO PT.     4     2
OUTFALL 11  OPEN.ADDNOW=   1.90001 TO PT.     3     6
OUTFALL  4  OPEN.ADDNOW=   0.19000 TO PT.     5     2
OUTFALL  7  OPEN.ADDNOW=  21.00000 TO PT.     4     3
OUTFALL  9  OPEN.ADDNOW=   2.00002 TO PT.     4     2
OUTFALL 11  OPEN.ADDNOW=   0.19000 TO PT.     3     6
OUTFALL  4  OPEN.ADDNOW=   0.01900 TO PT.     5     2
OUTFALL  7  OPEN.ADDNOW=   2.10001 TO PT.     4     3
OUTFALL  8  OPEN.ADDNOW=  22.00000 TO PT.     4     3
OUTFALL  9  OPEN.ADDNOW=   0.20000 TO PT.     4     2
OUTFALL 11  OPEN.ADDNOW=   0.01900 TO PT.     3     6
OUTFALL 14  OPEN.ADDNOW=  22.00000 TO PT.     6     2
OUTFALL  6  OPEN.ADDNOW=  23.00000 TO PT.     4     3
OUTFALL  7  OPEN.ADDNOW=   0.21000 TO PT.     4     3
OUTFALL  8  OPEN.ADDNOW=   2.20001 TO PT.     4     3
OUTFALL  9  OPEN.ADDNOW=   0.02000 TO PT.     4     2
OUTFALL 14  OPEN.ADDNOW=   2.20001 TO PT.     6     2
OUTFALL  5  OPEN.ADDNOW=  24.00000 TO PT.     5     1
OUTFALL  6  OPEN.ADDNOW=   2.30000 TO PT.     4     3
OUTFALL  7  OPEN.ADDNOW=   0.02100 TO PT.     4     3
OUTFALL  8  OPEN.ADDNOW=   0.22000 TO PT.     4     3
OUTFALL 14  OPEN.ADDNOW=   0.22000 TO PT.     6     2
OUTFALL  5  OPEN.ADDNOW=   2.40001 TO PT.     5     1
OUTFALL  6  OPEN.ADDNOW=   0.23000 TO PT.     4     3
OUTFALL  8  OPEN.ADDNOW=   0.02200 TO PT.     4     2
OUTFALL 14  OPEN.ADDNOW=   0.02200 TO PT.     6     2
OUTFALL  5  OPEN.ADDNOW=   0.24000 TO PT.     5     1
OUTFALL  6  OPEN.ADDNOW=   0.02300 TO PT.     4     2
OUTFALL  5  OPEN.ADDNOW=   0.02400 TO PT.     5     1
```

GRID POINT | SEGMENT

The effect on the continuity requirements of some methods by this type of shock load is often catastrophic.

Fig. 8.9.C gives the full projections of BOD/OD for high and low water for a 100 mgd simulation. Both predictions fall within the computed confidence limits projected from field data shown in fig. 9.4.B and C . However, the actual distributions are skew and disatisfactory. Segment 8, the R. Ebbw,has no data available and in any event , the first sub-segment point should be ignored, as the discharge is close to the head of a small stretch.

Indications are that diffusion is not sufficiently well represented and thus cannot dilute shock loads sufficiently well.

Levels of BOD transported to and from the bay range from 1.8 to 0.49.

Reducing the input BOD to zero on the boundary as well as setting D.O. levels to completely saturated, improves the BOD profile significantly, and the minimum D.O. by up to 10% sat. (fig. 8.9.D). At high water the beneficial effect is in the lower reaches only, showing low dispersion characteristics. This lack of mixing tends to retain the Eastern Valley output in a plug and as the tide comes in this is pushed as a plug towards Newbridge (fig. 8.9.E)

Simulating a doubling of the Eastern Valleys outfall created a plug of pollutant of BOD about 22 mg/l when 12-13 mg/l is to be expected with localised high DO deficits.

Switching off the diffusion present generated high levels around lower/middle Newport in the region 14-25 mg/l and unrealistic DO levels of only 30%-40% at low water for some 6 miles. The Eastern Valleys outfall plug remains fairly discrete at about 30 mg/l , and 20%-50% DO, oscillating with only convective motion between successive high waters. Whereas the conceptual

```
CONCENTRATIONS IN THE RIVER , AT END OF PHASE 3 OF CYCLE  1

SEG  1  PTS  3 LENGTH(MLS)  3.75227
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    46.12 * 0.0087  0.13720E 00  0.23351E-01
  7744.27 * 1.4667  0.76290E-01  0.75727E-02  x100
 17604.04 * 3.3341  0.84517E 00  0.81319E-01  %O.D.
SEG  2  PTS  4 LENGTH(MLS)  2.57443
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
  1022.36 * 0.1936  0.12930E 01  0.12659E 00
  3778.72 * 0.7157  0.33476E 01  0.33512E 00
  7449.53 * 1.4109  0.42766E 01  0.44172E 00
 11489.63 * 2.1761  0.71578E 01  0.47854E 00
SEG  3  PTS  5 LENGTH(MLS)  2.82500
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   325.69 * 0.0617  0.64274E 01  0.47583E 00
  2203.16 * 0.4173  0.63078E 01  0.46556E 00
  5555.73 * 1.0522  0.56908E 01  0.43186E 00
  9395.45 * 1.7794  0.51807E 01  0.40242E 00
 13175.04 * 2.4953  0.45806E 01  0.37463E 00
SEG  4  PTS  5 LENGTH(MLS)  1.02386
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   509.33 * 0.0965  0.45434E 01  0.37348E 00
  1527.98 * 0.2894  0.43477E 01  0.36564E 00
  2546.63 * 0.4823  0.46743E 01  0.35746E 00
  3813.58 * 0.7223  0.45474E 01  0.35260E 00
  4988.58 * 0.9448  0.44823E 01  0.34689E 00
SEG  5  PTS  4 LENGTH(MLS)  0.65492
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   456.70 * 0.0865  0.42190E 01  0.34168E 00
  1370.08 * 0.2595  0.39964E 01  0.33585E 00
  2283.48 * 0.4325  0.38138E 01  0.32941E 00
  3099.09 * 0.5869  0.39203E 01  0.32512E 00
SEG  6  PTS 10 LENGTH(MLS)  1.38447
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   272.02 * 0.0515  0.37370E 01  0.32026E 00
   816.05 * 0.1546  0.36009E 01  0.31535E 00
  1360.09 * 0.2576  0.34520E 01  0.30927E 00
  2097.44 * 0.3972  0.35258E 01  0.30408E 00
  3028.10 * 0.5735  0.34863E 01  0.29514E 00
  3958.77 * 0.7498  0.34613E 01  0.28715E 00
  4889.43 * 0.9260  0.31869E 01  0.27799E 00
  5820.09 * 1.1023  0.30290E 01  0.26922E 00
  6750.75 * 1.2786  0.31793E 01  0.26162E 00
  7262.98 * 1.3756  0.30308E 01  0.25828E 00
SEG  7  PTS 20 LENGTH(MLS)  2.47703
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   164.18 * 0.0311  0.27824E 01  0.24395E 00
   620.83 * 0.1176  0.26551E 01  0.23736E 00
  1123.67 * 0.2128  0.24430E 01  0.22413E 00
  1544.42 * 0.2925  0.23041E 01  0.21264E 00
  1965.17 * 0.3722  0.21570E 01  0.20019E 00
  2385.92 * 0.4519  0.20216E 01  0.18839E 00
  2806.67 * 0.5316  0.18981E 01  0.17729E 00
  3341.43 * 0.6328  0.17840E 01  0.16677E 00
  3990.18 * 0.7557  0.16708E 01  0.15588E 00
  4638.94 * 0.8786  0.15769E 01  0.14675E 00
  5287.69 * 1.0015  0.14892E 01  0.13841E 00
  5936.45 * 1.1243  0.13995E 01  0.13024E 00
  6585.20 * 1.2472  0.13190E 01  0.12336E 00
  7376.29 * 1.3970  0.12333E 01  0.11634E 00
  8309.71 * 1.5738  0.11377E 01  0.10918E 00
  9243.13 * 1.7506  0.10545E 01  0.10343E 00
 10176.55 * 1.9274  0.98218E 00  0.98919E-01
 11109.96 * 2.1042  0.91066E 00  0.95016E-01
 12043.39 * 2.2809  0.84382E 00  0.91993E-01
 12794.49 * 2.4232  0.79020E 00  0.90143E-01
SEG  8  PTS  4 LENGTH(MLS)  2.52708
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
     3.66 * 0.0007  0.25499E 03  0.19998E 01
  3868.23 * 0.7326  0.27025E 01  0.26106E 00
 10144.79 * 1.9214  0.92682E 00  0.10125E 00
 12951.68 * 2.4530  0.71714E 00  0.86627E-01
SEG  9  PTS 10 LENGTH(MLS)  0.88530
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   143.26 * 0.0271  0.72041E 00  0.82225E-01
   537.96 * 0.1019  0.70588E 00  0.88155E-01
  1040.83 * 0.1971  0.66245E 00  0.86098E-01
  1543.71 * 0.2924  0.61726E 00  0.88654E-01
  2046.58 * 0.3876  0.58032E 00  0.89784E-01
  2549.45 * 0.4829  0.54588E 00  0.91410E-01
  3183.52 * 0.6029  0.52008E 00  0.92977E-01
  3872.59 * 0.7334  0.50796E 00  0.93923E-01
  4345.00 * 0.8229  0.50394E 00  0.94163E-01
  4593.97 * 0.8701  0.50265E 00  0.94269E-01
                    B.O.D.       O.D
```

FEET FROM ZERO

MILES FROM ZERO

FIG 8.9.C
PT/PT1
PREDICTIONS
(L.W.) (i)

```
CONCENTRATIONS IN THE RIVER , AT END OF PHASE 4 OF CYCLE

SEG  1  PTS 14 LENGTH(MLS)  3.75227
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
     12.44 * 0.0024  0.15037E 00  0.24022E-01
   1675.88 * 0.3174  0.15036E 00  0.18198E-01
   3716.04 * 0.7038  0.11564E 01  0.12180E 00
   4483.23 * 0.8491  0.39512E 01  0.27239E 00
   5239.28 * 0.9923  0.65745E 01  0.52036E 00
   6612.85 * 1.2524  0.88410E 01  0.69139E 00
   8603.94 * 1.6295  0.80290E 01  0.73264E 00
  10595.02 * 2.0066  0.70880E 01  0.69112E 00
  12586.11 * 2.3837  0.64727E 01  0.65240E 00
  14577.20 * 2.7608  0.59879E 01  0.60656E 00
  16568.30 * 3.1379  0.56973E 01  0.57799E 00
  17938.53 * 3.3974  0.53769E 01  0.55748E 00
  18687.89 * 3.5394  0.52772E 01  0.55208E 00
  19437.25 * 3.6813  0.52491E 01  0.55120E 00
SEG  2  PTS  8 LENGTH(MLS)  2.57443
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    741.05 * 0.1404  0.48985E 01  0.53104E 00
   2378.22 * 0.4504  0.43096E 01  0.48915E 00
   4170.45 * 0.7899  0.38995E 01  0.44338E 00
