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CONTRIBUTIONS

TO

HYDRAULIC ENGINEERING

A collection of published papers submitted to the University of Natal, in partial fulfilment of the requirements for the degree of Doctor of Science in Engineering.

by

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February, 1984

PREFACE

Thirty-five papers are included in this submission. For convenience, they have been collected into four parts:

Part 1: Urban Hydrology and Hydraulics (UH)	10 papers
Part 2: Transients (TR)	10 papers
Part 3: Computer Aided Design (CAD)	8 papers
Part 4: Instrumentation and Control (IC)	7 papers

Within each group, papers in scientific journals are listed first, followed by papers in refereed conference proceedings, and, lastly, reports and books. In the Contents, starting on the following page, each article is identified by two serial numbers. The first (e.g. UH, TR, CAD, IC) refers to their sequence in this submission. The second (e.g. PC, UC, R, T) refers to the chronological listing in the Appendices. In the introduction to each part, a full declaration is given of the writer's share in the conjoint works submitted. Finally, a complete list of the writer's publications and of theses supervised is appended, in order to acknowledge the collaboration involved.

DECLARATION

I declare that, except where indicated otherwise, the work reported herein is directly the result of my own efforts, and has not been submitted for any degree in another University.

William James

William James

ACKNOWLEDGEMENTS

The Appendices list graduate theses supervised by the writer, and the original research reports upon which many of the submitted papers are based. In some cases, the papers themselves acknowledge experimental work carried out by research assistants. Collaboration with these former graduate students provided the real background for most of the work published here, and continues to form the most satisfying aspect of an academic career. One student in particular has continued to work with me for six years - the results of our effective cooperation are obvious in this submission.

Funding for the research was provided by the various agencies acknowledged in the individual papers.

For readers who may not know, these publications could not have seen the light of day without the continuous support and encouragement of Dr. Lyn James, happily, my wife.

CONTENTS

"Coordinated Multi-processing for Large Scale Data Acquisition and Simulation of Urban Drainage for the Hamilton-Wentworth Region", 5th Canadian Hydro-technical Conference, Canadian Society for Civil Engineering, University of New Brunswick, Fredericton, N.B., Canada, pp. 801- 815, May 1981, with Robinson, M.A. UH3 (C31)

"Introduction to Computational Hydrology and Hydraulics, 1982", CHI Publications, (530 pp.) January 1983. UH2* (R98)

"A Sufficient Quantity of Pure and Wholesome Water", Phelps Publishing Co., London, (150 pp.), September 1978, with James, E.M.. UH1* (R55)

* Not bound in this collection, but submitted as separate books.

PART 2: TRANSIENTS

"Resonator Studies for Kincardine Harbour, Lake Huron", Canadian Journal of Civil Engineering, Vol. 7 No. 1, pp. 133-143, March 1980, with Cuthbert, D.R. TR10 (P23)

"Power from Waves Using Harbour Resonators", Journal of Waterway, Port, Coastal and Ocean Division, A.S.C.E., Vol. 106, No. WW1, Proc. Paper 15210, pp. 99-114, February 1980. TR9 (P22)

"Two Innovations for Improving Harbour Resonators", Journal of the Waterways, Harbour and Coastal Engineering Division, A.S.C.E., Vol. 97, No. WW1, Proc. Paper 7871, pp. 115-122, February 1971. TR8 (P11)

"Desk-Top Model of Harbour Resonators", Journal of Acoustical Society America, (American Institute of Physics) Vol. 49, No. 1, (Part 1), pp. 31-32, January 1971. TR7 (P10)

"Response of Rectangular Resonators to Ocean Wave Spectra", Proceedings of the Institution of Civil Engineers (London), Vol. 48, Paper 7336, pp. 51-63, January 1971. TR6 (P9)

"An Experimental Study of End-Effects for Rectangular Resonators on Narrow Channels", Journal of Fluid Mechanics, Vol. 44, Part 3, pp. 615-621, November 1970. TR5 (P8)

"Resolution of Partial Clapotis", Journal of the Waterway, Harbours and Coastal Engineering Division, A.S.C.E., Vol. 96, No. WW1, pp. 167-170, February 1970. TR4 (P6)

"Numerical Computations for Tidal Propogation in the St. Lucia Estuary", Trans. S. African Institute Civil Engineers, Vol. 11, No. 12, pp. 323-326, December 1969, with Horne, C.W.D. TR3 (P5)

"Spectral Response of Harbour Resonator Configurations", Proceedings of the Twelfth Conference on Coastal Engineering, Washington, D.C., Chapter 132, pp. 2181-2194, A.S.C.E., September 1970. TR2 (C5)

"Rectangular Resonators for Harbour Entrances", Proceedings of the Eleventh Conference on Coastal Engineering, London, England, American Society of Civil Engineers, Chapter 98, pp. 1512-1530, September 1968. TR1 (C2)

PART 3: COMPUTER AIDED DESIGN

"Interactive Processors for Design Use of Large Program Packages", Canadian Journal of Civil Engineering, Vol. 9, No. 3, pp. 449-457, September 1982, with Robinson, M.A. CAD8 (P28)

"Standard Terms of Reference to Ensure Satisfactory Computer-Based Drainage Design Studies", Canadian Journal of Civil Engineering, Vol. 8, No. 3, pp. 294-303, March 1981, with Robinson, M.A. CAD7 (P26)

"A Program Package for Interactive Design of Optimal Pipe Networks for any Climatic Region in Canada", Canadian Journal of Civil Engineering, Vol. 6, No. 3, pp. 365-372, September 1979, with Robinson, M.A. CAD6 (P20)

"Developing Simulation Models", Journal of Water Resources Research, Vol. 8, No. 6, pp. 1590-1592, December 1972. CAD5 (P16)

"Computational Hydraulics Systems as Distributed Data Management", Recent Developments in Computer Applications in Civil Engineering, Canadian Society for Civil Engineering, 1983 Annual Conference, Ottawa, pp. 129-142, June 1-3, 1983 (invited paper) with Unal, A. CAD4 (C45)

"Interactive Design Using Micro-Processors Communicating with Large Batch-Oriented Packages at Remote Mainframes", Proceedings of the 2nd International Conference on Engineering Software, London, England, "Engineering Software II", pp. 291-305, March 1981, with Robinson, M.A. CAD3 (C29)

"FastSWMM", Canadian Society for Civil Engineering Conference on Computer Applications in Hydrotechnical and Municipal Engineering, Toronto, pp. 325-341, May 1978. CAD2 (C12)

"Development and Applications of Large Interactive Simulation Packages in Environmental Science", Proceedings 4th Ontario Universities Computing Conference, Toronto, Ontario, pp. 7-13, February 1973, with Zachar, P.P. CAD1 (C6)

PART 4: INSTRUMENTATION AND CONTROL

"Precipitation Instrumentation Package for Sampling of Rainfall", Institute of Electrical and Electronics Engineers (Transactions on Instruumentation and Measurement), Vol. IM32, No. 3, pp. 423-429, September 1983, with Haro, H. and Kitai, R. IC7 (P30)

"Power Spectral Analysis of a Forcemain Failure Caused by Water Hammer", Canadian Journal of Civil Engineering, Vol. 7, No. 1, pp. 36-44, March 1980, with Hennesy, R.R. IC6 (P24)

"Approximately Linear Low-Pass Wave Filters", Journal of the Waterways, Harbours and Coastal Engineering Division, A.S.C.E., Vol. 99, No. WW1, Proc. Paper 9534, pp. 69-68, February 1973. IC5 (P17)

"Multi-Lag Multi-Variate Auto-Regressive Model for the Generation of Operational Hydrology", Journal of Water Resources Research, Vol. 8, No. 4, pp. 1074-1076, August 1972, with Pegram, G.G.S. IC4 (P15)

"A Note on Inexpensive Telemetry of River Sediment Concentrations", Bulletin International Association Scientific Hydrology, Vol. XV1, No. 2, pp. 87-90, June 1971, with Finlayson, G.D. IC3 (P13)

"A New Recording Turbidity Meter for Rivers", Bulletin International Association Scientific Hydrology, Vol. 15, No. 1, pp. 91-95, March 1970. IC2 (P4)

"Accurate Wave Measurement in the Presence of Reflections", Journal Institution Water Engineers, (London), Vol. 23, No. 8, pp. 497-501, November, 1969. IC1 (P3)

APPENDICES: COMPLETE LIST OF PUBLICATIONS

INTRODUCTION

This is a collection of published papers; it documents the research and development of innovative computer-based solutions for an important class of hydraulic and hydrological problems. The purpose of this Introduction is to describe how the studies collected here have facilitated an overall approach to the problems of pollution and flooding arising when intense storms track across metropolitan or industrial areas, dumping polluted water into rivers, harbours, etc.

The techniques necessary for computer management of urban stormwater pollution and flooding include:

(a) Field instrumentation,

(b) Data capture and management,

(c) Atmospheric fallout, pollution build-up and wash-off,

(d) Storm dynamics modelling,

(e) Large scale continuous modelling of urban runoff,

(f) Unsteady flow in complex networks,

(g) Receiving water dynamics,

(h) Statistical post-processing and modelling, and

(i) Real-time control.

The earlier papers presented herein are subsumed under the title "Transients" and summarise innovative approaches to the problem of handling transients and unsteady flow in harbours. The writer's work has been summarised by Silvester in "Coastal

Engineering" (Elsevier, 1974). This background provided sufficient rigour for the subsequent methods used in unsteady flow in complex networks, the basic transport mechanism in urban stormwater.

Papers under "Instrumentation and Control" led towards a synthesis of microcomputer-based field instrumentation operating Transfer Function models similar to the statistical approaches used in ARIMA modelling, for real-time control.

Numerous studies were undertaken with various government agencies and private concerns, (e.g. the Ontario Ministry of the Environment, the Hamilton-Wentworth Regional Engineering Department and the Natural Science and Engineering Research Council of Canada, jointly provided funds for a major investigation into the urban drainage system in the City of Hamilton). Other studies were funded to develop computer software and field instruments to aid in the understanding of stormwater management. The Stormwater Management Model (SWMM3) has been substantially enhanced and aid is offered to a broad user group in North America and abroad.

There is a critical need for measuring rainfall intensity and stormwater discharge accurately in urban areas. This requires high spatial and time resolution, typically at one-minute intervals. To meet this need, rainfall intensity monitoring equipment was developed, based on single-chip microcomputers utilizing reusable cassette tape as a recording medium. The equipment is currently in use in Hamilton (14 gauges), in Ontario as part of the acid precipitation network (6 gauges), in the

Arctic (3 gauges), the Ottawa area (6 gauges), in Halifax (20 gauges), in Toronto (6 gauges), Oslo (6 gauges) and Kentucky. Software (FASTPLOT) was developed to process the data and to archive it on tape (INTAPE).

An important interdisciplinary research topic is the relation of atmospheric pollution to stormwater modelling. Fallout from industry and vehicular traffic is thought to be a major contributor to surface pollutant loadings in large metropolitan areas. An attempt was made to identify the sources of pollutants available for washoff, and the mechanisms of build-up and washoff. The computer program is known as ATMDST.

The development of appropriate software to determine the movement of stormwater pollutants through a city-wide network of combined sewers is a closely related problem. This software processes information on the dates of street cleaning activities and also runs in a continuous mode using a variable time-step. The program is called CHGQUAL.