   5962.68 * 1.1293  0.37517E 01  0. 1391E 00
   7754.91 * 1.4687  0.31866E 01  0.36620E 00
   9547.15 * 1.8082  0.30756E 01  0.33802E 00
  11339.39 * 2.1476  0.27916E 01  0.30464E 00
  12914.24 * 2.4459  0.22113E 01  0.27935E 00
SEG  3  PTS 10 LENGTH(MLS)  2.92500
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    502.82 * 0.0952  0.21173E 01  0.26907E 00
   1677.50 * 0.3177  0.18799E 01  0.24911E 00
   3021.23 * 0.5722  0.16791E 01  0.21358E 00
   4540.21 * 0.8599  0.14738E 01  0.19234E 00
   6234.44 * 1.1808  0.13743E 01  0.17291E 00
   7928.66 * 1.5016  0.11292E 01  0.15792E 00
   9622.89 * 1.8225  0.99688E 00  0.14587E 00
  11317.12 * 2.1434  0.89445E 00  0.13724E 00
  13011.35 * 2.4643  0.80346E 00  0.13040E 00
  14387.21 * 2.7248  0.74025E 00  0.12639E 00
SEG  4  PTS  5 LENGTH(MLS)  1.02386
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    241.94 * 0.0458  0.71495E 00  0.12450E 00
   1168.00 * 0.2212  0.69438E 00  0.12246E 00
   2536.23 * 0.4803  0.65294E 00  0.11992E 00
   3904.47 * 0.7395  0.57205E 00  0.11863E 00
   4997.29 * 0.9465  0.55995E 00  0.11823E 00
SEG  5  PTS  3 LENGTH(MLS)  0.65492
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    667.09 * 0.1263  0.54879E 00  0.11793E 00
   2004.10 * 0.3796  0.53492E 00  0.11754E 00
   3066.01 * 0.5807  0.52895E 00  0.11734E 00
SEG  6  PTS  7 LENGTH(MLS)  1.33447
DISTANCE         ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    215.90 * 0.0409  0.52213E 00  0.11715E 00
   1051.10 * 0.1991  0.51765E 00  0.11699E 00
   2289.69 * 0.4337  0.50992E 00  0.11660E 00
   3528.27 * 0.6682  0.50696E 00  0.11623E 00
   4766.86 * 0.9028  0.50653E 00  0.11581E 00
   6005.45 * 1.1374  0.54166E 00  0.11537E 00
   6967.36 * 1.3196  0.52867E 00  0.11490E 00
```

FIG 8.9.C H.W. (i)

```
SEG  7  PTS 14 LENGTH(MLS)  2.47708
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    263.36 * 0.0499  0.48527E 00  0.11439E 00
   1046.97 * 0.1983  0.48533E 00  0.11419E 00
   2087.45 * 0.3954  0.48559E 00  0.11353E 00
   3127.94 * 0.5924  0.48607E 00  0.11286E 00
   4168.42 * 0.7895  0.48669E 00  0.11220E 00
   5208.90 * 0.9865  0.48734E 00  0.11157E 00
   6249.39 * 1.1836  0.48796E 00  0.11100E 00
   7255.60 * 1.3742  0.48860E 00  0.11042E 00
   8227.54 * 1.5582  0.48906E 00  0.10999E 00
   9199.47 * 1.7423  0.48966E 00  0.10944E 00
  10171.41 * 1.9264  0.49006E 00  0.10908E 00
  11143.35 * 2.1105  0.49063E 00  0.10855E 00
  12115.29 * 2.2946  0.49107E 00  0.10814E 00
  12840.10 * 2.4313  0.49126E 00  0.10795E 00
SEG  8  PTS 24 LENGTH(MLS)  2.52706
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
      0.99 * 0.0002  0.33769E 01  0.39753E 00
    835.91 * 0.1583  0.34231E 01  0.39153E 00
   2191.62 * 0.4151  0.13908E 01  0.16242E 00
   3047.27 * 0.5771  0.70168E 00  0.12526E 00
   3663.93 * 0.6939  0.56046E 00  0.11768E 00
   4230.55 * 0.8012  0.50096E 00  0.11627E 00
   4703.81 * 0.8909  0.48898E 00  0.11593E 00
   5082.79 * 0.9626  0.48426E 00  0.11552E 00
   5461.77 * 1.0344  0.48308E 00  0.11502E 00
   5840.75 * 1.1062  0.48320E 00  0.11449E 00
   6219.73 * 1.1780  0.48371E 00  0.11398E 00
   6680.97 * 1.2653  0.48429E 00  0.11349E 00
   7224.46 * 1.3683  0.48513E 00  0.11282E 00
   7767.95 * 1.4712  0.48592E 00  0.11218E 00
   8311.43 * 1.5741  0.48678E 00  0.11149E 00
   8854.92 * 1.6771  0.48753E 00  0.11086E 00
   9398.41 * 1.7800  0.48822E 00  0.11029E 00
   9943.38 * 1.8832  0.48882E 00  0.10979E 00
  10489.84 * 1.9867  0.48938E 00  0.10933E 00
  11036.29 * 2.0902  0.48990E 00  0.10890E 00
  11582.74 * 2.1937  0.49031E 00  0.10857E 00
  12129.20 * 2.2972  0.49072E 00  0.10823E 00
  12675.65 * 2.4007  0.49129E 00  0.10776E 00
  13145.90 * 2.4898  0.49170E 00  0.10744E 00
SEG  9  PTS  9 LENGTH(MLS)  0.88580
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
     24.23 * 0.0046  0.49217E 00  0.10709E 00
    378.09 * 0.0716  0.49248E 00  0.10681E 00
   1015.07 * 0.1922  0.49277E 00  0.10655E 00
   1629.79 * 0.3087  0.49352E 00  0.10588E 00
   2244.51 * 0.4251  0.49397E 00  0.10548E 00
   2859.23 * 0.5415  0.49495E 00  0.10459E 00
   3473.95 * 0.6579  0.49531E 00  0.10426E 00
   4088.67 * 0.7744  0.49620E 00  0.10346E 00
   4536.51 * 0.8592  0.49681E 00  0.10291E 00
```

FIG 8.9.C H.W. (ii)

```
CONCENTRATIONS IN THE RIVER , AT END OF PHASE 3 OF CYC

SEG  1  PTS  3 LENGTH(MLS)  3.75227
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    46.12 * 0.0087  0.13729E 00  0.20867E-01
  7744.27 * 1.4667  0.76209E-01  0.63116E-02
 17604.04 * 3.3341  0.84517E 00  0.68210E-01
SEG  2  PTS  4 LENGTH(MLS)  2.57443
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
  1022.36 * 0.1936  0.12930E 01  0.10765E 00
  3778.72 * 0.7157  0.33476E 01  0.22853E 00
  7449.53 * 1.4109  0.42766E 01  0.38424E 00
 11439.63 * 2.1761  0.71579E 01  0.42024E 00
SEG  3  PTS  5 LENGTH(MLS)  2.92500
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   325.69 * 0.0617  0.64274E 01  0.41885E 00
  2203.16 * 0.4173  0.6[illegible]078E 01  0.41366E 00
  5555.73 * 1.0522  0.56909E 01  0.38612E 00
  9395.45 * 1.7794  0.5[illegible]806E 01  0.36196E 00
 13175.04 * 2.4953  0.45885E 01  0.33944E 00
SEG  4  PTS  5 LENGTH(MLS)  1.92306
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   509.33 * 0.0965  0.45403E 01  0.33904E 00
  1527.99 * 0.2904  0.43475E 01  0.33302E 00
  2546.63 * 0.4823  0.4[illegible]740E 01  0.32690E 00
  3013.53 * 0.7023  0.45479E 01  0.32337E 00
  4988.58 * 0.9448  0.44817E 01  0.31882E 00
SEG  5  PTS  4 LENGTH(MLS)  0.65492
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   456.79 * 0.0865  0.42191E 01  0.31494E 00
  1370.09 * 0.2595  0.39953E 01  0.[illegible]1039E 00
  2283.49 * 0.4725  0.30122E 01  0.30527E 00
  3099.09 * 0.5869  0.30192E 01  0.30179E 00
SEG  6  PTS 10 LENGTH(MLS)  1.78447
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   272.02 * 0.0515  0.3[illegible]343E 01  0.2[illegible]E 00
   816.95 * 0.1546  0.3[illegible]E 01  0.2[illegible]E 00
  1360.09 * 0.2576  0.344[illegible]E 01  0.2[illegible]E 00
  2097.44 * 0.3972  0.3[illegible]2[illegible]E 01  0.28454E 00
  3029.19 * 0.5735  0.34709E 01  0.27640E 00
  3959.77 * 0.7495  0.3[illegible]509E 01  0.26935E 00
  4889.43 * 0.9260  0.31779E 01  0.26072E 00
  5820.09 * 1.1023  0.3[illegible]19[illegible]E 01  0.25232E 00
  6750.75 * 1.2786  0.31491E 01  0.24498E 00
  7262.98 * 1.3756  0.30090E 01  0.24172E 00
SEG  7  PTS 20 LENGTH(MLS)  2.47798
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   164.19 * 0.0311  0.27406E 01  0.22786E 00
   620.93 * 0.1176  0.26159E 01  0.22129E 00
  1123.67 * 0.2128  0.23920E 01  0.20799E 00
  1544.42 * 0.2925  0.22301E 01  0.19602E 00
  1965.17 * 0.3722  0.20597E 01  0.18295E 00
  2385.92 * 0.4519  0.18960E 01  0.17040E 00
  2806.67 * 0.5316  0.17437E 01  0.15848E 00
  3341.43 * 0.6328  0.16009E 01  0.14715E 00
  3990.18 * 0.7557  0.14597E 01  0.13547E 00
  4639.94 * 0.8786  0.13451E 01  0.12534E 00
  5287.69 * 1.0015  0.12454E 01  0.11723E 00
  5936.45 * 1.1243  0.11409E 01  0.10893E 00
  6585.20 * 1.2472  0.10605E 01  0.10201E 00
  7376.29 * 1.3970  0.98655E 00  0.94892E-01
  8309.71 * 1.5738  0.89821E 00  0.87356E-01
  9243.13 * 1.7506  0.82309E 00  0.80869E-01
 10176.55 * 1.9274  0.75972E 00  0.75257E-01
 11109.96 * 2.1042  0.6[illegible]974E 00  0.69772E-01
 12043.39 * 2.2809  0.64691E 00  0.64799E-01
 12794.49 * 2.4232  0.60737E 00  0.60979E-01
SEG  8  PTS  4 LENGTH(MLS)  2.52708
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
     3.66 * 0.0007  0.25499E 03  0.19763E 01
  3868.23 * 0.7326  0.27015E 01  0.24539E 00
 10144.79 * 1.9214  0.90903E 00  0.90726E-01
 12951.68 * 2.4530  0.5[illegible]963E 00  0.59944E-01
SEG  9  PTS 10 LENGTH(MLS)  0.88580
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   143.26 * 0.0271  0.55662E 00  0.56056E-01
   537.96 * 0.1019  0.54597E 00  0.54955E-01
  1040.23 * 0.1971  0.51372E 00  0.51573E-01
  1543.71 * 0.2924  0.477[illegible]3E 00  0.47854E-01
  2046.58 * 0.3876  0.44596E 00  0.44508E-01
  2549.45 * 0.4829  0.41361E 00  0.41092E-01
  3183.52 * 0.6029  0.3[illegible]7[illegible]3E 00  0.39343E-01
  3872.59 * 0.7334  0.37369E 00  0.36869E-01
  4345.09 * 0.8229  0.37029E 00  0.36509E-01
  4593.97 * 0.8701  0.36804E 00  0.36356E-01
```

FIG. 8.9.D
ZERO BOUNDARY CONDITIONS
(L.W.)

```
CONCENTRATIONS IN THE RIVER , AT END OF PHASE 4 OF C

SEG  1  PTS 14 LENGTH(MLS)  3.75227
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
     12.44 * 0.0024  0.15037E 00  0.19600E-01
   1675.88 * 0.3474  0.15036E 00  0.13730E-01
   3716.04 * 0.7038  0.11564E 01  0.95106E-01
   4493.23 * 0.8401  [illegible]  [illegible]
   5239.28 * 0.9923  0.6?745E 01  0.432??E 00
   6612.85 * 1.2524  0.8?41?E 01  [illegible]
   8603.04 * 1.6295  0.8?20?E 01  0.92294E 00
  10595.02 * 2.0066  0.7?8??E 01  [illegible]
  12586.11 * 2.3837  0.64726E 01  [illegible]
  14577.20 * 2.7608  0.50874E 01  [illegible]
  16562.30 * 3.1370  [illegible]  [illegible]
  17930.53 * 3.3974  [illegible]  [illegible]
  18687.39 * 3.5394  0.527??E 01  [illegible]
  19437.25 * 3.6813  0.524??E 01  [illegible]
SEG  2  PTS  8 LENGTH(MLS)  2.52443
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    761.05 * 0.1404  0.4?944E 01  0.4?321E 00
   2379.22 * [illegible]  [illegible]  [illegible]
   4170.45 * 0.7000  [illegible]  [illegible]
   [illegible] * [illegible]  [illegible]  [illegible]
   7754.01 * [illegible]  0.31277E 01  [illegible]
   [illegible] * 1.8??2  0.20???E 01  [illegible]
  11339.?? * 2.1474  [illegible]  [illegible]
  [illegible] * 2.4???  [illegible]  [illegible]
SEG  3  PTS [illegible] LENGTH(MLS)  [illegible]
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
   [illegible] * [illegible]  [illegible]  0.237??E 00
   [illegible] * 0.3477  [illegible]  [illegible]
   3024.37 * [illegible]  [illegible]  [illegible]
   [illegible] * [illegible]  [illegible]  [illegible]
   6234.44 * 1.1???  [illegible]  [illegible]
   7920.66 * 1.5016  0.9?437E 00  0.1?0??E 00
   9622.89 * 1.8225  0.85172E 00  0.10437E 00
  11317.12 * 2.1434  0.75773E 00  0.90045E-01
  13011.35 * 2.4643  0.65113E 00  0.79360E-01
  14397.21 * 2.7248  0.53672E 00  0.69602E-01
SEG  4  PTS  5 LENGTH(MLS)  1.02336
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    241.04 * 0.0456  0.55665E 00  0.64799E-01
   1168.00 * 0.2212  0.52820E 00  0.?534?E-01
   2536.23 * 0.4803  0.46863E 00  0.47482E-01
   3904.47 * 0.7395  0.36791E 00  0.37840E-01
   4997.20 * 0.9465  0.31306E 00  0.32374E-01
SEG  5  PTS  3 LENGTH(MLS)  0.65402
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
    667.09 * 0.1263  0.25707E 00  0.27370E-01
   2004.10 * 0.3796  0.20006E 00  0.19881E-01
   3066.01 * 0.5807  0.22442E 00  0.16753E-01

SEG  6  PTS  7 LENGTH(MLS)  1.30447
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CO
    215.00 * 0.0400  0.145??E 00  0.13960E-01
   1051.10 * 0.1991  0.13431E 00  0.12158E-01
   2209.60 * 0.4237  0.10652E 00  0.88452E-02
   3520.27 * 0.6672  0.90133E-01  0.62050E-02
   4766.36 * 0.9028  0.40577E-01  0.44195E-02
   6005.45 * 1.1374  0.74968E-01  0.30464E-02
   6967.34 * 1.3196  0.57417E-01  0.22402E-02
SEG  7  PTS 14 LENGTH(MLS)  2.47708
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CO
    243.34 * 0.0400  0.10142E-01  0.15012E-02
   1046.07 * 0.1973  0.87625E-02  0.10052E-02
   2087.45 * 0.3954  0.44606E-02  0.45045E-03
   3127.04 * 0.5924  0.24757E-02  0.32100E-03
   [illegible] * [illegible]  [illegible]  [illegible]
   [illegible] * [illegible]  [illegible]  [illegible]
   [illegible] * 1.1???  [illegible]  0.43670E-04
   7255.40 * 1.3742  0.64588E-04  [illegible]
   8287.54 * 1.5??2  0.25???E-04  0.38242E-05
   9420.47 * 1.7?27  0.1?474E-04  0.15417E-05
  10471.4? * 1.9??4  0.45706E-05  0.?734?E-06
  11143.35 * 2.1105  0.23050E-05  0.29496E-06
  12115.29 * 2.2946  0.1?551E-05  0.15490E-06
  12840.10 * 2.4318  0.85443E-06  0.12547E-06
SEG  8  PTS 24 LENGTH(MLS)  2.52708
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CO
      0.99 * 0.0002  0.33743E 01  0.36086E 00
    835.91 * 0.1583  0.34201E 01  0.36432E 00
   2191.62 * 0.4151  0.13490E 01  0.13942E 00
   3047.27 * 0.5771  0.55625E 00  0.67546E-01
   3663.98 * 0.6939  0.30715E 00  0.43995E-01
   4230.55 * 0.8012  0.11330E 00  0.16590E-01
   4793.81 * 0.9009  0.56202E-01  0.21032E-02
   5087.79 * 0.9626  0.24207E-01  0.34936E-02
   5461.77 * 1.0344  0.90222E-02  0.14157E-02
   5840.75 * 1.1062  0.30790E-02  0.55956E-03
   6219.73 * 1.1780  0.15517E-02  0.22411E-03
   6680.97 * 1.2653  0.61509E-03  0.89120E-04
   7224.46 * 1.3683  0.22600E-03  0.32773E-04
   7767.95 * 1.4712  0.10115E-03  0.14687E-04
   8311.43 * 1.5741  0.47070E-04  0.98446E-05
   8854.92 * 1.6771  0.23302E-04  0.33911E-05
   9398.41 * 1.7800  0.11102E-04  0.16500E-05
   9943.39 * 1.8832  0.56833E-05  0.82843E-06
  10480.84 * 1.9867  0.31903E-05  0.46671E-06
  11036.29 * 2.0902  0.19515E-05  0.28628E-06
  11582.74 * 2.1937  0.13121E-05  0.19167E-06
  12129.20 * 2.2972  0.91030E-06  0.13306E-06
  12675.65 * 2.4007  0.60232E-06  0.88096E-07
  13145.90 * 2.4898  0.46807E-06  0.68630E-07
SEG  9  PTS  9 LENGTH(MLS)  0.88500
DISTANCE          ..CONC SUBS 1..CONC SUBS 2..CONC SUBS
     24.23 * 0.0046  0.34555E-06  0.50655E-07
    378.00 * 0.0716  0.29923E-06  0.42546E-07
   1015.07 * 0.1922  0.23901E-06  0.35125E-07
   1629.79 * 0.3087  0.1?1?9E-06  0.23674E-07
   2244.51 * 0.4251  0.1005?E-06  0.14733E-07
   2859.23 * 0.5415  0.4?878E-07  0.68660E-08
   3473.95 * 0.6579  0.234?9E-07  0.34315E-08
   4088.67 * 0.7744  0.76540E-08  0.11221E-08
   4586.51 * 0.8682  0.29405E-08  0.43235E-09
```

FIG 8.9.E

ZERO BOUNDARY CONDITIONS (H.W.)

approach to handling pollutants has undoubted advantages, more development is needed here before radical projections can be accepted with confidence. Diffusion and dispersion are undoubtedly continuous parallel processes.To attempt to model them as discrete serial processes could meet with problems if relative time scales are large.

8.10 A River Model

It is essential that at least a part of each headwater stretch operates as a river so that the tide budget can be balanced rather than lose volume out of the top of the system. However, there is no requirement for each segment to contain tidal influences. Neither need the seaward boundary forcing function be tidal in nature. Sluices or piped input will be equally applicable. Consequently, the whole system can be used solely for modelling rivers.

8.11 Model Execution Timings

F2

As no iterative phase is involved, timings for a set geometry were very constant for fixed time steps. For a river step of 1 min and 3 min. for the bay , and graph plotting, required 2000 time units for 63 hours simulation real-time, or 32 time units per hour. Including drying out in the bay phases and allowing internal iteration for greater accuracy will require about 75 time units per hour. For long runs, restart facilities are available on the full of each hour of simulation time.

F1

The model has more options generating output and graphs, and so still requires 28 units per hour; a 140 hrs. simulation having taken 3900 units.

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Chapter 9

Applying the Stochastic Model

Contents

Chapter 9

9.0 Introduction

The Stochastic Model is the most expensive model of the three int erms of running costs for the mainframe computer and in data requirements for the latter phases of simulation.

Two versions are available. Version ST is completely self-contained, and ST2 can use intermediate files generated by the models F1 and F2 via software interfaces and a small run time parameter file.

The first step in the model is to compute the mean values of B.OD. and O.D. (oxygen deficit) at various times throughout the cycle for the length of the system. Again, depending on the accuracy of t he initial conditions supplied by the user, the time taken to reach numerical steady state can vary from 400 computer time units to 2,600. Fig. 9.0.A shows initial conditions found suitable for a wide range of inputs to the Usk.

A minimum time of 20-30 hours real-time simulation was required to start considering output to be independant of start up effects. Because of the rapidity of tidal motion, the time step varies dramatically and is at times very much lower than for other sestuaries. Fig. 9.0.B illustrates time steps required in the Usk and Delaware Estuaries[1].

Much of the problem was caused by the tidelocking of the majority of the discharges in Newport. Fig. 9.0.C illustrates the times of discharge of the various outfalls in the tidal cycle. The Runge-Kutta-Merson Integration assumes a continuous function, which is now not the case as the function is only piecewise continuous. This could well threaten the stability of the system.

Furthermore, after prolonged closure a reservoir effect produced a shock load on opening so accentuating the discontinuity. Using this method, time steps changed so rapidly that a two tidal cycle simulation required 6,000 time units.

To moderate the problem, a 'leek' rate is computed instead of complete closure. This ensures that the shock load is moderated and the discontinuity is vaguely continuous. This has a stabilizing effect on the time step and reduces the time for a 40 hour simulation to 3,000 time units. This means that a simulation is not cost prohibitive.

To make the model more cost effective, several actions were taken:

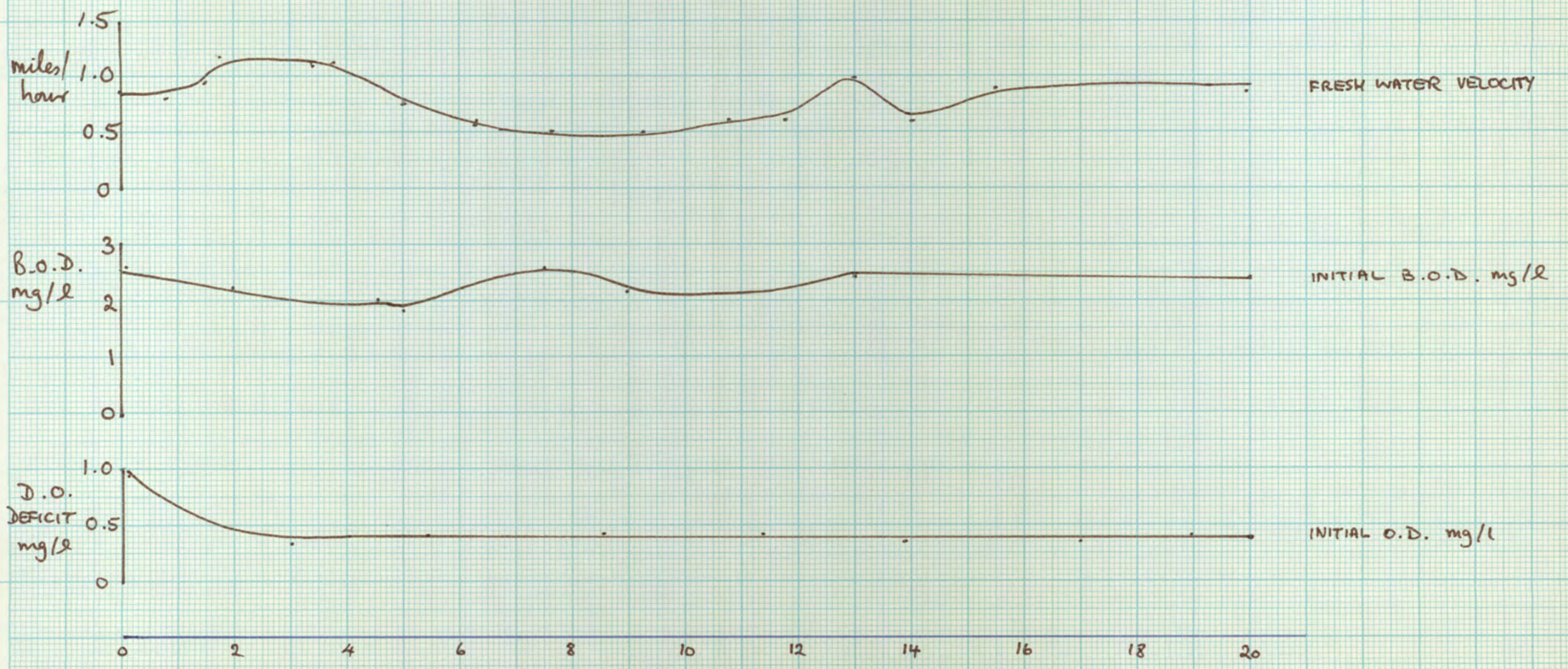
a) After investigation of time spent by the program it was found that the time consuming part of the integration was the array access for the variables. Many of the arrays are single valued for all practical purposes and an amended routine INPUT senses if a batch of these arrays are single valued. If so, a switch is set to enable function evaluations to be made using global constants, thus making a function evaluation up to 12% more efficient.

b) The derivations calculated in the first phase are of the first and second order, and computed up to sixth order accuracy. Alternatives for second order accuracy were incorporated. Use of this in the start-up period assisted stability. The second order accuracy derivatives may be used throughout for a maximum $\pm$ 0.02 deviation on O.D./B.O.D. projections. The overall program speed increased by 3%.

c) A second order Runge Kutta method was incorporated in place of the fourth order method. As less functional evaluations were required, program speed increased by up to 28%. However, as no inbuilt error estimate is available, the time step is fixed and can easily generate cumulative errors. Reducing the time step to the minimum used by the fourth order method increases core time by more than 40%. This facility should only be used for short duration simulations (less than one tidal cycle) and only where a good starting distribution is available.

Despite all the above enhancements, the most efficient simulation still required 56 time units per hourof real time simulation. Although 76% less costlt than the original version it was still considered too expensive for everyday use. In any event, it was finally available only after termination of the project.