A computer program package, RAINPAK, has been developed for simulating storm dynamics. Variations in storm speed and direction were found to produce storm flows and pollutant concentrations significantly different from the conventional assumptions of stationary storm distributions. The model accounts for temporal and spatial variations in storms, such as ageing, merging, splitting of storm cells, and so on. Analysis of data from rain sensors is performed by the programs STOVEL, THOR3D, THOR4D, etc., in RAINPAK.

A computer model, OVRFLO3, was developed for modelling side-weir diversion structures. This was motivated by the need to model the first flush loadings from urban areas which will reach the treatment facility, or be diverted to the outfalls.

A computer model, TOTSED, was developed to predict bed sediment and suspended sediment load as a function of time and distance along a one-dimensional quasi-steady-state receiving area near a combined sewer outfall. The model was calibrated using data obtained through a sampling program carried out in the Chedoke Creek outfall channel in Cootes Paradise in Hamilton. TOTSED provides an interface between an urban drainage network and a receiving water body.

In order to facilitate the use of these models in a teaching environment, a series of pre-processing programs has been developed. These programs, FASTSWMM3, FASTHEC2 and FASTHYMO, prompt the user at a terminal for input data in the appropriate sequence, to which he responds by providing data in free-format and/or optional commands which direct the job path along various routes. The pre-processors take care of all system job control language (JCL), design file manipulation, and submit the program to the batch queue for the user. In this way the user can focus on the hydrologic problems without having to be concerned about the computer system, thereby reducing time expended on learning and/or carrying out calibration, validation or sensitivity analyses.

The models ATMDST, CHGQUAL, RAINPAK, OVRFLO and TOTSED are designed to be incorporated as additional "blocks" of SWMM3. The interactive processors are also designed to drive SWMM3. The whole package is called FASTSWMM3.

A field program is an essential part of the research. The activities were generally as follows:

a. Install and maintain flow gauges at urban streams, overflow structures and outfalls,
b. Install and maintain a rainfall intensity recording network in metropolitan areas,
c. Collect stormwater samples at each flowgauging location for selected storm events at approximatley five minute intervals. These samples were subsequently analyzed for suspended solids, BOD5, nitrogen and phosphorous, as well as, occasionally, other constituents.

Using observed event rainfall intensity, discharge and pollutant concentration data, obtained from the field program, a discrete event model of the Hamilton urban drainage system was constructed, calibrated and validated. The RUNOFF Block of SWMM3 was used for this purpose. The basic time-step for the general model is five minutes. A coarse model of the system (time-step of one hour) was also developed; all diversion structures were assumed to be operating such that flow was directed to the receiving waters.

The coarse model of the drainage system was run continuously for a period of nine years (May to October inclusive) at a time-step of one hour using rainfall data from the Environment

Canada Archives in order to develop long-term loadings of pollutants to the Hamilton receiving waters. Special routines (DATANAL) were written to interface with these data tapes.

The results of the continuous modelling were subjected to statistical analyses, in order to develop "easy-to-evaluate" equations for predicting stormwater pollutant loadings for interfacing with a model of the Hamilton harbour. A number of studies were carried out which dealt directly with the topic of urban pollutant loadings.

In order to supply some ideas to sponsoring authorities for lessening the impacts of pollutant loadings to the receiving waters, an investigation of management alternatives for combined sewer overflows for Hamilton has been carried out. The study examines alternatives such as real-time control of the drainage system and in-line storage, using microcomputers located in diversion structures and in storage tanks. A detailed study of the design of a microprocessor circuit for installation in a specific, existing overflow structure was conducted. This is integrated with the rainfall and discharge monitoring equipment mentioned earlier to control overflows and make maximum use of in-system storage.

All of the foregoing is synthesized in the writer's recent work on a new single-user multi-microcomputer configuration to manage the drainage system of a large industrial city. One computer models atmospheric pollutants, another tracks an in-coming storm, another monitors the stormwater washoff from the city, and so on; each computer shares the same data base, and the whole configuration is expected to show considerable savings and improvements over existing techniques.

The writer's work on urban hydrology has recently been independently placed in a national and international perspective by Marsalek in "Progress Since 1979 in Canada", in Urban Hydrology, (Proceedings of an International Symposium), Published by the American Society of Civil Engineers, pp. 19-37, June 1983.

PART 1: URBAN HYDROLOGY AND HYDRAULICS (10 papers)

Apart from the exceptions listed below, I wrote and produced all the papers. Except for papers UH1, UH3 and UH4, the co-authors were postgraduate students, working for Ph.D. or M.Eng. (by research or by instructional course) degrees, under my direct supervision. In all cases, the original idea, the lines of research, and the theoretical and experimental procedures to be followed, were laid down by me, and a considerable proportion of the basic analysis carried out by myself. The co-authors had no previous experience in these fields and their program of work was decided by me. They were generally responsible for carrying out the experimental procedures, developing the computer code, and undertaking the computation, all under my day-to-day supervision.

EXCEPTIONS:

Paper UH7: The computer code was developed over three years by two successive M.Eng. students (Z. Schtifter and R. Scheckenberger) both under my supervision. The storm model follows the same innovative principles of spatially distributed dynamic boundary conditions as in UH9 and UH8 and was my own, although it was considerably helped along by discussions with Dr. J. Drake, a colleague in Geography at McMaster University. In UH9, the ideal of spatially distributed surface drag and its theoretical derivation was my own and quite novel at the time. In UH8, the idea of spatially distributed bed drag due to flexible roughness elements (aquatic weeds) was similarly original and entirely due to me - that analysis was set out in detail by myself.

Book UH1: This book, although based on my own seven year study, was jointly written by myself and my wife, Dr. Evelyn James. All of the research and engineering background was provided by me. The book bears joint responsibility, however.

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Modelling side-weir diversion structures for stormwater management

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Modelling side-weir diversion structures for stormwater management

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Combined sewer diversions are designed to divert significant flows directly to the receiving water during rainstorms. To estimate pollutant loadings to the recipient water it is necessary to compute continuous hydrographs and pollutographs for the full period of potential overflow. Since a diversion is active only during part of a storm, diverting only part of the flow, a rating curve for the diversion structure must be obtained. Few structures have been adequately calibrated and their rating curves are not usually available.

Overflow and diversion structures may be classified according to the shape of the sill or overflow control: end weirs, oblique weirs, and side spillways. Many structures experience hydraulic jumps of complex shape over much of the operating range. This paper reviews the hydraulics of side spillways and the SWMM-EXTRAN computer program. A new program called OVRFLO3 has been developed to dovetail with the SWMM package. Internal SWMM coding has not been changed. As a stand-alone program, OVRFLO3 will produce rating curves for side-weir diversion structures. The program has been applied to urban catchments in Hamilton, Canada.

Validation of OVRFLO3 is discussed in the paper. Comparisons were also made between OVRFLO3, SWMM-EXTRAN, and a series of previous laboratory experiments conducted by others.

La chambre de dérivation d'un égoût unitaire est calculée pour laisser passer directement au cours d'eau récepteur une partie importante des eaux d'orage. Pour déterminer la charge organique ainsi transmise aux eaux du récepteur, il faut calculer des hydrogrammes et des pollugrammes pour toute la période du trop-plein. Puisque l'ouvrage de dérivation ne sert que pendant une partie d'un orage en ne laissant passer qu'une fraction des eaux de l'hydrogramme, on doit établir sa courbe d'étalonnage. Peu d'ouvrages ont été adéquatement étalonnés et les courbes correspondantes ne sont habituellement pas disponibles.

Les organes contrôlant le trop-plein et la dérivation peuvent être classifiés d'après la forme de leur seuil: déversoir transversal, déversoir oblique et évaluateur latéral. Plusieurs ouvrages donnent lieu à des ressorts hydrauliques de forme complexe sur une partie importante de leur plage de fonctionnement. L'article présente, sous l'aspect hydraulique, une étude synthétique de l'évacuateur latéral et le programme SWMM-EXTRAN. On a mis au point un nouveau programme appelé OVRFLO3 compatible avec le logiciel SWMM dont le codage interne reste inchangé. Exploité seul, le programme OVRFLO3 calcule les courbes d'étalonnage des ouvrages de dérivation à déversoir latéral. Il a été appliqué à l'étude des eaux urbaines à Hamilton (Canada).

Les auteurs font état des travaux de vérification du programme OVRFLO3. Au cours de leur étude, ils ont comparé les résultats tirés de OVRFLO3, de SWMM-EXTRAN et d'essais antérieurs menés en laboratoire par d'autres hydrauliciens.

[Traduit par la revue]

Can. J. Civ. Eng., 9, 197–205 (1982)

Introduction

The development of OVRFLO3 was part of a joint study by McMaster University, the Hamilton-Wentworth Regional Engineering Department, and the Ontario Ministry of the Environment, of the urban drainage system in the City of Hamilton. The purpose of this joint study is to formulate a model of the Hamilton urban drainage system in order to estimate annual loadings to the harbour receiving water of suspended solids, BOD_5, nitrogen, and phosphates.

Several aspects of the problem have been covered. James and Robinson (1981) developed interactive packages using microprocessors communicating with large-scale batch-oriented programs at remote mainframes. James and Drake (1980) developed a kinematic storm model THOR to produce rainfall hyetographs. El-Zawahry (1981) wrote an algorithm SEDTOT for sedi-

TABLE 1. Verification tests on EXTRAN

Test No.	Condition	Q_i (ft^3/s)	Q_i (m^3/s)
1A	Single pipe under pressure	20	0.566
2A	Continuous pipes under pressure	20	0.566
3A	Simple network free surface	3	0.085
		2	0.057
		1	0.028
4A	Simple network pressure flow	30	0.849
		20	0.566
		10	0.283
1B	Orifice, free surface	67.143	1.9
2B	Orifice, pressure flow	150	4.245
1C	Transverse weir	150	4.245
2C	Side weir	150	4.245

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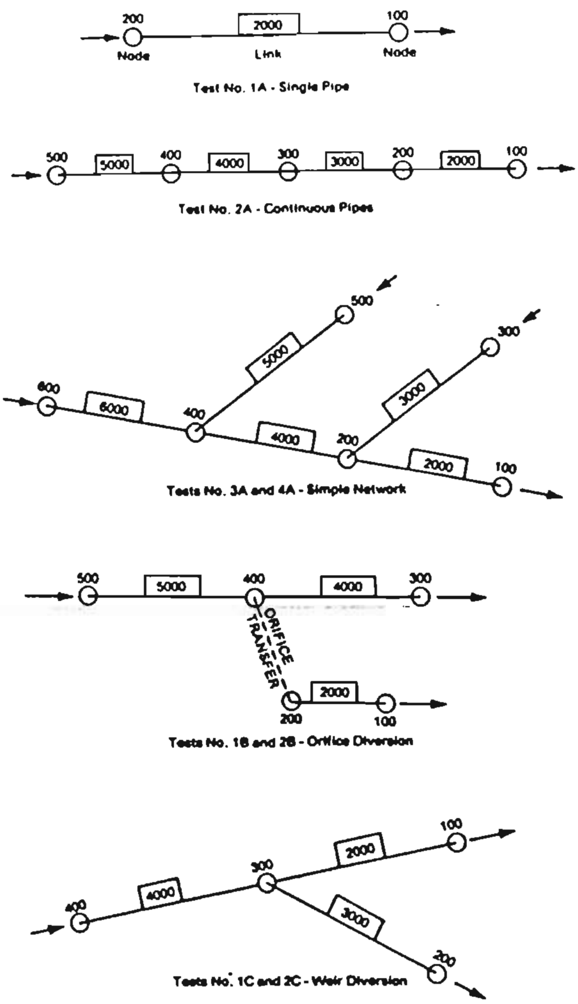

FIG. 1. Representation of verification tests showing numbering scheme.