Fig 9.0.A STARTING POINTS OF MODELS ST/STR



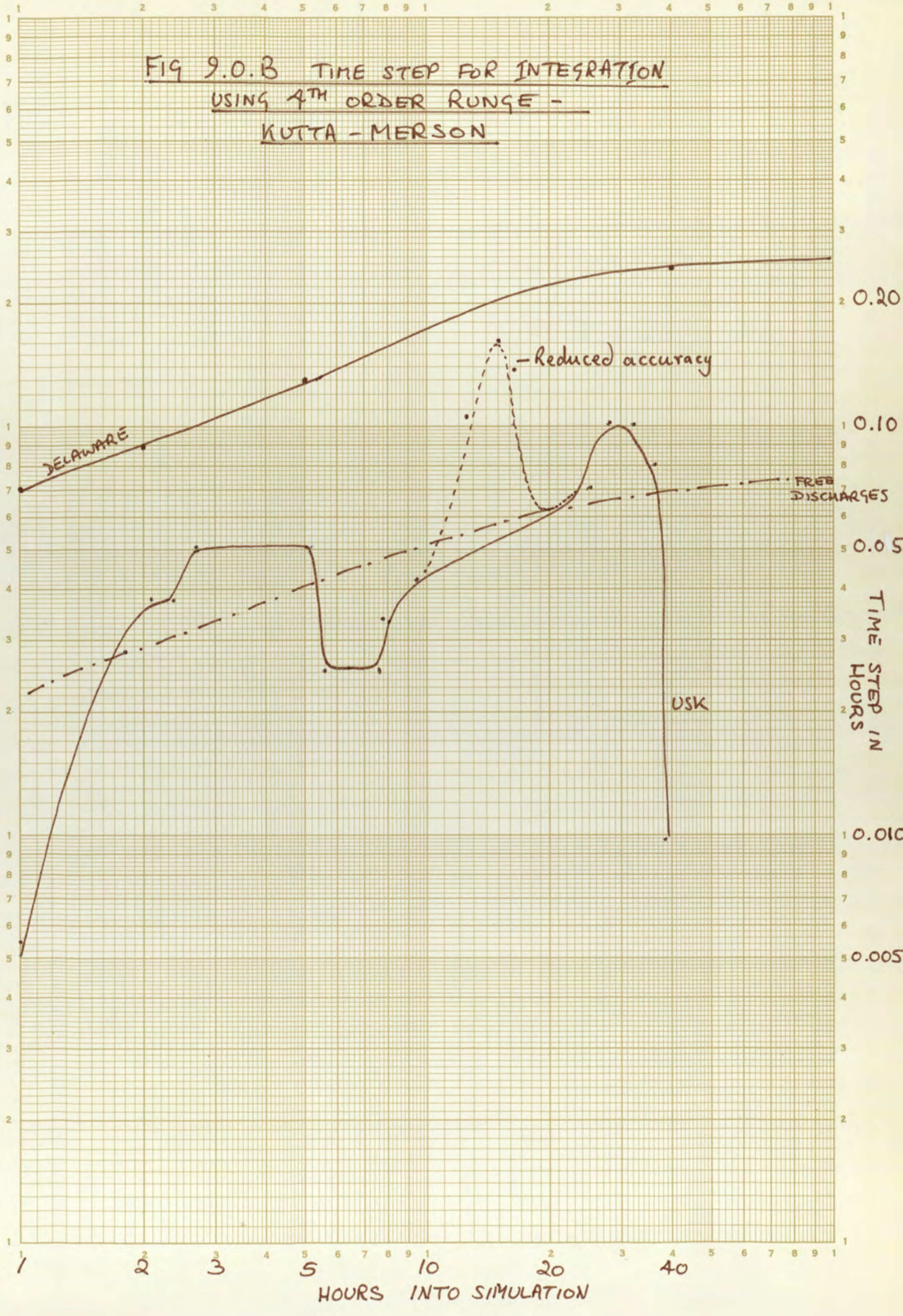
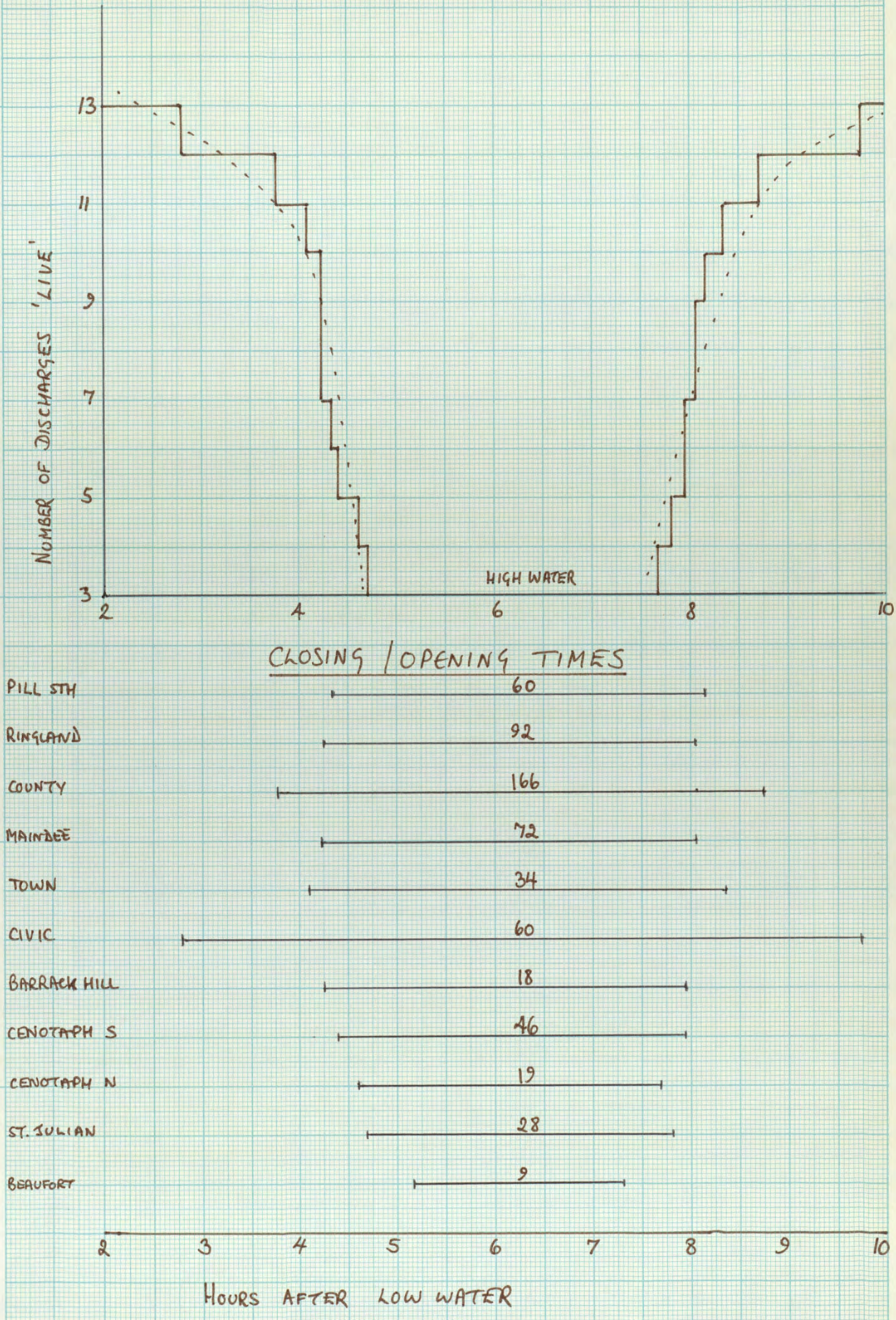


FIG 9.0.B TIME STEP FOR INTEGRATION USING 4TH ORDER RUNGE - KUTTA - MERSON
DELAWARE
-Reduced accuracy
FREE DISCHARGES
USK
0.20
0.10
0.05
0.010
0.005
TIME STEP IN HOURS
1
2
3
5
10
20
40
HOURS INTO SIMULATION

FIG 9.0.C TIDELOCKED DISCHARGES



9.1 Input Parameters

Again the policy of using best estimates of parameters is adopted. As most parameters are estimated, there is little point in trying to tune the model to one or several of the parameters.

There is some flexibility on the B.O.D./O.D.(oxygen deficit) grid selected. They need not coincide. However, the B.O.D. spacings must be an integer multiple of the O.D. spacing. At most, three times as many B.O.D. points are required as O.D. points. Usually grids coincide for convenience. In the Usk, 150 points were used for both grids.

Up to 20 points can be selected for printing. For ease of eventual data interpretation, full date-time handling facilities are included.

The initial time step is merely a convenience in the full model, as the optimum step is soon reached. If the second order method is to be used, then it is very important as the step is used throughout simulation.

Other initial conditions have been discussed in 9.0.

Other parameters were from reviewed literature [2][3][4]:

Oxygen Deficit Diffusion Coefficient	= 0.1 sq. mile/hour
B.O.D. Diffusion Coefficient	= 0.1 sq. mile/hr.
B.O.D. Decay Rate	= 0.01 per hour (Chapter 6)
Re-aeration	= 0.0035 lbs per hour
Rate Constant for decrease of B.O.D by other mechanisms	= 0.001 (K_d)
Land Runoff , nominally	= 1000 lbs per hour per cubic mile

(ie. at 5% of observed value[1], almost negligable)

The solution was not found to be sensitive to Land Runoff values less than 50,000 lbs per hour per cubic mile. Benthal Demand was not covered by basic data available.

Outfall input data required opening and closing times for tidelocking calculation, position and point averaged cross sectional areas in square miles and mean loadings.

9.2 Software Acceptance Trials

Sufficient information was available in the literature to simulate the Potomac Estuary. Input and output matched those of reference [1] (fig. 9.2.A and B) This gave a useful validation source after many versions of the original were developed to incorporate previously mentioned enhancements, as well as graphical output.