TABLE 2. Test group A (% errors)

Test	Continuity error	Depth error	Velocity error
1A	0.0	−0.25	−3.125
2A	0.0	−2.25	0.16
3A	0.0	0.68	−1.69
4A	Unbalanced system		

TABLE 3. Test groups B and C (% errors)

Test	Continuity error	Orifice flow error	Weir flow error
1B	0.02	5.20	—
1C	0.0	—	42.43
2C	0.0	—	46.40

ment deposition and resuspension in the outfall zone, and applied it to Chedoke Creek. Meanwhile, Shivalingaiah[1] is evaluating pollutional loads from stormwater, Henry[1] is studying microprocessor-controlled in-line storage as a solution to the problem, and Scheckenberger[1] is further developing the THOR model as a design storm tool. Further publications on the project are available from the present authors.

Study problem

A widely used method for the removal of excess water from a sewer network is by means of side weirs. When the water level rises above the weir crest, overspill is diverted from the channel. Dry weather flow proceeds to the sewage treatment plant, but in times of heavy rainfall most of the incoming flow passes over the weir to the combined sewer outfall. Many city drainage systems incorporate hundreds of such diversions.

A complete analytical solution of the equations governing flow in side-weir channels for an arbitrary geometry of the diversion structure is not possible. Until quite recently, methods were based on empirical design data derived from experiments conducted over a limited range of the many variables involved. However, relatively simple step computation procedures permit solutions for many shapes of cross section and for a wide range of variables. Athough the underlying hydrodynamic equations are not solved rigorously, flow behaviour in the region of the side weir and diverted flow for various flows upstream and downstream can be compared.

Suitable computer coding for diversion structures would be vital to most urban drainage models. Of the computer models that are in current use the Stormwater Management Model (SWMM) (Huber *et al.* 1975) was chosen in this study. SWMM was developed in 1971 and gradually improved by several programming groups in the United States. In this paper the computed results are compared with laboratory experiments conducted by previous workers.

Flow diversions in SWMM

The Extended Transport (EXTRAN) block of SWMM consists of two sections, quantity and quality (Urban Drainage Seminar 1979). Only the quantity section is considered here. A set of verification tests was carried out by the present authors on the EXTRAN subprograms, especially for flow regulation devices such as outfalls, orifices, and weirs. EXTRAN uses a link-node description of the sewer system. Links transmit flow from node to node and represent pipes whereas

[1]Thesis work in progress, Department of Civil Engineering, McMaster University, February 1982.

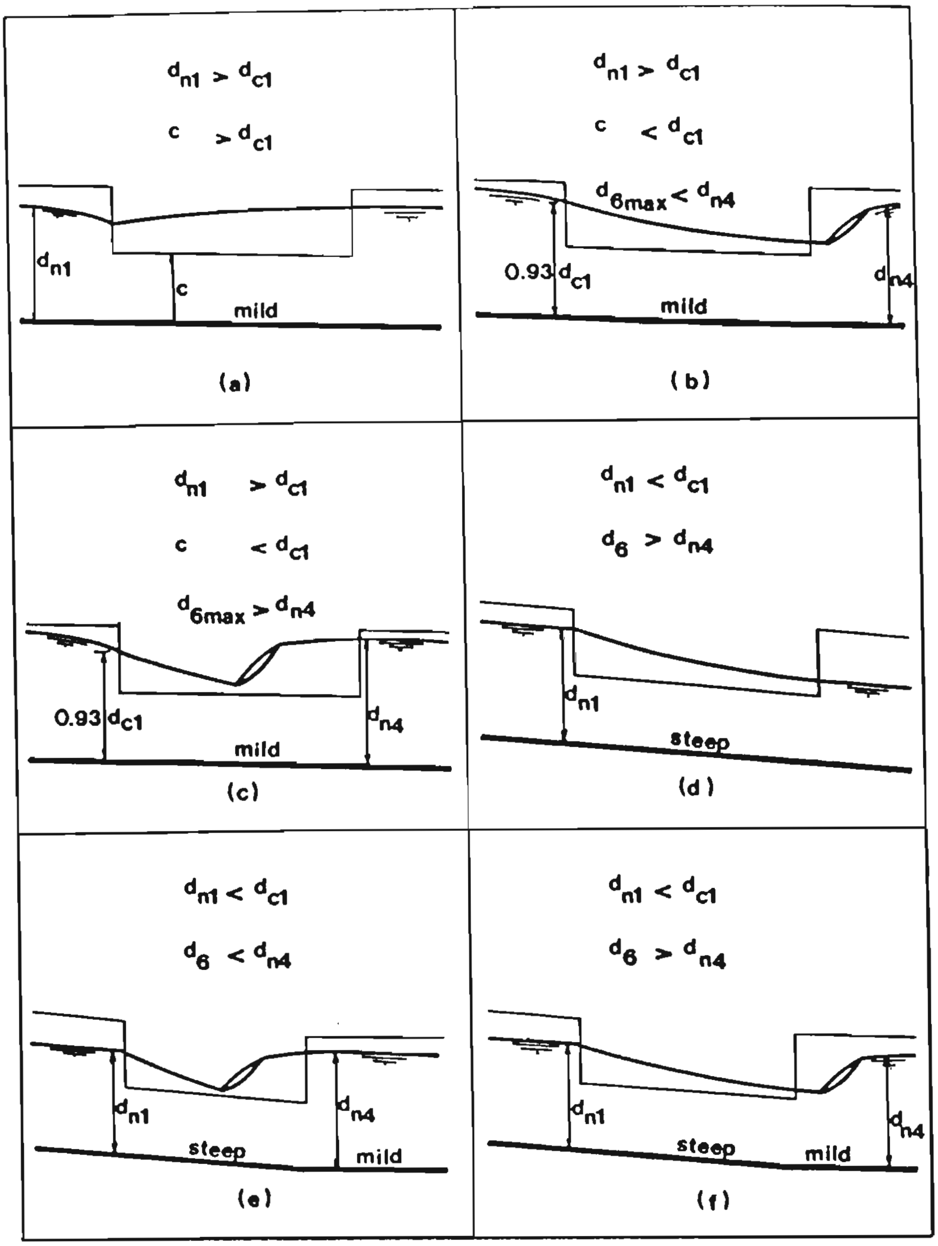

FIG. 2. Various flow profiles in a channel with a side weir.

nodes are the storage elements and correspond to pipe junctions or manholes.

The expression used in EXTRAN to calculate the flow through an orifice is the usual Torricelli theorem, and the coding appears to give good results.

The expression used in EXTRAN to calculate weir overflows (Q_s) is:

$$[1] \quad Q_s = C_w L\left(\left(h + \frac{V^2}{2g}\right)^a - \left(\frac{V^2}{2g}\right)^a\right)$$

where C_w = discharge coefficient, L = weir length, h = driving head on the weir, V = approach velocity, and a = weir exponent ($\frac{3}{2}$ for transverse weirs and $\frac{5}{3}$ for side weirs). However, Ackers (1957) revealed that the assumption of constant velocity in calculating the overflow is hardly valid.

Description of verification tests

Verification tests carried out on EXTRAN were classified into three groups, one for each control device: group A — outfalls; group B — orifice diversions; group C — weir diversions.

UH10

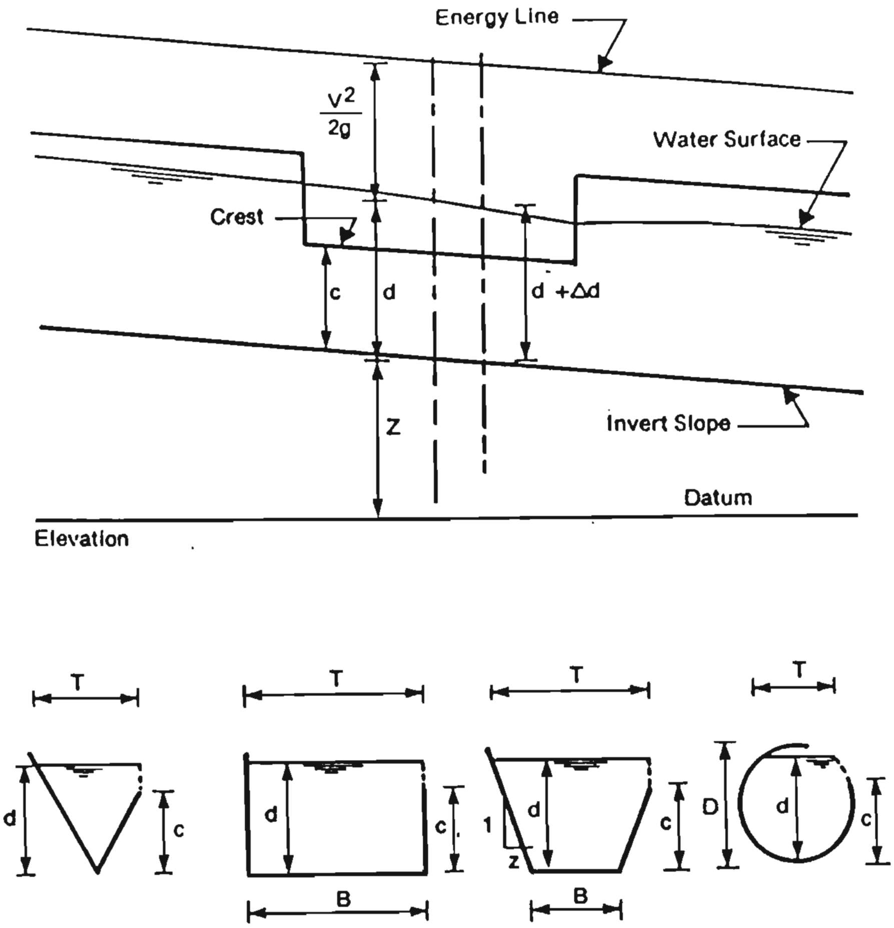

FIG. 3. Definition sketch for notation.

Although the flow condition in the link may be both nonuniform and unsteady, it was assumed to be steady in all the tests, using a constant inflow (Q_i) to the system. Table 1 presents details of the tests. A constant Manning coefficient n of 0.013 was used in all tests. For each test computed continuity, flow depth at the outlet, and velocity in the conduits were checked manually. At the outlet of the system the flow depth is critical since the outfall is free. The critical depth was calculated with the aid of Chow's tables (Chow 1959). Figure 1 shows the conceptual representation of each test.

Tables 2 and 3 summarize the results. In addition, the following observations can be made:

1. EXTRAN keeps the system balanced, in terms of equating the sum of inflows to the sum of outflows, until the system is under pressure, when the numerical scheme tends to become unstable.

2. The error between EXTRAN and hand calculated velocities in conduits with low flows may reach 51% (test 3A, discharge 1 ft^3/s (0.028 m^3/s)).

3. The error in the freeflow orifice discharge test was only 5.20%. The test could not be evaluated for pressure flow.

4. The expression used for describing the flow over weirs does not seem to be applied correctly by the program, calculated errors of 46% being obtained.

As can be seen from Table 3, the simulation of weir diversions in EXTRAN does not appear to be accurate (46.40% error).

TABLE 4. Still flow forward velocity U

Approach channel	Along side weir	Q_s/Q_0	Relationship
Subcrit.	Subcrit.	<0.50	$U=0.91(d/c)V$
Subcrit.	Subcrit.	>0.50	$U=1.08V_1(d_1-c)/(d-c)$
Subcrit.	Supercrit.		$U=F_1V$
Supercrit.	Supercrit.		$U=V$

Side weirs

The hydraulic behaviour of diversion structures incorporating side weirs has received considerable atten-

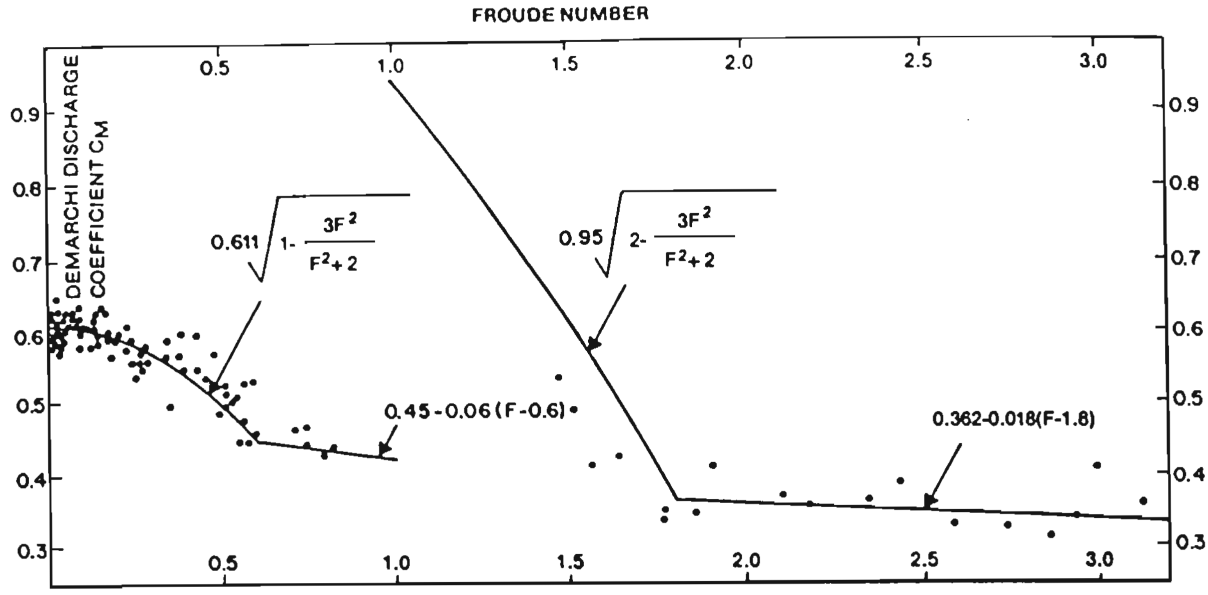

FIG. 4. Variation of C_M with Froude number F.

TABLE 5. Comparison between the programs

	OVRFLO1	OVRFLO2	OVRFLO3
Main channel section	Rectangular, trapezoidal	Rectangular, trapezoidal	Rectangular, trapezoidal, triangular
Channel bed along side weir	Prismatic or tapering	Prismatic	Prismatic or tapering
Basic approach	Constant energy less friction	Momentum approach	Momentum approach
Discharge coefficient	Assumed	Assumed	Computed

tion since the beginning of the century. A large number of the studies were empirical (DeMarchi 1934; Frazer 1957; El-Khashab and Smith 1976). The water surface profile along the side weir for various flow conditions is shown in Fig. 2. The profiles are best described by the differential equation derived by El-Khashab and Smith (1976) based on the momentum balance along the side weir. Briefly restated, the equation is:

$$[2] \quad \frac{\mathrm{d}d}{\mathrm{d}x} = \frac{S_0 - S_f - \frac{1}{gA}(2\beta V - U)\frac{\mathrm{d}Q}{\mathrm{d}x}}{1 - \beta Q^2T/(gA^3)}$$

where: S_0 = slope of channel bed, S_f = slope of friction line, β = momentum correction factor, V = mean velocity in channel, U = longitudinal component of spill velocity (parallel to supply channel), g = gravitational acceleration, A = cross-sectional area of flow, $\mathrm{d}Q/\mathrm{d}x$ = rate of change of main channel flow, x = distance along the side weir, Q = main channel flow, and T = water surface width (the notation is also defined in Fig. 3).

It should be noted that this equation applies only to prismatic channels of triangular, rectangular, trapezoidal, or circular section. However, Smith (1973) derived an expression for trapezoidal tapering channels (i.e., the breadth decreases with distance along the side weir).

Our general equation applicable to tapering channels is

$$[3] \quad \frac{\mathrm{d}d}{\mathrm{d}x} = \frac{S_0 - S_f - \frac{1}{gA}(2\beta V - U)\frac{\mathrm{d}Q}{\mathrm{d}x} + F(\mathrm{sh})}{1 - \beta Q^2T/(gA^3)}$$

where:

$F(\mathrm{sh})$ = function of the shape of cross section

$= \beta \frac{Q^2}{gA^3} d \frac{\mathrm{d}B}{\mathrm{d}x}$; rectangular or trapezoidal section

$= \frac{\beta V^2}{gA}\frac{\mathrm{d}D}{\mathrm{d}x}\left(\frac{D}{2}\cos^{-1}\left(\frac{D-2d}{D}\right) - \frac{T}{2}\right)$; circular section

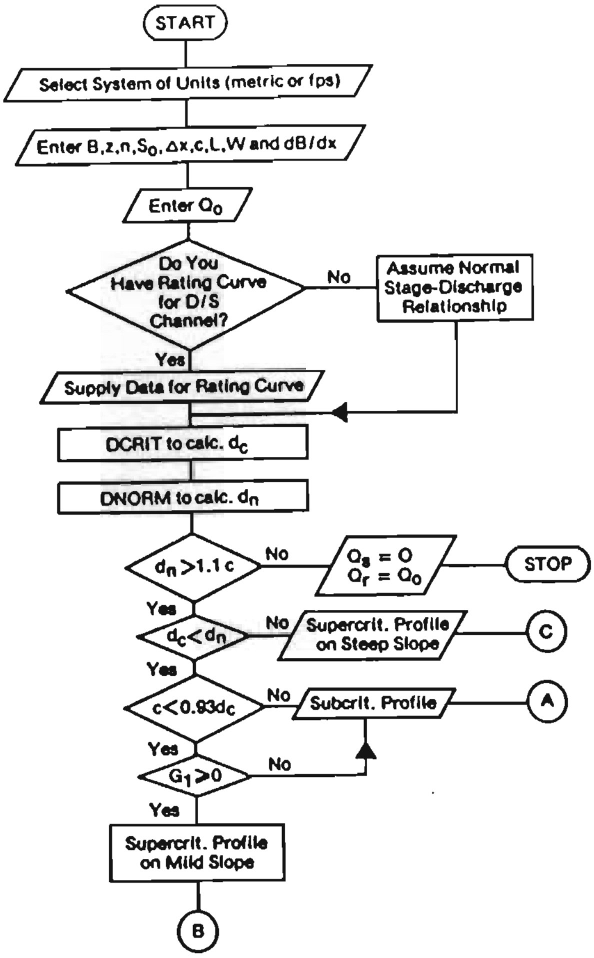

FIG. 5. Control logic part of flowchart for OVRFLO3.

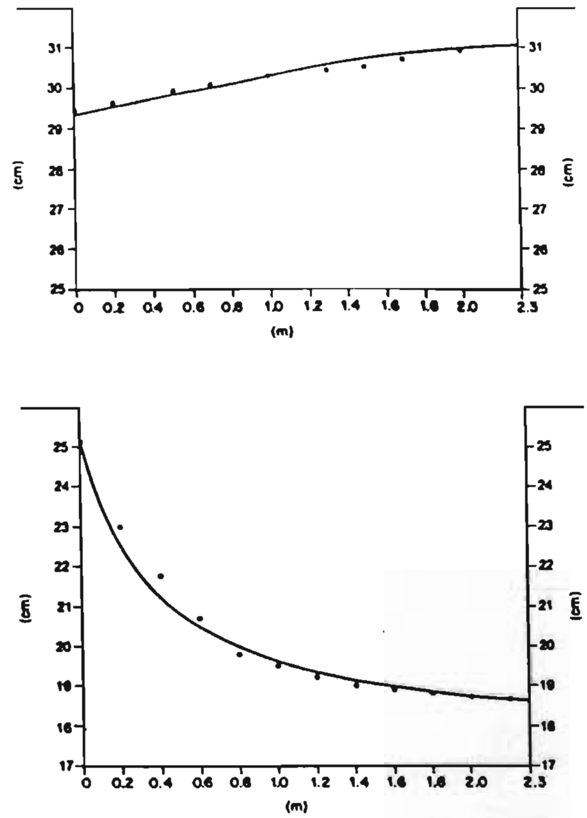

FIG. 6. Comparison of Computer results with observations for water profiles. NOTES: • observed; —— computed. Subcritical: Q_0/Q_s = 0.546; C_M = 0.539; F = 0.339; B = 0.445 m; S_0 = 0.001; n = 0.012; c = 0.25 m; L = 2.30 m; W = 1; Q_0 = 0.25 m³/s. Supercritical: C_M = 0.89003; F = 1.093; S_0 = 0.0015; Q_0 = 0.20 m³/s; c = 0.17 m.

After a series of experiments El-Khashab (1975) proposed several relationships for the forward velocity U, depending on the flow condition upstream of the side weir and the ratio (Q_s/Q_0) of the spill flow Q_s to the approach channel flow Q_0. Table 4 presents these relationships.

The discharge per unit length of weir may be expressed as

$$[4] \quad \frac{dQ}{dx} = -\frac{2}{3} W C_M \sqrt{2g}(d-c)^{1.5}$$

where: W = 1 for a weir on one side of the channel or W = 2 for weirs on both sides of the channel, d = depth of flow in main channel, c = height of weir crest above channel bed, and C_M = DeMarchi discharge coefficient.

DeMarchi discharge coefficient C_M

The coefficient of discharge is used to estimate the overflow. However, no relationships are available to estimate the discharge coefficient for a given flow condition. Allen (1957) developed an empirical expression for discharge coefficients in circular pipes only. Collinge (1959) related C_M to the Froude number F_1 at the upstream end of a side weir and recommended an average value for C_M of 0.616 for F_1 less than 0.95 and greater than 1.15. Frazer (1957) conducted a large number of experiments and finally proposed values of C_M ranging from 0.424 to 0.647 for subcritical flow in the main channel and from 0.806 to 0.555 for supercritical flow. Subramanya and Awasthy (1972) derived two relationships for both subcritical and supercritical flow in the main channel:

$$[5] \quad C_M = 0.611\left(1-\left(\frac{3F_1^2}{F_1^2+2}\right)\right)^{1/2}; \qquad F_1 < 1$$

$$[6] \quad C_M = 0.36 - 0.08F_1; \qquad F_1 > 2$$

Figure 4 indicates these two relationships. The following points may be noted.