Fig 9.2.A

Input for Potomac Estuary Run

TIDAL VELOCITY

0.0000E 00	0.9766E-01	0.1855E 00	0.2637E 00	0.3320E 00	0.3868E 00	0.3926E 00	0.3983E 00	0.4040E 00
0.4097E 00	0.4153E 00	0.4208E 00	0.4263E 00	0.4317E 00	0.4371E 00	0.4421E 00	0.4429E 00	0.4450E 00
0.4484E 00	0.4531E 00	0.4591E 00	0.4665E 00	0.4752E 00	0.4826E 00	0.4887E 00	0.4985E 00	0.5120E 00
0.5292E 00	0.5501E 00	0.5785E 00	0.6183E 00	0.6571E 00	0.6948E 00	0.7314E 00	0.7670E 00	0.8016E 00
0.8397E 00	0.8758E 00	0.9095E 00	0.9407E 00	0.9695E 00	0.9957E 00	0.1019E 01	0.1041E 01	0.1052E 01
0.1063E 01	0.1072E 01	0.1081E 01	0.1090E 01	0.1098E 01	0.1105E 01	0.1112E 01	0.1118E 01	0.1121E 01
0.1124E 01	0.1126E 01	0.1129E 01	0.1131E 01	0.1134E 01	0.1137E 01	0.1139E 01	0.1142E 01	0.1144E 01
0.1147E 01	0.1149E 01	0.1152E 01	0.1155E 01	0.1157E 01	0.1160E 01	0.1162E 01	0.1165E 01	0.1167E 01
0.1170E 01	0.1173E 01	0.1176E 01	0.1179E 01	0.1181E 01	0.1184E 01	0.1186E 01	0.1188E 01	0.1190E 01
0.1191E 01	0.1193E 01	0.1194E 01	0.1195E 01	0.1197E 01	0.1198E 01	0.1199E 01	0.1199E 01	0.1200E 01
0.1200E 01	0.1200E 01	0.1200E 01	0.1200E 01	0.1200E 01	0.1200E 01	0.1200E 01	0.1200E 01	0.1200E 01
0.1200E 01	0.1200E 01	0.1200E 01	0.1197E 01	0.1193E 01	0.1189E 01	0.1184E 01	0.1179E 01	0.1173E 01
0.1167E 01	0.1161E 01	0.1154E 01	0.1146E 01	0.1138E 01	0.1130E 01	0.1121E 01	0.1112E 01	0.1096E 01
0.1081E 01	0.1067E 01	0.1053E 01	0.1040E 01					

FRESH WATER VELOCITY

0.5070E 00	0.4457E 00	0.3888E 00	0.3362E 00	0.2881E 00	0.2444E 00	0.2050E 00	0.1701E 00	0.1395E 00
0.1134E 00	0.9160E-01	0.7907E-01	0.8316E-01	0.8670E-01	0.8966E-01	0.9206E-01	0.9390E-01	0.9517E-01
0.9588E-01	0.9602E-01	0.9416E-01	0.9205E-01	0.8990E-01	0.8772E-01	0.8550E-01	0.8326E-01	0.8098E-01
0.7775E-01	0.7481E-01	0.7232E-01	0.7027E-01	0.6867E-01	0.6923E-01	0.7055E-01	0.7154E-01	0.7218E-01
0.7249E-01	0.7245E-01	0.7208E-01	0.7099E-01	0.6792E-01	0.6507E-01	0.6245E-01	0.6006E-01	0.5790E-01
0.5596E-01	0.5426E-01	0.5277E-01	0.5200E-01	0.5151E-01	0.5112E-01	0.5081E-01	0.5059E-01	0.5046E-01
0.5042E-01	0.5047E-01	0.5061E-01	0.5083E-01	0.5175E-01	0.5329E-01	0.5459E-01	0.5564E-01	0.5646E-01
0.5703E-01	0.5736E-01	0.5745E-01	0.5730E-01	0.5584E-01	0.5392E-01	0.5211E-01	0.5041E-01	0.4881E-01
0.4731E-01	0.4592E-01	0.4464E-01	0.4346E-01	0.4238E-01	0.4189E-01	0.4140E-01	0.4089E-01	0.4033E-01
0.3973E-01	0.3910E-01	0.3844E-01	0.3773E-01	0.3678E-01	0.3576E-01	0.3477E-01	0.3382E-01	0.3291E-01
0.3204E-01	0.3120E-01	0.3040E-01	0.2964E-01	0.2904E-01	0.2852E-01	0.2801E-01	0.2752E-01	0.2703E-01
0.2656E-01	0.2610E-01	0.2565E-01	0.2522E-01	0.2479E-01	0.2438E-01	0.2398E-01	0.2359E-01	0.2321E-01
0.2285E-01	0.2249E-01	0.2215E-01	0.2182E-01	0.2150E-01	0.2120E-01	0.2090E-01	0.2062E-01	0.2035E-01
0.2009E-01	0.1984E-01	0.1961E-01	0.1939E-01					

F FEED RATES – Point Discharges

0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.1507E 07	0.0000E 00	0.0000E 00	0.0000E 00	0.1120E 08	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.7280E 06	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.3700E 05	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.1800E 05	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00
0.0000E 00	0.0000E 00	0.0000E 00	0.0000E 00					

FIG 9.2.B OUTPUT FOR POTOMAC – MATCHING [1]

TIME, HRS 13.0000 DELTA T,HRS 0.1756 TIDE 1.0385

STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00
G--PROFILE OXYGEN DEFICIT	1.000	0.445	0.278	0.453	0.683	0.690	0.678	0.572	0.507	0.427	0.376	0.333	0.308	0.272	0.235	0.209
F--PROFILE B.O.D DEFICIT	3.600	0.668	0.297	1.077	2.044	2.427	3.810	0.918	0.551	0.186	0.078	0.044	0.034	0.032	0.033	0.031
STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00

TIME, HRS 14.0000 DELTA T,HRS 0.1756 TIDE 1.1192

STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00
G--PROFILE	1.000	0.638	0.291	0.403	0.717	0.752	0.770	0.743	0.642	0.530	0.462	0.404	0.361	0.316	0.272	0.238
F--PROFILE	3.600	1.035	0.296	0.714	1.902	2.242	3.541	3.141	1.371	0.504	0.199	0.082	0.047	0.034	0.039	0.032
STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00

TIME, HRS 15.0000 DELTA T,HRS 0.1756 TIDE 1.1998

STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00
G--PROFILE	1.000	0.934	0.370	0.345	0.657	0.760	0.808	0.845	0.856	0.738	0.613	0.528	0.461	0.379	0.324	0.281
F--PROFILE	3.600	1.754	0.435	0.418	1.468	1.949	2.885	3.406	3.535	2.022	0.789	0.349	0.142	0.046	0.038	0.040
STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00

TIME, HRS 16.0000 DELTA T,HRS 0.1756 TIDE 1.2805

STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00
G--PROFILE	1.000	1.158	0.578	0.368	0.530	0.669	0.779	0.853	0.891	0.929	0.887	0.751	0.624	0.484	0.387	0.335
F--PROFILE	3.600	2.524	0.826	0.395	0.888	1.447	2.476	2.835	3.047	3.424	2.979	1.633	0.686	0.151	0.046	0.040
STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00

TIME, HRS 17.0000 DELTA T,HRS 0.1756 TIDE 1.3611

STATION POSITIONS	0.00	3.00	4.80	6.00	8.00	9.00	10.00	11.00	11.80	13.00	14.00	15.00	16.00	17.80	20.00	22.00

9.3 <u>The Model in the Usk</u>

As the first phase aims to predict the mean values over multiple tides for a point in time and space, and accepting that the Steady State Model is reasonably sound, predictions from one may be used on the current model.

Figure 9.3. A shows the scatter of predictions about a mean Steady State value generated from the Steady State Model. There seems to be reasonable agreement, although the modelled situation is not very severe.

In the absence of data to see the process through to its ultimate conclusion, i.e. probabilistic limits on the means predictions, little further effort was possible.

The complete basic data set is shown in fig. 9.3.B.

Introducing a small benthal demand (500 ibs per hour per cubic mile) had no effect on the O.D. and only a very slight effect on the B.O.D.

Fig. 9.3.C shows the projected effect of doubling the Eastern Valley loadings. The mean effect agrees well with that projected by the Steady State Model. Note that there is no net improvement in the predictions as they reach the two dimensional phase boundary, as the boundary is open and not necessarily constrained to be a constant. If this is required, the one dimensional model has to be extended out into the two dimensional phase until equilibrium is reached. This then has no physical interpretation, however.

Halving diffusion of the oxygen deficit affects predictions by at most 4%-5% in the section below the Eastern Valley discharge to the open boundary. Reducing both diffusion coefficients to zero affects the efficiency of solution as the inputs remain integral and so the function is extremely irregular.

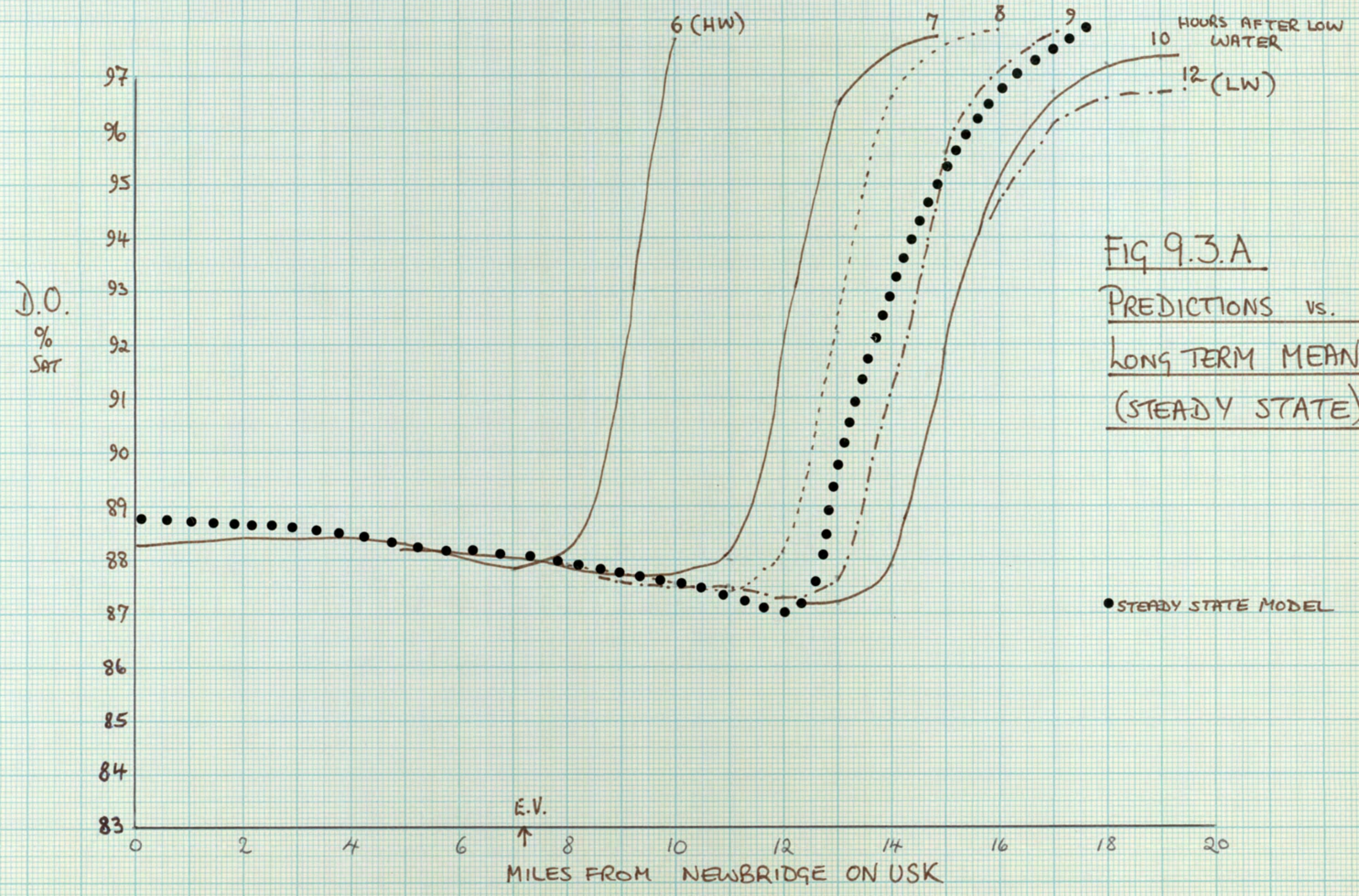


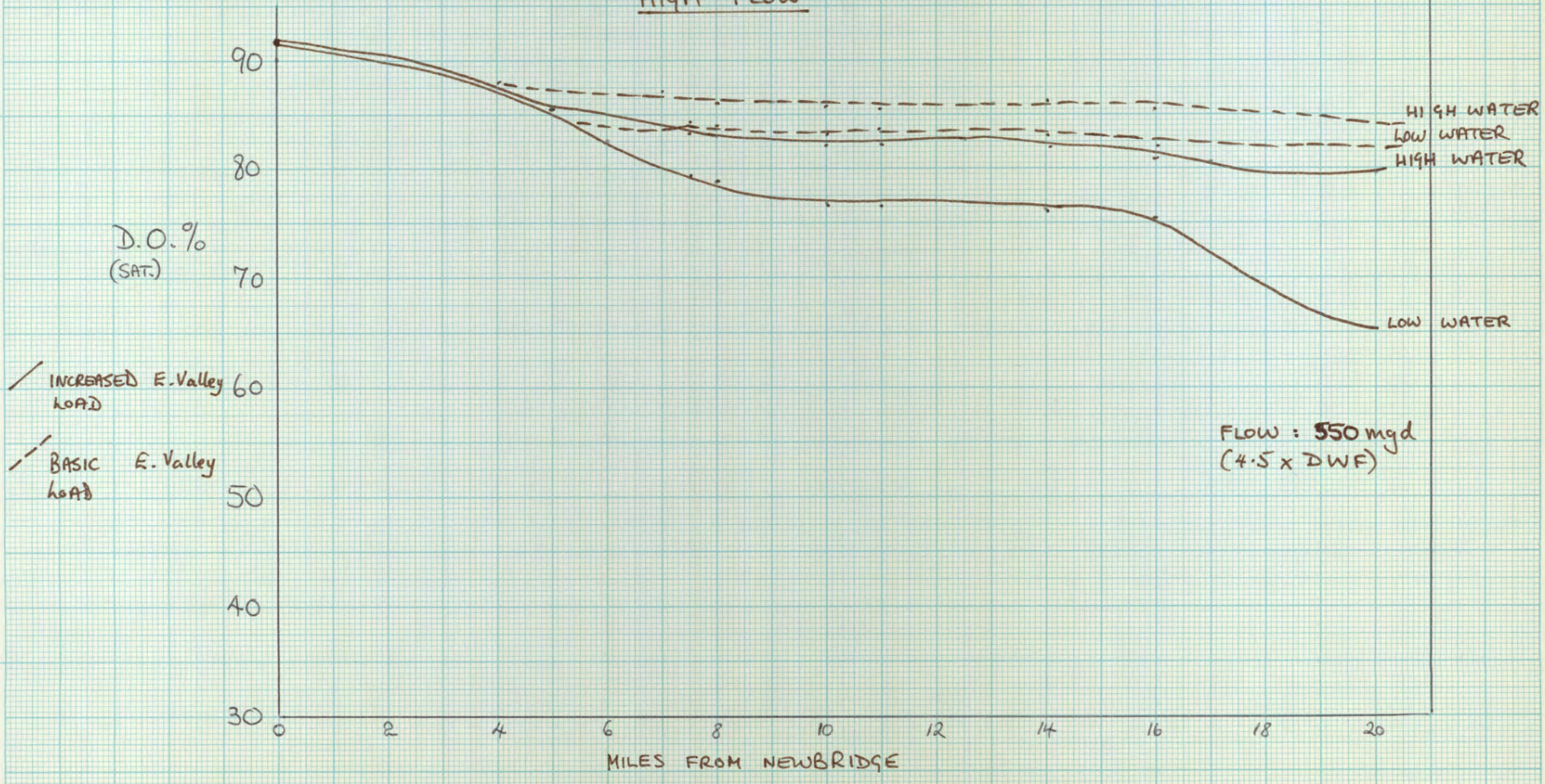
FIG 9.3.A
PREDICTIONS vs. LONG TERM MEAN (STEADY STATE)

FIG 9.3.B

COMPLETE INPUT SET FOR USK ESTUARY

```
      150  NG B.O.D GRID                         150   NF O.DGRID POINTS
       17  NS PRINT GRID            75 8 19 10 20   DATE
       0.  START POINT                           20.   END POINT
      1.5  START TIME                           300.   END TIMR
      0.3  PRINT STEP                            .05   PRINT TOLERANCE
      20.  AV TEMPERATURE                         0.   TSTART TEMP DATA
     .262  FEED FREQUENCY                       2.50   UPSTREAM B.O.D.