1. If F_1 takes any value between 1.0 and 2.0 the Subramanya and Awasthy relations do not apply.

TABLE 6. Test results

Test No.	Q_0 (m^3/s)	c (m)	L (m)	S_0	S_d	Q_s/Q_0 Comp.	Obs.
10V	0.116	0.20	2.3	0.0012	0.00036	0.52	0.49
11V	0.128	0.10	2.3	0.0015	0.00150	0.27	0.27
12V	0.116	0.10	2.3	0.0015	0.00150	0.25	0.23
13V	0.130	0.10	1.2	0.0015	0.00150	0.22	0.20

TABLE 7. Comparison tests

Group	Test	Q_0(m^3/s)	c(m)	S_0	S_d	Q_w(m^3/s)
A	1	0.0360	0.1000	0.0177	0.0177	0.40
	2	0.0450	0.1000	0.0200	0.0200	0.40
	3	0.0410	0.0515	0.0650	0.0650	0.35
	4	0.0160	0.0515	0.0515	0.0105	0.45
	5	0.0215	0.0515	0.0209	0.0209	0.40
	6	0.0180	0.0515	0.0138	0.0138	0.40
	7	0.0290	0.0515	0.0392	0.0392	0.40
B	8	0.0113	0.10	0.0017	0.0002	0.55
	9	0.0105	0.10	0.0013	0.0001	0.55
	10	0.0125	0.10	0.0020	0.0001	0.55
	11	0.0115	0.10	0.0020	0.0003	0.55
	12	0.0155	0.10	0.0030	0.0003	0.55
	13	0.0115	0.10	0.0016	0.0002	0.55
C	14	0.0480	0.1000	0.001	0.001	0.45
	15	0.0500	0.0515	0.009	0.009	0.45
	16	0.0290	0.1000	0.002	0.002	0.45
	17	0.0205	0.0515	0.001	0.001	0.45

2. If F_1 takes values between 0.6 and 1.0, C_M will range between 0.45 and 0.0, values that seem to be unreasonable.

3. Between $F_1 = 0.6$ and 1.0 the relations are not supported by experiments.

4. C_M should increase as F_1 decreases from 1.6 to 1.0.

For coding convenience in OVRFLO3 the present writers have used four expressions to cover the complete range of F_1 (as shown in Fig. 4).

Computer program OVRFLO3

Smith (1973) developed a computer program (OVRFLO1) to calculate the water surface profile along a side weir as well as the overflow. El-Khashab (1975) developed a second program (OVRFLO2), based on the momentum equation, to compute the water surface profile. The computer program developed in this study (OVRFLO3) is based on the above two programs. The numerical scheme adopted for water surface computation is presented by Prasad (1970) and by Rajau *et al.* (1979). Table 5 lists the differences between the programs. OVRFLO3 comprises six subroutines controlled by the main program; Fig. 5 is a flow diagram showing the control logic.

OVRFLO3 validation tests

The program was run for the same conditions as those tested by El-Khashab (1975). Results of 13 tests, both for the water profile along the weir and the overflow, showed good agreement. Results for two typical runs for the water profile are presented in Fig. 6. Table 6 presents results for a further four tests.

Comparison between OVRFLO3, SWMM-EXTRAN, and experimental results

Olsson and Svensson (1979) conducted a series of experiments to measure the overflow from a channel with a side weir for different flows upstream and along the side weir. The experiments were classified into three groups according to the flow profile along the weir: group A — supercritical profile on a steep slope; group B — subcritical profile; and group C — supercritical profile on a mild slope.

The main channel used by Olsson and Svenssen mea-

TABLE 8. Results of comparison tests

		Q_r/Q_0 (%)		
		Computed		
Group	Test No.	OVRFLO3	EXTRAN	Observed
A	1	90	98	90
	2	86	97	87
	3	91	99	90
	4	80	96	81
	5	88	97	86
	6	82	97	83
	7	91	98	90
B	8	37	85	41
	9	28	86	24
	10	25	84	16
	11	45	85	48
	12	38	80	35
	13	37	85	35
C	14	73	72	71
	15	53	83	58
	16	86	81	86
	17	70	72	68

sured 0.15 m wide, 0.33 m high, and 7.3 m long. The height of the weir crest varied from 0.0515 to 0.10 m whereas the weir length was kept constant at 0.57 m in all tests. The average value of Manning's coefficient n was 0.01. Table 7 provides details of the 17 tests used for comparing computed results with measurements. In Table 8 the term Q_r/Q_0 is the ratio of the residual flow in the downstream channel to the incoming flow in the approach channel. The discharge coefficients given in Table 7 were not used in OVRFLO3 (discharge coefficients are computed in OVRFLO3); however, they were used in EXTRAN, and were recommended by Olsson and Svenssen. Results of these tests are given in Table 8 and it can be seen that OVRFLO3 produces results that are much closer to observations.

A new block for SWMM

The SWMM program is widely used in urban runoff modelling in Canada and the U.S.A. At McMaster University, the local version of SWMM was modified to include OVRFLO3 for calculating the spill flow in a channel with a side weir. This has been achieved by creating a new block in SWMM, called SIDWEIR, that includes the main OVRFLO3 program and its six subroutines. OVRFLO3 was adjusted to accept a set of input flows (i.e., inflow hydrograph) and produce two arrays, the spill flow and the associated depth at the upstream end of the side weir. These arrays are then stored on the disc file specified by the EXECUTIVE block of SWMM. When using the EXTRAN block at a side-weir node, all necessary input stored on the disc file is read. Knowing the depth at the upstream end of the weir, the corresponding spill flow is estimated by simple interpolation.

Conclusions

The general conclusions to be drawn from this work are that (a) simulation of side-weir diversion flows in the SWMM-EXTRAN program appears not to be acceptable for certain conditions, (b) program OVRFLO3 appears to provide a more satisfactory method for computing side-weir water surface profiles and overflows than is presently provided by SWMM-EXTRAN, and (c) the new block SIDWEIR can be used to produce overflow hydrographs for diversion structures with side-weirs, using a slightly modified form of the SWMM package.

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A three-dimensional model of Hamilton Harbour incorporating spatial distribution of transient surface drag

WILLIAM JAMES AND BASEM EID

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A three-dimensional model of Hamilton Harbour incorporating spatial distribution of transient surface drag

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This paper discusses the formulation of surface boundary conditions for a three-dimensional transport model for shallow lakes, specifically for Hamilton Harbour. The same hydrodynamic equations that describe the circulation of the ocean and the Great Lakes were used in this study. However, the boundary conditions (bed topography, shoreline configuration, and surface and bottom shear stress fields) have bigger effects on circulation in shallow enclosed lakes.

In this study the flow is assumed to be incompressible and in hydrostatic equilibrium. A layered system is used in which the lake is considered to consist of a number of unequal layers in the vertical. The hydrodynamic equations are integrated vertically over each layer, and both vertical and horizontal eddy viscosities are introduced.

The over-water wind stress is determined using the logarithmic wind velocity distribution and Von Karman's integral equation for turbulent flow over a rough movable surface of variable roughness, in conjunction with equations for wind-wave generation. Thus the wind drag coefficient is determined as a function of wind and wave characteristics, and is time- and space-dependent.

Cet article présente une critique du mode de formulation des conditions aux limites de surface pour les modèles tridimensionnels de simulation du transport dans les lacs peu profonds (e.g., le port de Hamilton). On a utilisé dans cette étude les mêmes équations hydrodynamiques qui décrivent la circulation dans l'océan et les Grands Lacs. Cependant, les conditions aux limites (la topographie du fond, la configuration des rives, et les champs de cisaillement de surface et de fond) exercent une plus grande influence sur la circulation dans les lacs peu profonds.

Dans la présente étude, on admet que l'écoulement est incompressible et en équilibre hydrostatique. On recourt à un modèle stratifié qui consiste en un certain nombre de couches inégales superposées. Les équations hydrodynamiques sont intégrées par rapport à la verticale dans chaque couche et les composantes verticales et horizontales des diffusivités tourbillonnaires sont introduites simultanément.

Les contraintes de frottement dues au vent sont déterminées en fonction d'une répartition logarithmique des vitesses du vent et au moyen des équations intégrales de Von Karman pour les écoulements turbulents au-dessus d'un fond mobile de rugosité variable, en combinaison avec les équations des vagues générées par le vent. Le coefficient de traînée du vent s'établit ainsi comme une fonction des caractéristiques du complexe vent–vagues; il est donc également fonction du temps et de l'espace.

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Introduction

Recently there has been increased interest in using numerical hydrodynamic models for scientific and practical purposes in order to explore physical, chemical, and biological processes of lakes. In the last few years, numerical models of lake currents have become quite detailed and realistic. Several three-dimensional time-dependent models have been developed (e.g., Simons 1973; Leendertse 1973, 1975). Such models are necessary for stratified lakes and when surface currents, velocity, and density profiles need to be calculated.

An alternative approach in the study of the three-dimensional circulation structure is the use of well-known multi-layer models (e.g., Simons 1973; Leendertse 1973, 1975; Lick 1976). In these models the lake is divided into several horizontal layers, and the interfaces between layers (except at the free surface) are defined to be permeable and fixed in space at least for the duration of a time step. Fluid moves vertically through each layer. The free surface (or thermocline) is treated as an impermeable movable surface. Integration of the hydrodynamic equations is performed for each layer to arrive at a system of equations in terms of the layer-average variables. Interlayer stresses and bottom resistance are expressed as functions of the layer-averaged velocity. These hydrodynamic equations are typically solved using finite-difference methods.

Most models that include both friction and time-dependent flow have been applied to specific real

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basins; the modelers have not attempted to generalize their results. In other words, most of these models use constant values of certain parameters, such as wind drag coefficient, bottom friction, and eddy viscosities and diffusivities, and the model prediction is usually in close agreement with the observations. Extensive research is required to establish a more generalized and reliable approach and to relate these parameters to lake and flow characteristics.

Although there are a large number of studies on the Great Lakes (e.g., Csanady 1967, 1968, 1972; Lick 1976; Simons 1973), there are a surprisingly small number of studies regarding circulation in basins of much smaller dimensions (Bengtsson 1973; Eid 1976; James and Eid 1978). In the authors' previous study on wind-driven currents in a small shallow lake of a recreational size (Eid 1976; James and Eid 1978), it was found that bottom topography and roughness, shore configuration, and vertical eddy viscosity are much more important. We found that Coriolis forces should also be considered, even for such small dimensions.

The currents in these enclosed lakes are primarily driven by wind. The relationship of wind shear stress to the wind and water surface conditions is very difficult to determine. In most, if not all, lake models, the wind drag coefficient has been assumed constant in space and time. However, this drag coefficient should be a function of wind and surface water characteristics.

In this paper a preliminary attempt has been carried out to calculate the drag coefficient as a function of wind speed, wind fetch, and wave height and celerity (representative of surface water roughness). Thus temporal and spatial variation of the drag coefficient, and consequently surface shear stress over the lake surface, can be considered when solving the hydrodynamic equations.

This method has been applied to Hamilton Harbour (at the western end of Lake Ontario). The harbour is approximately 8 km long in the northeast direction and 4.8 km long in the southwest direction with a maximum depth of 24 m and a mean depth of 13 m. The harbour is a very complicated dynamic environmental system. The water quality in the harbour was found to be below standard by the Ontario Ministry of the Environment (1974, 1975). The study of the water quality in a lake requires a knowledge of the physical processes and the interaction between these physical processes and the biochemical processes in the water body (Harris 1978; Palmer and Poulton 1976; Polak and Haffner 1978).

In any study of Hamilton Harbour, a time-dependent, three-dimensional, stratified model is necessary. The model developed by Simons (1973) for circulation in Lake Ontario has been modified to handle such a small shallow lake. The common hydrodynamic equations for lake currents have been used. The flow is assumed to be incompressible and in hydrostatic equilibrium. The lake is considered to consist of a number of horizontal layers. This concept of the 'layers' is a geometrical concept only, and should not be confused with the layers in stratified flow. The thickness of the layers is not necessarily the same for all layers. Layer thickness may be taken to be a function of temperature or density gradient. The hydrodynamic equations (conservation of mass, momentum, and energy) have been integrated over each layer. This system of partial differential equations is numerically solved using a suitable finite-difference scheme in time and space.

Basic Model Equations

Currents due to through flows are taken to be comparatively small except locally near the mouths of the rivers or outlets. In addition, temperature, and hence density, gradients cause currents or modify the existing currents. This effect must be considered in a study of lake circulation during the late spring, summer, and early fall when stratification occurs and the density gradients are large. Coriolis forces due to earth's rotation should also be considered (Eid 1976).

The basic equations used are the usual equations for conservation of mass, momentum, and energy, plus an equation of state. The fluid motions in the lake are predominantly horizontal, and the vertical acceleration is extremely small and may be neglected. Thus, the vertical equation of motion may be replaced by the hydrostatic equation. The density is considered only dependent on the temperature and can be determined through the equation of state. The flow is also considered to be incompressible. The nonlinear advective acceleration terms in the equations of motion are neglected to save computational time, as their effect is minor (Bengtsson 1973; Simons 1973). However, the effect of omitting these terms will be checked for small lakes in our future work.

Based on the above discussion, the integrated equations of motion, continuity, and heat flux, with U, V, P, and T representing the layer averages of particular layer k, and $k + 1/2$ as a subscript designating interface values of the layer, follow.

(*a*) Momentum equations:

$$[1] \quad \frac{\partial U_k}{\partial t} = f V_k - \frac{h}{\rho}\frac{\partial P}{\partial x}(\tau^{xz}/\rho)_{k+1/2} - (\tau^{xz}/\rho)_{k-1/2} + A_h\left(\frac{\partial^2 U}{\partial x^2} + \frac{\partial^2 U}{\partial y^2}\right)_k$$

[2] $$\frac{\partial V_k}{\partial t} = -fU_k - \frac{h}{\rho}\frac{\partial P}{\partial y} + (\tau^{yz}/\rho)_{k+1/2} - (\tau^{yz}/\rho)_{k-1/2} + A_h\left(\frac{\partial^2 V}{\partial x^2} + \frac{\partial^2 V}{\partial y^2}\right)_k$$

(*b*) Continuity equation, integrated from the bottom to the water surface:

[3] $$\frac{\partial \xi}{\partial t} + \sum_{L=1}^{b}\left(\frac{\partial U}{\partial x} + \frac{\partial V}{\partial y}\right)_L = 0$$

(*c*) Heat balance equation, neglecting sources and sinks of heat:

[4] $$\frac{\partial hT_k}{\partial t} + \frac{\partial UT_k}{\partial x} + \frac{\partial VT_k}{\partial y} + (wT)_{k-1/2} - (wT)_{k+1/2} = hD_H\left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2}\right)_k + \left(D_v\frac{\partial T}{\partial z}\right)_{k+1/2} - \left(D_v\frac{\partial T}{\partial z}\right)_{k-1/2}$$

(*d*) The integrated hydrostatic pressure equation:

[5] $$P = \rho_0 g(\xi - z) + \int_{-z}^{\xi} \sigma\, dz$$

and $\sigma = (\rho - \rho_0)$ is a measure of the density anomaly.

From the equation of state:

[6] $$\sigma = -\varepsilon(T - T_0)^2$$

where U and V are the integrated layer transport components in x and y directions; x, y are horizontal coordinates, positive to the east and north, respectively; z is the vertical coordinate, positive upward from the mean still water level; u, v, and w are the average layer velocity components along these axes, respectively; t is time (s); f is Coriolis parameter (for Hamilton Harbour = 10^{-4} rad/s); ρ is the water density for a given layer at a given temperature; ρ_0 is the maximum density of the water (at 4°C); ξ is the fluctuation of the free-surface elevation measured from the still water level; P is the pressure at any point; T is the temperature at any level; T_0 is the temperature of maximum water density = 4°C; τ^{xz}, τ^{yz} are the interfacial shear stress components in x and y directions; A_v and A_h are the vertical and horizontal eddy viscosities; D_v and D_H are the vertical and horizontal eddy diffusivities of heat; h is a layer thickness; g is the gravitational acceleration; ε is a constant and has a value = -6.8×10^{-6}°C (Simons 1973); L is the layer number; and b is the total number of layers.

The above system of differential equations is used to compute the three-dimensional lake transports for given initial and boundary conditions. These boundary conditions are:

(*a*) At the free surface where $z = \xi$:

[7] $$\tau^{xz} = \tau_{sx};\quad \tau^{yz} = \tau_{sy}$$

where τ_{sx} and τ_{sy} are the wind shear stresses in x and y directions. This stress is discussed in more detail in the next section. As a preliminary simplification of the model, the heat flux at the water surface as well as other sources and losses of heat are set equal to a constant or to zero. This is acceptable for non-continuous simulation.

(*b*) At an interface between any two layers:

[8] $$\tau^{xz} = \rho A_v(\partial U/\partial z);\quad \tau^{yz} = \rho A_v(\partial V/\partial z)$$

(*c*) At the bottom where $z = -H$:

[9] $$\tau^{xz} = \tau_{bx} = \rho K\frac{(U_b^2 + V_b^2)^{1/2}}{h_b^2}U_b$$
$$\tau^{yz} = \tau_{by} = \rho K\frac{(U_b^2 + V_b^2)^{1/2}}{h_b^2}V_b$$

where τ_{bx} and τ_{by} are bottom shear stress components in x and y directions; U_b and V_b are the integrated bottom layer transport components; h_b is the thickness of bottom layer; and K is a nondimensional skin friction coefficient.

In most lake models the skin friction coefficient has been taken to be constant throughout the lake. In the authors' previous study on a shallow lake with aquatic weed growth (James and Eid 1978), the bottom friction coefficient has been expressed as a function of local large scale roughness due to weed growth. Thus, the bottom shear stress can be varied spatially and temporally in the solution scheme for the hydrodynamic equations for such lake models. In addition to the above boundary conditions, the components of velocity and heat flux normal to the solid boundary vanish at that boundary. Surface currents can be determined by extrapolation of the layers' horizontal velocity components to the water surface.

Numerical Solution

The numerical method developed by Simons (1973) is the basis of this study, but of course modifications are necessary to calculate wind-driven currents in a stratified small lake, such as Hamilton Harbour.

The harbour is horizontally divided into equal squares of 1000 ft × 1000 ft (305 m × 305 m) size. The size has been chosen to adequately describe the boundaries, to reduce computation to manageable limits, and to correspond with our field data collection techniques. The scheme is shown in Fig. 1. Two lattice-type numerical grids are arranged such that

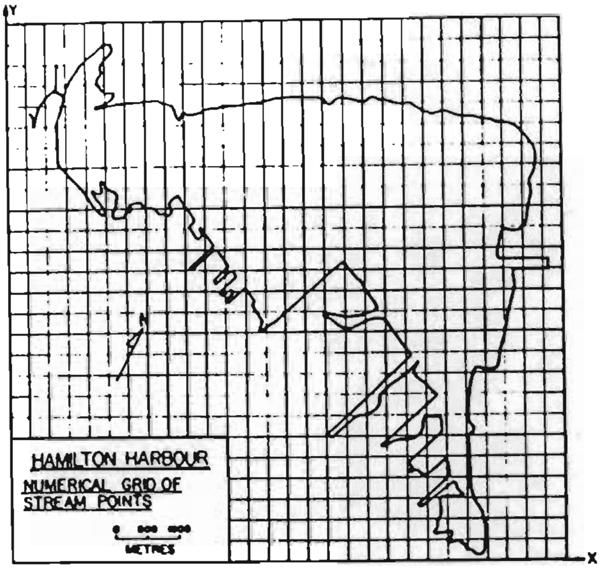

FIG. 1. Numerical grid of stream points for Hamilton Harbour.

both components of the transport U and V are specified at the grid points (or stream points) forming one lattice, while each point at the centre of each square is an elevation point at which ξ, T, P, and W are defined, forming the second lattice. Water depths are given at the stream points. The detailed description of the finite-differences technique is given by Simons (1973).

In a small lake such as Hamilton Harbour, the variables can change rapidly over a short distance in the vertical direction. Consequently, a grid size is required that is much smaller in the vertical than the horizontal direction. In the vertical direction, six unequal-thickness layers are employed such that the layer thickness decreases with increasing temperature gradient. In a nonstratified lake, the thicknesses of the layers are chosen to be smaller near the surface because of the extensive shallow zones and in order to have a sufficient number of layers for extrapolation to the surface.

Numerical Stability

For efficiency in the numerical calculation of space- and time-dependent lake problems, one would like to use as large space and time steps as are consistent with the accuracy and the physical details desired. However, there are usually other restrictions on the allowable space and time steps which might affect the stability of the numerical method used in the calculation. For example, consider the explicit, forward-time, central-space scheme used in the numerical solution in this study. Simple theory indicates that limits on the time step Δt and space steps Δz and Δx in the vertical and horizontal are approximately given by (Lick 1976):

$$[10] \qquad \Delta t < \Delta x/(gH_{max})^{1/2}$$

where H_{max} is the maximum water depth.

This means that the numerical time step must be less than the time it takes a surface gravity wave (of celerity $= (gH)^{1/2}$) to travel a grid distance Δx. For Hamilton Harbour $\Delta t < 20$ s:

$$[11] \qquad \Delta t < (\Delta z)^2/2A_v$$

i.e., the time step must be less than the time for diffusion in the vertical. In this study, assuming minimum $\Delta z = 1.0$ m and average A_v of the order of 20 cm^2/s, Δt should be of the order of 250 s.

In the method used herein, the external (vertically averaged) and internal modes of the flow field are treated separately. The time step used in the calculation of the external modes is limited by the stability condition [10] (i.e., $\Delta t \leq 20$ s); for the internal structure it is limited by the second stability requirement (i.e., $\Delta t \leq 250$ s).

Modeling of Wind Induced Surface Drag

It is necessary to determine the shear stresses imposed as a boundary condition at a lake surface when solving the hydrodynamic equations of lake transport. The relation of this stress to the wind speed is very difficult to determine theoretically and its value is usually based on semi-empirical formulae and observations. A quadratic relationship, commonly used in the bulk of the literature, is:

$$[12] \qquad \tau_s = \rho_a C_d |U_a| U_a$$

where C_d is surface drag coefficient, ρ_a is the air density (g/cm^3), and U_a is the wind speed at a certain distance z above the water surface (usually taken to be 10 m).

Many different values of the drag coefficient have been used. In these studies, the drag coefficient was usually considered constant (i.e., independent of wind speed and surface roughness) and its value was chosen to bring the model's results into close agreement with observations. The need to develop the functional dependence of the drag coefficient on wind and surface water conditions is obviously important. Many studies have attempted to determine the drag coefficient in both micrometeorology and lake modeling. The aquatic micrometeorologists, through wind profile observations, have found that the drag coefficient is nearly constant and less than that used in numerical models by a factor of two or three. Because micrometeorologists measure the drag coefficients for a fully developed wave field and for nearly steady wind conditions, the resulting values are low. Numerical modelers, on the other hand, do not use steady conditions and thus the roughness is not fully developed. They usually use higher drag coefficients (Donelan 1975).