       1.  UPSTREAM O.D.                       1.571   REAFRATION CORRECTION
       1.  FEED CONSTANT                         .05  FEED FACTOR
100.       TIME FOR STEADY STATE OUTPUT
       06    IPLOT
        6  ACCURACY FACTOR
     12.4 TIDAL DURATION
0.0005           RELATIVE ERROR
0.0005           ACCURACY
20.              ABSOLUTE ERROR
20. 1.     NEW TIME STEP AFTER XHRS
31           PRINTING STYLES
 00        GRID POINT TO FOLLOW
4            ORDER OF RUNGE KUTTA METHOD USED
0.009            INITIAL TIME STEP
5.  7 8 9 10 10.5 11 11.5 12. 12.5 13 14 15 16 17 18 19                          XS GRID
06           INITIAL OD PROFILE                                   0.0
 1. 1. 5. .04 7. .04 14 .04 15 .04 20 .045
09           INITIAL BOD PROFILR                                  2.0
0. 2.5 5. 1.99 7 2.34 8 2.31 9 2.18 11 2.17 12 2.23
 15 2.02 20 1.993
16           FRESH WATER FLOW ( MILES PER HOUR )
0. .846 1.55 .907 3.75 1.132 5. .777 6.32 .545 7.74 .498 9.15 .5
10.17 .566 10.82 .6 11.51 .566 12.2 .477 13. .743 13.85 .62
14.68 .641
15.56 .825 20. .825
17           TIDAL VELOCITIES
0. 0. 3.75 0 5. 0 6.32 .91 7.74 2.2 9.15 2.5 10.17 2.54 10.82 2.4 11.5
2.44 12.2 2.1 12.7 1.8 13.2 1.92 13.7 2. 14.18 2.11 14.7 1.65 15.6
1.66 20.     1.66
-2           OXYGEN DEF.DIFFUSUION COEFFICIENT                     .1
-2           B.O.D.DIFF.COEFFS                                     .1
-2           BOD DECAY RATE CONSTA PER HOUR                        .01
-2           REAAERARTION RATES                                    .0035
-2           BOD (KD) PROCESSES                                    .001
-2           LANRUNOFF                                          1000.
-2           BENTHAL DEMAND                                        0.0
14
E.VALLEY F 6.25 6.25 5.85 187.5 0. .00008
BEAUFORT T 5.2 7.3 7.32 7.5 0.5 .000110
CAERLEON F 6.25 6.25 7.64 83.3 0. .000120
ST.JULIA T 4.7 7.8 8.15 21.25 1. .000135
BRYNBETT F 6.25 6.25 9.57 91.7 0. .000160
CENOTAPN T 4.6 7.7 9.68 14.6 1. .00017
CENOTAPS T 4.4 7.95 9.94 33.3 1.5 .000175
BARRACKH T 4.25 7.95 10.01 12.5 1. .000177
CIVIC&   T 2.8 9.75 10.21 26.3 1. .00018
TOWN&    T 4.1 8.35 10.41 22.5 1. .000188
MAINDEE  T  4.25 8.05 10.78 50. 1.5 .00021
COUNTY75 T 3.8 3.75 11.29 100. 2.5 .000245
RINGLAND T  4.25 8.05 11.44 63.8 2. .00026
PILL STH T 4.35 8.15 12.09 141.8 1.5 .00036
```

FIG 9.3.C DOUBLED EASTERN VALLEY - PREDICTED MEANS FOR HIGH FLOW



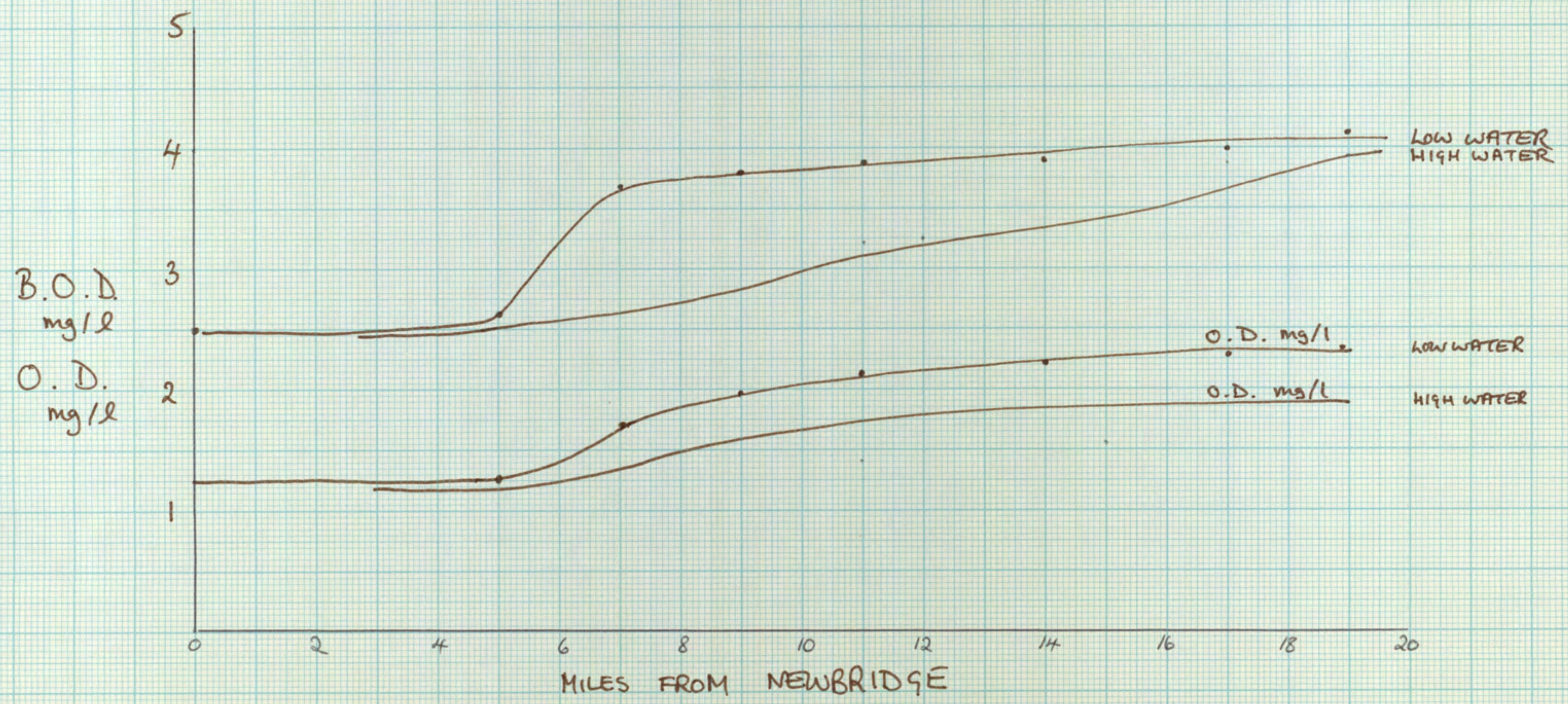
Whereas with halved diffusion, only 39% extra time is needed, now an extra 603% are required to simulate the same time span. A 25% increase in B.O.D. is observed and a 12%-20% worsening in the Dissolved Oxygen profile at low water. Altering the upstream boundary condition reflects in a net movement of the whole profile due to the high flows for which the data is available.

Reducing the velocities gives a general build up of B.O.D. (fig.9.3.D) some of which remains to be converted to an oxygen deficit.

However, the whole system remained insensitive to changes in reaeration. When halved, only a 3% effect on the oxygen deficit was predicted. This is possibly due to the high flow levels, giving little momentum to the process at such low levels of deficit.

More validation data and better input velocities are required to make this model useful.

FIG 9.3.D. REDUCED F.W.F. VELOCITIES



9.4 <u>The Stochastic Coefficient</u>

As discussed in Chapter 5, the stochastic coefficient needs to be established as accurately as possible via comprehensive selected data.

Table 9.4.C shows estimates of the coefficient Δ using results from just 2 surveys(Table 9.4.A and B). Most values are reasonably well grouped apart from two in excess of unity and one low value.

Examination of the data record shows that the high estimates all contained values of D.O. within a zone of reaction that the model is not designed for. All readings below 1mg/l D.O. should be discarded. Recalculation shows the new Δ values are more reasonable. However, calculations include all data collected and other selections have not been carried out. Consequently values are likely to be overestimated. It is not unusual to find an order of magnitude difference for Δ within an estuary[1][4].

The value at the seaward boundary is expected to be low. Theoretically, the 'sea' is tha constant boundary condition and so should theoretically have a variance of zero. Practically, some variance does exist of course, but as net values are usually numerically high (in excess of 8mg/l) the ratio defining Δ tends to reduce this. Similarly, the upstream values should decrease as the point of calculation moves to the upstream boundary, which is theoretically a constant for the purposes of the model. In between the boundaries the coefficient will rise to maximum in the area of greatest mean oxygen deficit as the variance will maximize at a point with also the depressed mean to increase their mutual ratio. Fig. 9.4.A illustrates that the trend is already apparent even with very small amounts of noisy data. (fig. 9.4.B).

Table 9.4.A D.O. Summary Statistics

Distance of Survey Point (km.)	Neap Samples	Tides Mean mg/l	SD	Spring Samples	Tides Mean mg/l	SD
0.1	20	7.36	0.47	24	8.47	1.94
4.3	25	6.51	2.57	26	8.15	0.72
8.0	27	3.92	2.31	25	6.00	2.45
10.0	26	3.34	1.55	23	4.37	1.53
13.0	26	3.37	1.54	25	4.12	1.14
16.0	25	2.82	1.40	25	3.76	1.52
18.5	28	4.00	2.27	26	4.30	1.87
21.0	25	4.44	2.13	27	5.96	1.15
23.5	26	5.22	1.69	23	5.85	0.65
27.0	18	5.97	0.77	25	6.26	0.40

Combined Summary

	Samples	Mean	SD	95% Confidence Limits	
0.1	44	7.96	1.55	4.85	11.08
4.3	51	7.35	2.03	3.28	11.41
8.0	52	4.91	2.58	0	10.08
10.0	49	3.82	1.61	0.6	7.05
13.0	51	3.74	1.40	0.94	6.54
16.0	50	3.29	1.52	0.24	6.34
18.5	54	4.15	2.08	0	8.30
21.0	52	5.29	1.85	1.53	8.93
23.5	49	5.51	1.28	2.96	8.06
27.0	43	6.15	0.69	4.95	7.34

(SD - Std. Deviation, assuming normal distribution)

Table 9.4.B B.O.D.[1] Summary Statistics

Distance of Survey Point	Neap Samples	Tides Mean	SD	Spring Samples	Tides Mean	SD
0.5	7	2.09	0.16	-	-	-
4.3	7	2.37	0.72	7	2.37	1.47
8.0	7	2.49	0.75	8	1.95	1.02
10.0	4	2.25	0.65	5	1.7	0.73
13.0	5	3.74	1.76	5	2.5	1.25
16.0	6	3.25	2.00	6	1.87	1.70
18.5	8	2.21	1.56	5	1.78	1.01
21.0	5	2.06	1.4	7	1.93	1.37
23.5	7	0.73	.35*	6	1.28	0.85
	5	0.92	.15*			
27.0	12	0.81	.63*	3	0.6	0.1
	7	1.21	.53*			

* includes estimates for inexact B.O.D. measurements on samples

(1) 5-day standard non inhibited B.O.D. Test at 20°C.