In this paper we determine drag coefficients and consequently surface shear stresses as a function of wind speed, wind fetch, wave height, and wave age.

Methodology

The Von Karman–Prandtl logarithmic velocity profile for flow over a rough surface (Schlichting 1955) can be written:

$$[13] \quad \frac{U_z}{U_*} = \frac{1}{k}\ln\left(\frac{z+z_0}{z_0}\right) \quad \text{and} \quad U_* = (\tau_0/\rho_a)^{1/2}$$

where U_z is the wind speed at the anemometer level z above water surface; U_* is the shear velocity; τ_0 is the local shear stress at the water surface; ρ_a is the air density; k is Von Karman's universal constant = 0.4; and z_0 is the effective roughness length.

The roughness length z_0 is a parametric representation of the surface roughness due to wind generated waves. It is a function of significant wave height through the empirical relation suggested by Donelan (1975), based on observations on Lake Ontario:

$$[14] \quad z_0 = H/48$$

where H is the wave height. Equation [13] can be rewritten:

$$[15] \quad \tau_0 = \rho_a C_0 U_z^2$$

where

$$[16] \quad C_0 = \left[k/\ln\left(\frac{z+z_0}{z_0}\right)\right]^2$$

Equation [15] is similar to [12], which has been used in most numerical models. However, in [15], τ_0 is the shear stress at a fixed (nonmovable) surface, which is not the case for water surfaces. Thus C_0 in the above equation may be referred to as the 'skin friction coefficient' of a fixed rough surface. Donelan (1975) has proposed a wind–wave coupling model in which the wind drag coefficient is a function of wave height, wind speed, and wave age U_z/c (c is the wave celerity); we have used a modified empirical relation:

$$[17] \quad C_d = C_0[1 - (0.83c/U_{10})]^2$$

where C_d is wind drag coefficient for given wind speed U_{10} at 10 m above the water surface.

Now replacing C_0 by C_d in [15], we obtain an equation similar to [12], but which can be used to determine wind induced shear stress at any location in the lake.

As shown previously, the drag coefficient C_d, and consequently wind stress, can be calculated; it is a function of wind and wave conditions. The spatial variation of the drag coefficient can be obtained as follows:

(*a*) Assume that a steady wind (constant speed and direction) blows for a sufficient duration. The spatial variation of the wind, for example, observed in Hamilton Harbour during the authors' 1976 summer field work, could be taken into account by interpolation of records from (say) three wind stations. (We use weight factors inversely proportional to the square of the distances between in-water location and onshore wind stations.)

(*b*) Wind fetch is determined at each stream point on the numerical grid (distance from the upwind shore to that point along wind direction).

(*c*) Use the Sverdrup–Munk–Bretschneider (SMB) method (United States Army Coastal Engineering Research Center 1973) for determining wave characteristics as shown in the Appendix.

(*d*) Calculate C_d using [14], [16], and [17] at each grid node.

(*e*) Use [13] to get surface shear stress at each grid node for use in the hydrodynamic equations at each time step.

Results of the Numerical Model

The computer model was run for various wind conditions and model parameters in order to study the model performance. Extensive field data are still needed to fully evaluate the method but the tests are encouraging when compared with observations of surface currents in Hamilton Harbour for the summer of 1976.

The computer runs were first carried out for a nonstratified harbour. The model reached a steady state, when a constant wind was applied, after about 6 h, as shown in Fig. 2. The time to steady state when

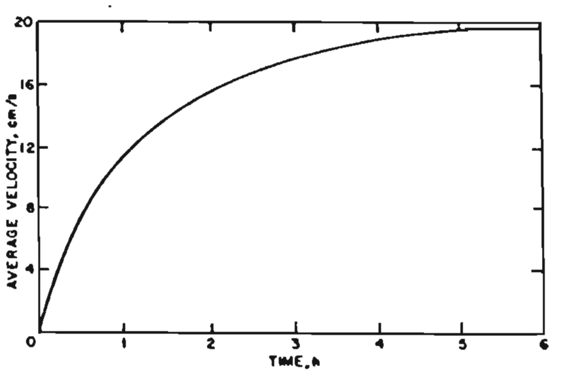

FIG. 2. Time to steady state (nonstratified lake); wind speed = 6.52 m/s; A_v = 20 cm²/s.

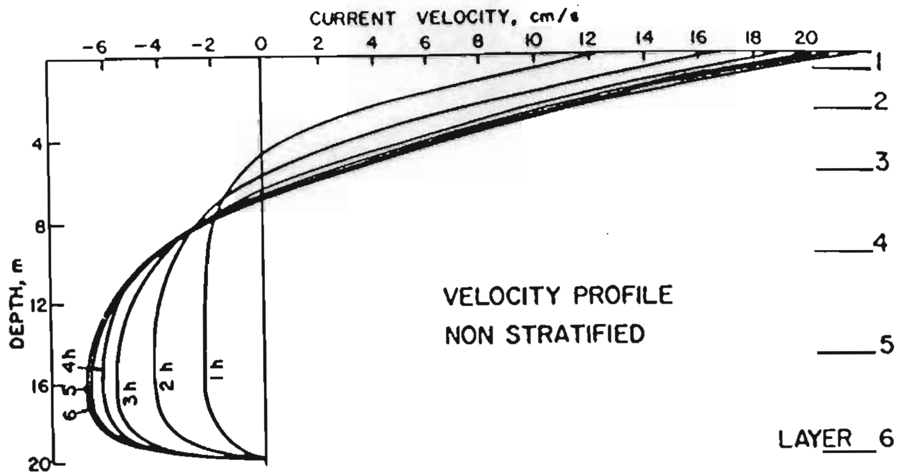

FIG. 3. Velocity profiles at the indicated simulation times.

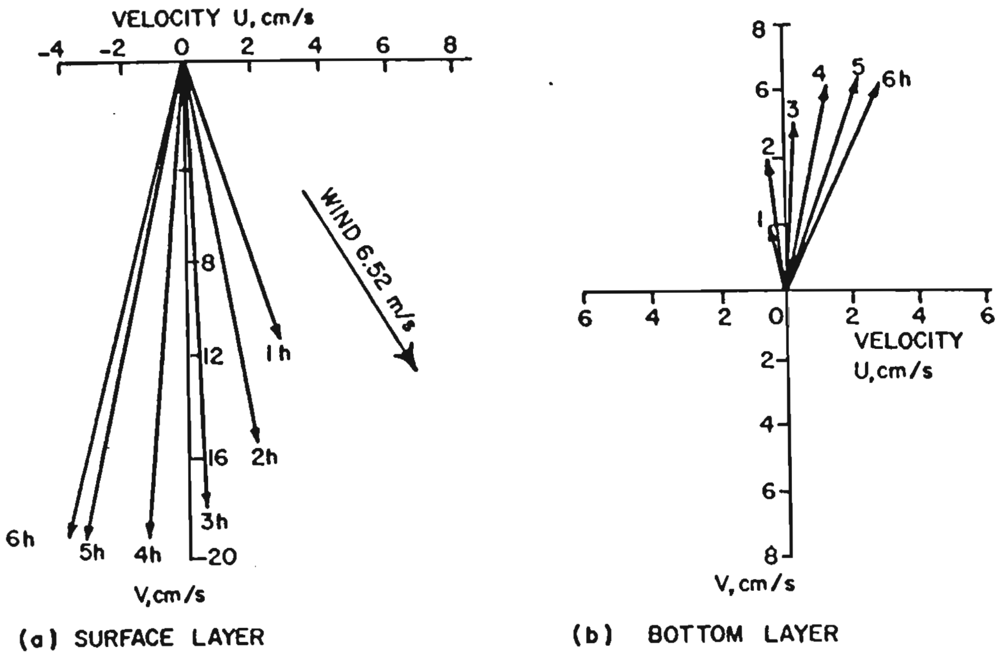

FIG. 4. Surface currents at grid point (16, 16) after the indicated simulation time.

taking stratification into account is a little longer than the time taken for nonstratified lakes. In each of these tests an approximately steady wind occurred, but of course the model can accept transient wind conditions.

The change of surface and bottom currents with time is shown in Figs. 3 and 4. As shown, the current velocity changes in both magnitude and direction. The change of direction is due to the effect of Coriolis forces (Eid 1976). A sample of the model current pattern in the first layer (about 50 cm below the harbour surface) for an observed wind event (July 16, 1976) is shown in Fig. 5. The results are deemed to be satisfactory at this stage of the model development.

The effect of temperature stratification on the model results has been studied. It was found that the harbour stratification should be considered, as it has a significant effect on the model results, as shown in Fig. 6.

The preliminary runs of the model for one wind event (e.g., $U_a = 6.52$ m/s, northwest wind) showed that the drag coefficients ranged between 1.1×10^{-3} and 1.5×10^{-3} with an average value of 1.24×10^{-3}. This value is smaller than that used in the numerical model for the Great Lakes. However, it is

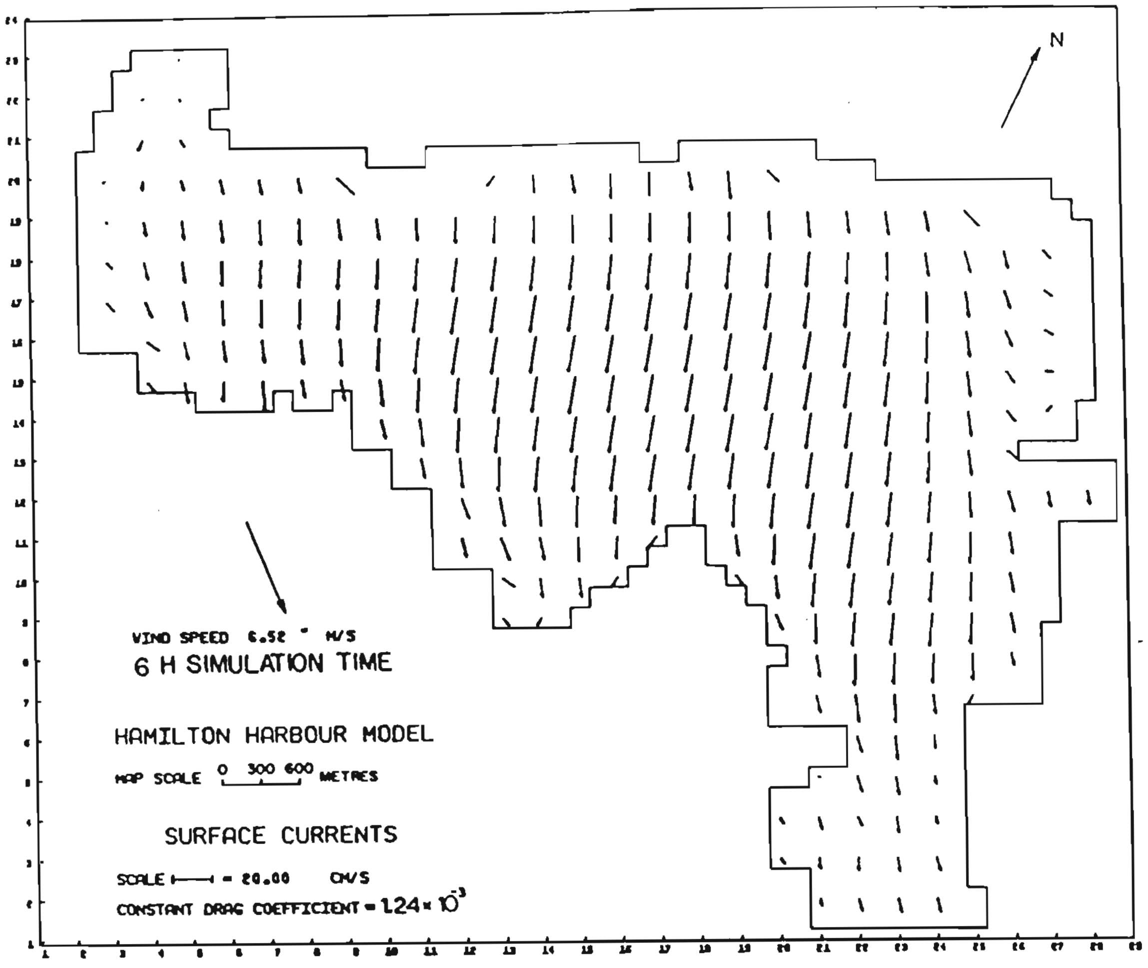

FIG. 5. Surface currents for constant $C_d = 1.24 \times 10^{-3}$.