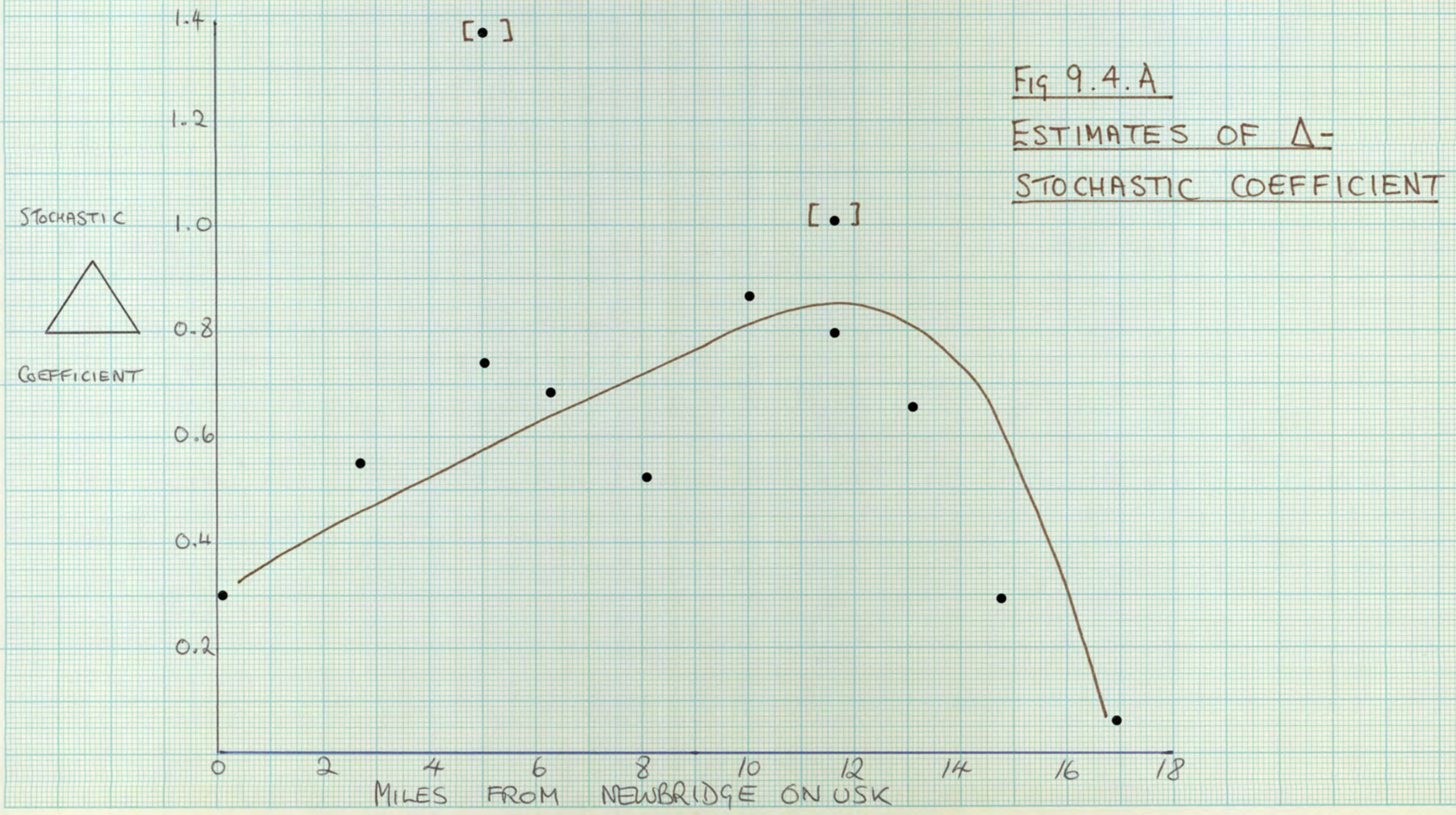


Fig 9.4.A
ESTIMATES OF Δ-
STOCHASTIC COEFFICIENT
STOCHASTIC
Δ
COEFFICIENT
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
2
4
6
8
10
12
14
16
18
MILES FROM NEWBRIDGE ON USK

FIG 9.4.B D.O. CONFIDENCE LIMITS

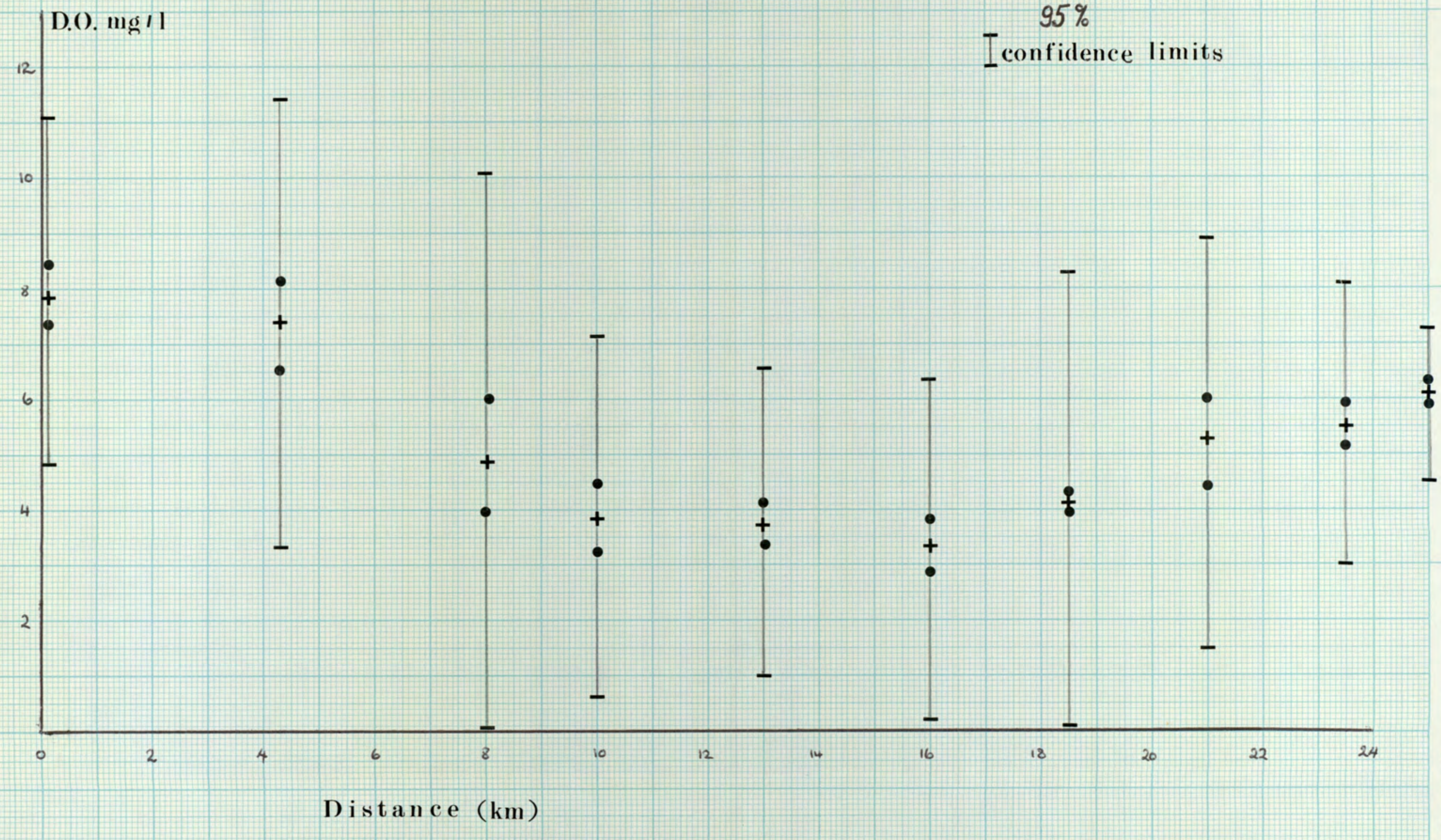


FIG 9.4.C B.O.D. CONFIDENCE LIMITS

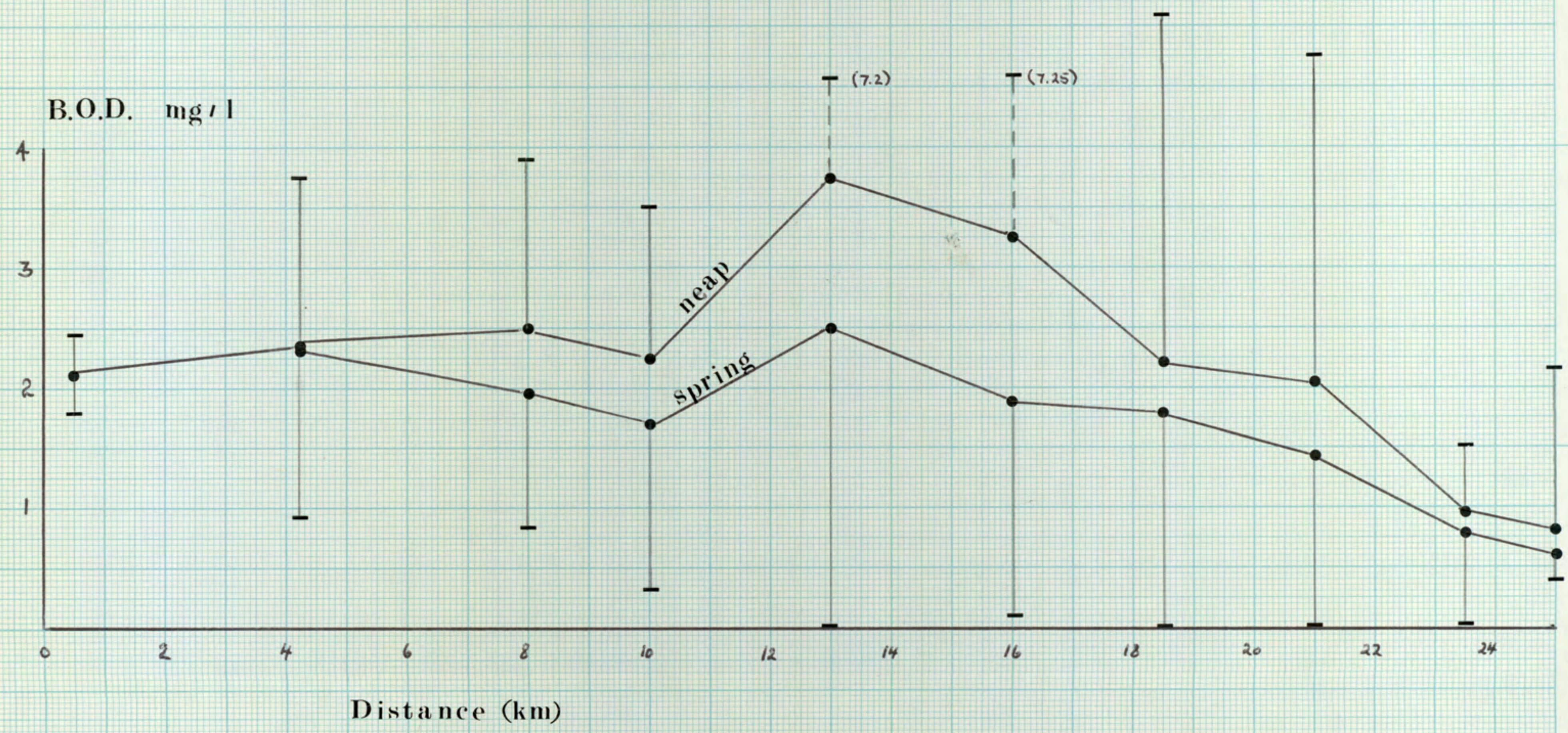


Table 9.4.C Estimates of the Stochastic Coefficient

Point from Newbridge	Samples Quantity	Mean	Variance	Stochastic Coefficient
0.0	44	8.0	2.4	0.30
7.7	51	7.4	4.1	0.55
5.0	52	4.9	6.7	1.37 (0.74)
6.3	49	3.8	2.6	0.68
8.1	51	3.7	2.0	0.53
10.0	50	3.3	2.3	0.86
11.6	54	4.2	4.3	1.02 (0.79)
13.1	52	5.3	3.4	0.64
14.7	49	5.5	1.6	0.29
16.9	43	6.2	0.4	0.06

9.5 A River Model

At no stage in the model development in Chapter 5 was the essential requirement that the system be an estuary. Naturally the development assumed this and so the algorithm is geared for this application.However, the model can be simply used for one dimensional river stretches simulations by switching out the tidal velocity component. There is also a reduced effort calculating Δ although it tends to be subject to more severe interferences[4].

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Chapter 10

Further Considerations

Chapter 10

Contents

10.0 Introduction

There are 105 estuaries in the U.K.[3] (water bodies 5km long with discernable tidal actions) of which only about one half had an analytical effort on them (B.O.D., Ammonia, pH). More data is always useful where little is available, but there is little advantage in random gathering. Estuary data is expensive to acquire, either through telemetry or more time honoured methods. A policy should exist for this work.

Three questions need to be answered:

a) What data is being collected, and why

b) How much data is required to fulfil functions

c) Effective gathering methods and related accuracy.

10.1 Data Collection for Models

This should act as a building block towards some higher goal, e.g. a local or regional model.

Data as a collection of figures to satisfy political demands is wasteful.

URD Management now have three models available. None have been calibrated extensively. Each has different strong points and weaknesses. A decision should be made to settle these questions:

a) Which models are to be used by management, if any ?

b) Which are to be used by numerate staff ?

c) Which are to be validated ?

d) Are any to be cross-calibrated ?

e) Are the questions to be asked suitable for any or one of the models ?

f) Who will translate questions into input and generate output ?

g) How good is the data applied to each model ?

Many of these points were not considered when the project was formulated.

A point to be noted is that neither instruments nor humans are infallible. Reliance on either in isolation will be less effective than a combination of both resources.

10.2 Cost Effectiveness of the Research Programme

The effectiveness of the research undertaken is best measured against proposed costs for alternative methods.

The cost of 31 years computing and ancillary expenses of the study was £18,000 at 1974 prices. This was low because the project was supported financially by SRC, Aston University and Cardiff University (Computing charges about 20% of bureau costs).

The tidal prism of the Usk is large compared to other physical dimensions. The maximum inflow rate is estimated at an equivalent 40,000 m.g.d. (2,00 cumecs) relative to a dry weather flow of 83 m.g.d.[1].

This large volume is difficult to trace and measure.A 1967 cost study gave the following cost estimates:

A)	One Radio-active tracing experiment, carried out and completed by Water Research Centre.	£35,000
B)	Two retention time surveys plus one dilution survey using students and U.R.Authority boats,based on bacteriological methods.	£ 4,900
C)	A network of 6-8 monitoring stations to record certain physical and chemical parameters.Capital costs only (3 estimates)	£13,000- £29,000

(All above estimates are 1967 prices)

All three field studies generate data but do not provide any sort of management tool for long term use.The afvantage of a mathematical model is that, after completion of the initial work, it can , under certain circumstances, stand alone without technical support.

Compared to the cost of even small capital projects(Chapter1/2) models are very cost effective even if used lightly.

10.3 Future Data Aquisition

Discussed elsewhere in this submission, the following broad recommendations are made:

a) If the full network of telemetry cannot be established, or anything approaching a reasonable coverage, the two current stations should be dismantled.

b) About 10 saturation surveys should be carried out in the Usk estuary to determine reasonably cohesive parameter relationships.

c) Regular estuary surveys should be geared towards being part of a long-term program for data acquisition for modelling.

d) The reaeration rate experiment should be repeated several times for differing conditions.

e) One large-scale tracer study is essential to acquire some idea of travel times, lateral and vertical mixing, and most important, to estimate the magnitude of D_x. Either dye-tracing (Rhodamine WT) or radioactive tracer (^{85}Br) would be suitable. Alternatively several short-term experiments are required.

10.4 Model Development

10.4.1 The Steady State Model

The following areas are of interest for further work:

a) The applicability of the definition of the mixing exchange F as defined in section 7.0.

b) Associated with a), the comparison of D_x values obtained using e.g. 7.0.A and field measurement.

c) Refining the restricted oxidation of ammonia path in the interactive iterative phase. This is not necessary for the vast majority of situations in the Usk , as it rarely reaches such low levels in simulations unless the now closed British Glue Co. is included (10,000 lbs BOD per day). However, it is very relevant in other Estuaries in the Region.

d) Data acquisition to enable validation of estimates regarding input loadings for tidelocked discharges, and also on the breakdown of loads into fast and slow components.

e) Data acquisition to enable some validation of the carbonaeceous and nitrogenous components, as well as ammonia and nitrate. A better established baseline should improve optimisation of natural purification powers for these components.

f) Incorporating the model into a cost optimizing routine for use by resource planners and operations for financial projections of likely effective schemes. This will involve the relative new filed of shadow cost benefit analysis and allocation of social cost benefits. This can be a highly subjective area with possibility of public involvement.

g) Compounding the various predicted distributions into a B.O.D. estimate, or writing a simple BOD/DO version of the model using compounded loads.

h) Investigating the possibility of replacing the deterministic BOD consent standard usually employed by either a composite standard or by a more reproducible standard such as Total Oxidizable Carbon, which appears to be closely correlated with BOD and more easily determined.

10.4.2 The Models F1/F2

The following areas are of interest for further work :

a) Establishing wind stress rpobability distribution for the Severn estuary in view of the influence of wind on elevations.

b) Extending the two dimensional phase to allow drying out areas via Moving Boundaries Method.

c) Incorporating self-stopping method when 'steady' or oscillating phase for a tidal cycle is reached.

d) Incorporating variable mesh spacing in the two dimensional phase to allow easier modelling of smaller scale diffusion and dispersion, as well as allowing examination of infr-structure where desired.

e) Extending the flexibility of the network capabilities of the software.

10.4.3 The Models PT / PT1

The following areas are of interest for further work :

a) Considering the nature of the alternatives to the conceptual method of pollutant transport used in the one dimensional phase.Methods of permitting dispersion to be continuous within the discretized process.

b) Establishing D_x for the one dimensional phase and its relevance to the classical value of D_x as discussed elsewhere.

c) Establishing absolute values of D_x and D_y for the bay phase.Then considering the relevance to the representation of diffusion as in the model currently.

10.4.4 Jointly for Models F1/F2 and PT/PT1

The following areas are of interest for further work :

a) Integration to a database to retain simulations from the hydrodynamic phase and allowing PT and PT/1 to use sequences of these with all the flexibility afforded by integrated use.

b) Incorporating multiple seaward segments from one estuarine system to the bay to allow use in complicated deltas like the Nile and Ganges.

c) Incorporating facilities for allowing several discrete one dimensional phases to be discharging to the same two dimensional phase. For the Severn Estuary this would allow the Usk,Wye,Severn,Avon and Yeo to be modelled as sub-sections of the Severn Estuary in an integrated manner.

10.4.5 The Models ST/ST2

The following areas are of interest for further work :

a) Interchangeability of modules between these models and the model of Chapter 4, F_1 and F2.

b) Data acquisition to satisfy requirements outstanding for validation of the various phases of the model.

c) In-depth study of selected stations to collect sufficient relevant data to compute the stochastic coefficient Δ for the latter phase with a reasonable degree of reliability.

d) Formulating a policy on setting probabilistic consent standards rather than the current deterministic method of one or two figures for at most BOD and Suspended Solids in most instances. This would require a major policy shift and generally a much increased data collection task. However, the increasing flexibility to the pollutant generator is beneficial and in any event, would only formalise informal and legally transgressing arrangements in existance for the occasional exceedance of prescribed standards.

e) Investigating ways of compounding the various phases of the model to give a continuous facility for the combined entry of field and theoretical data to yield a probabilistic estimate of BOD/DO profile without operator intervention. A modular structure will still allow sectional operation of the various phases in isolation. Also, as for F1/F2 a bank of simulations can be built up to avoid duplication of expensive simulations.

f) Involving the restricted oxidation of Ammonia and de-nitrification in the formulation of the differential equations for the problem of the mean value predictor.

g) A strenuous investigation of the solution method for piecewise continuous discharges.

h) Developing a method for use in Network Databases for all phases of the model.

i) Investigating the method for non-constant boundary conditions at the upstream limit.

j) Jointly with i) , simulating the input stretch as a river (say from Brecon via Abergavenny to Newbridge) to determine a distribution for upstream boundary limits.

k) Extending the model to deal with probabilistically defined discharges. This would be especially useful in a load fluctuation situation (eg. tidelocking, high seasonal population)

l) Investigating the suitability of using the Steady State Model with the stochastic components of this model.

10.5 Models in a Database

A database is a cohesive method of data handling using an information orientated language. This is usually directly compatible with common high level languages like FORTRAN IV, ALGOL or COBOL.

Very much in the ascendant, databases will become a feature of computer use and everyday life at work and in the home. Routine analytical data, survey field data and hydrographical data will be entered to databases as a matter of routine.Already 6 regional water authorites are using a common archive available via the Water Data Unit at Reading despite its current shortcomings.[2]

Financial data is currently on less specialised databases like ADABAS (the Adaptable Database Management System[8]) and is available for use in cost models attached to quality modules. Fig. 10.5.A outlines how models fit into such a philosophy to replace the rule-of-thumb method still widely used.The key role is that of interpretation, and so experience of the real situation will and must remain to command a premium.

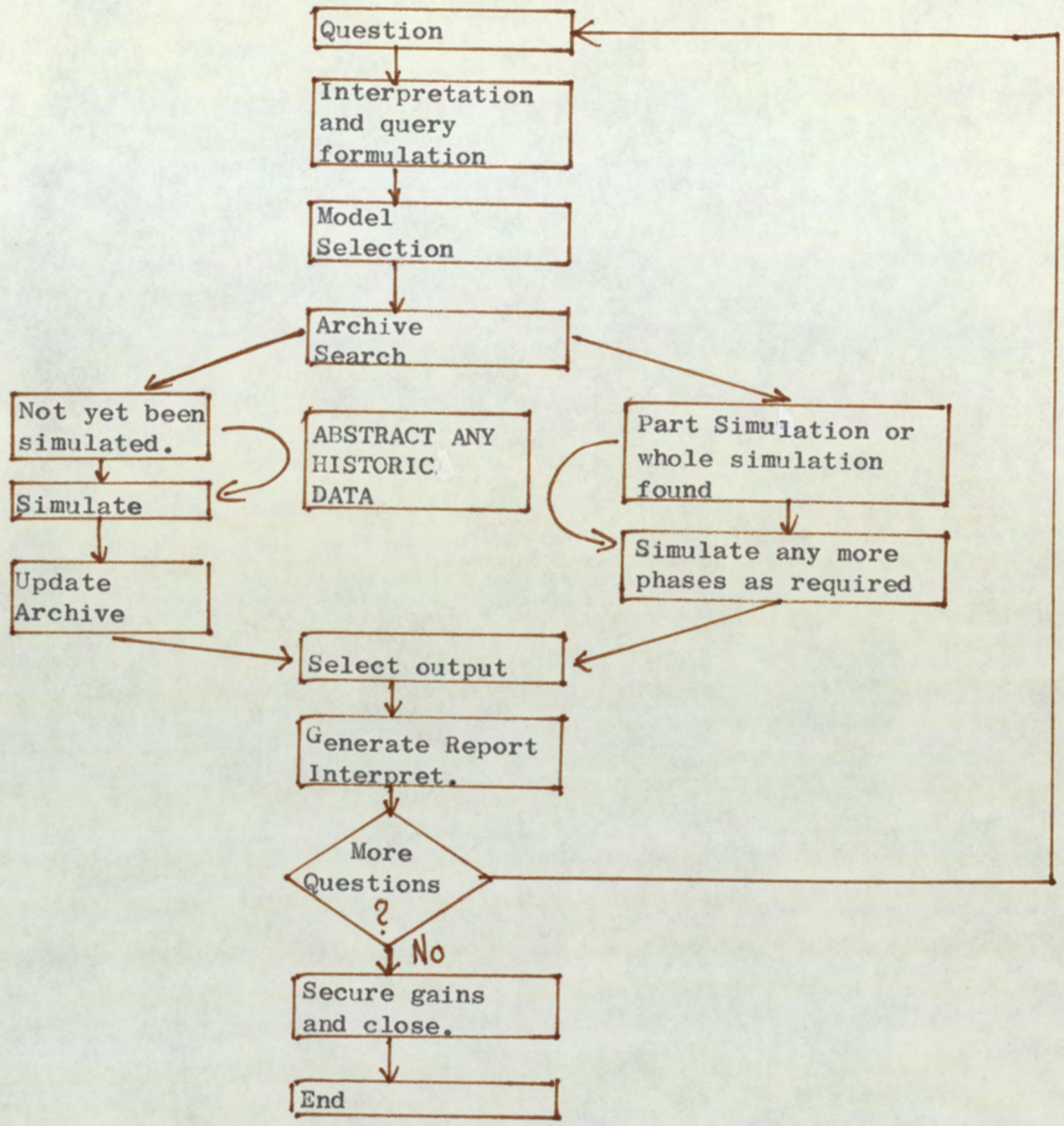
Model design should be strictly modular and machine independant and in itself not specific to any one Estuary situation.Model selection is usually not a management function, but their major role is in understanding the new power available and then

FORMULATING A SOUND SCIENTIFIC QUESTION . WHAT IS REQUIRED OF THE SYSTEM MODEL . HOW IS IT TO BE DEVELOPED AND WHAT RESOURCE DOES IT REQUIRE?

In eventual use ,

FORMULATING PRECISE, COHESIVE AND RELEVANT QUERIES AND REMEMBERING THE LIMITS OF THE PREDICTIVE PROCESS.

Fig. 10.5.A Models in a Database



10.6 Network Modelling Command Language

An outline specification for eventual integration of all the models was written but work did not progress beyond some trial modules[4].The advent of extensive finite element applications to the pollution field has gone some way towards this using specific high level languages such as FEHPOL[5][6][7].Together with the national Hydrographic Referencing Scheme, advantages of a Network Philosophy are becoming more apparent to management.

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