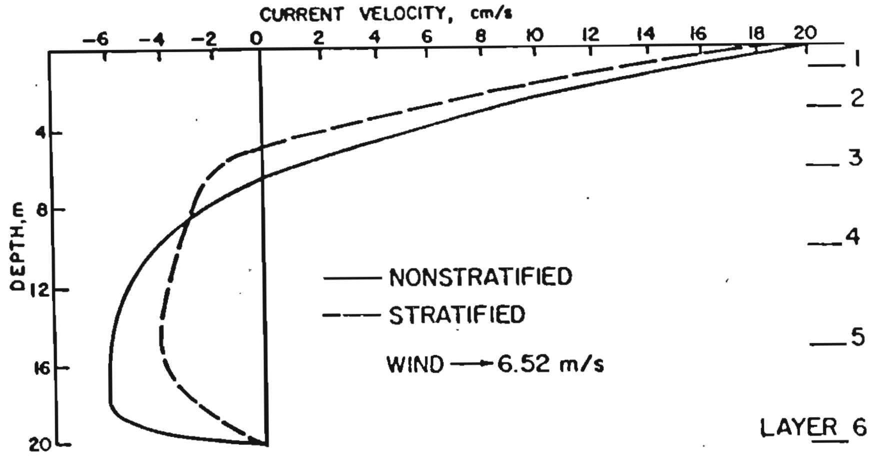

FIG. 6. Effect of temperature stratification on the velocity profile (at grid point (16, 16) after 3 h simulation time).

similar to that suggested by Bengtsson (1973) for small lakes subjected to low wind speeds. This preliminary run showed that the spatial variation of the drag coefficient has a small effect on the computed surface currents when compared to those computed using a constant drag equal to the average value (1.24×10^{-3}). Of course, this is not the authors' final conclusion as many numerical and

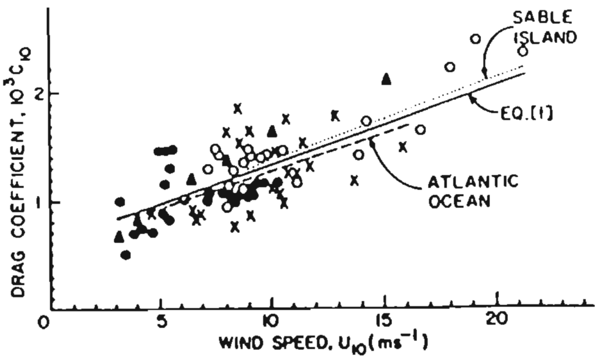

FIG. 7. Drag coefficient and wind speed: (X) Atlantic Ocean; (●) Lake Ontario; (○) Sable Island; (▲) our model for Hamilton Harbour (for northwesterly wind). Regression lines: Atlantic Ocean, dashed; Sable Island, dotted; all points, solid. (From Smith and Banke 1975.)

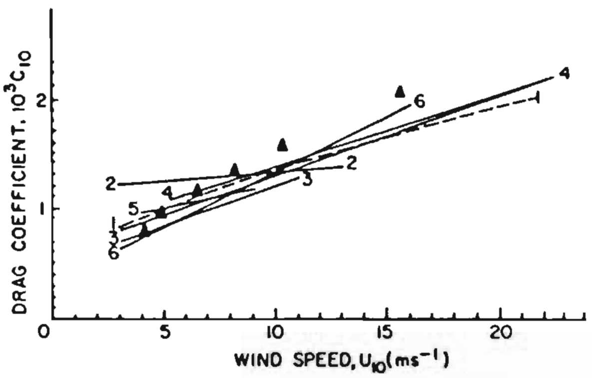

FIG. 8. Drag coefficient and wind speed correlation used in different studies (regression lines 1–6) (see Smith and Banke 1975); (▲) our model's results using wind profile method.

field experiments are still under study. However, this method may give a good rational basis for calculating the average drag coefficient as a function of wind speed, fetch, and wave characteristics.

The average drag coefficients computed for different wind speeds (northwest winds) ranging between 3 and 15 m/s may be compared to those obtained by Smith and Banke (1975). Smith and Banke applied a regression analysis on data collected through extensive observations on different water bodies, as shown in Fig. 7. The results were summarized by them in Fig. 8. As shown in Figs. 7 and 8, a good agreement between the authors' and the other results was obtained.

Conclusions

A three-dimensional time-dependent hydrodynamic model for lake transport has been developed for use in shallow stratified (or nonstratified) lakes. The model has been applied to a small nonstratified lake (James and Eid 1978) and a stratified lake (Hamilton Harbour).

The over-water wind stresses are determined using a wind profile method. In this method the logarithmic wind velocity distribution and Von Karman's integral equation for turbulent flow over a rough movable surface of variable roughness are used. Surface drag coefficients are calculated as a function of wind and wave characteristics (wind speed, wind fetch, and wave height and celerity). Spatial and temporal variation of the drag coefficient, and consequently surface shear stress, can be obtained for any wind condition. The drag coefficients calculated from the present model are smaller than those used typically by ocean modelers. However, they are in good agreement with those obtained by Smith and Banke 1975.

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In addition we wish to record with gratitude the encouragement of J. Simons, M. Donelan, and D. Lam of the Canada Centre for Inland Waters.

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Appendix: Wave Prediction in Deep and Shallow Waters

In this study we have used the significant wave method originally introduced by Sverdrup and Munk in 1947 and revised by Bretschneider in 1952 and 1958 with additional empirical data. This predictive relationship is now called the Sverdrup–Munk–Bretschneider (SMB) method (United States Army Coastal Engineering Research Center 1973).

This simplified wave prediction scheme is based on the assumption that the waves being considered are entirely due to a wind blowing at constant speed and direction for a specific fetch and duration.

SMB Method for Prediction of Waves in Deep Water

The significant wave height H and significant wave period T for given wind speed, fetch, and duration are given by:

$$[\text{A1}] \quad \frac{gH}{U^2} = 0.283\left[\tanh 0.0125\left(\frac{gF}{U^2}\right)^{0.42}\right]$$

$$[\text{A2}] \quad \frac{gT}{2\pi U^2} = 1.2\left[\tanh 0.077\left(\frac{gF}{U^2}\right)^{0.25}\right]$$

$$[\text{A3}] \quad \frac{gt}{U} = K\exp\left\{\left[A\left(\ln\left(\frac{gF}{U^2}\right)\right)^2 - B\ln\left(\frac{gF}{U^2}\right) + C\right]^{1/2} + D\ln\left(\frac{gF}{U^2}\right)\right\}$$

The above equations are dimensionally consistent; H = wave height; T = wave period (s); U = wind speed; F = wind fetch (the distance from the forecasting point to the opposite shore, measuring along the wind direction); t = minimum duration required to establish steady-state generation for a particular wind speed and length of fetch; $K = 6.5882$; $A = 0.0161$; $B = 0.3692$; $C = 2.2024$; $D = 0.8798$; and g = gravitational acceleration.

In order to use [A1] and [A2] correctly, the actual wind duration t_a should be more than or equal to the minimum duration. If $t_a < t$, the significant wave parameters are functions of wind speed and the limiting duration t, rather than functions of wind speed and fetch length. In this case calculate the corresponding fetch F from [A3], then use this fetch to calculate wave height and period from [A1] and [A2] (Lalande 1975).

For simplification, the wind is assumed to blow over the lake for a time long enough so that the wind durations are bigger than minimum durations required to satisfy the above equations. The above assumption seems reasonable especially when dealing with small lakes (fetch < 10 km) and for small wind speed (less than 10 m/s).

Wave Prediction for Shallow Water

If $d/T^2 < 2.5$ ft/s^2 (0.76 m/s^2), where d is the water depth, then the wave effectively 'feels the bottom', and the depth and bottom friction should enter as additional factors in the wave forecasting equations. Bretschneider found that the best agreement between wave data and numerical methods was obtained when a bottom friction factor of $f = 0.01$ was selected (Ippen 1966). This f is also called 'a calibration friction factor' which would take into account other influential factors not normally included in the friction factor term. The quasi-empirical–quasi-theoretical relationships established by Bretschneider in 1953 are used for wind–wave generation in shallow water with constant depth:

$$[\text{A4}] \quad \frac{gH}{U^2} = 0.283\left[\tanh 0.578\left(\frac{gd}{U^2}\right)^{0.75}\right] \times \tanh\frac{0.0125\left(\frac{gF}{U^2}\right)^{0.42}}{\tanh 0.578\left(\frac{gd}{U^2}\right)^{0.75}}$$

$$[\text{A5}] \quad \frac{gT}{U^2} = 1.20\left[\tanh 0.520\left(\frac{gd}{U^2}\right)^{0.375}\right] \times \tanh\frac{0.077\left(\frac{gF}{U^2}\right)^{0.25}}{\tanh 0.520\left(\frac{gd}{U^2}\right)^{0.375}}$$

The above equations can be extended to a bottom of constant slope or irregular bottom topography by segmenting the bottom into elements, each element having a mean depth assumed to be constant.

In the present numerical model wave characteristics (H, T) are first calculated for deep water (eqs. [A1], [A2]). If d/T^2 is greater than 2.5 ft/s^2 (0.76 m/s^2), then the computed wave height and wave period are correct; otherwise, apply [A4] and [A5] for shallow water wave prediction.

Then the procedure is to calculate wave clerity c:

[A6] $$c = L/T$$

The wave length L can be obtained from the implicit equation:

[A7] $$L = (gT^2/2\pi)\tanh(2\pi d/L)$$

for deep water:

$$\tanh(2\pi d/L) = 1$$

and

[A8] $$L_0 = gT^2/2\pi$$

Thus, we have a complete set of equations for estimating wave characteristics in both deep and shallow water. These can then be used in computing wind shear stress over the water surface in a lake as previously mentioned.

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Spatial distribution of the transient bed boundary condition for small lakes with weeds

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Spatial distribution of the transient bed boundary condition for small lakes with weeds

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Although there are a large number of studies on large lakes such as the Great Lakes, little work is being done on small lakes such as those that are used for recreational purposes and that abound in North America. Such water bodies often suffer from nutrient loadings and prolific weed growth; there appears to be a definite need for model development in this area. In this study, wind-driven currents in a shallow homogeneous lake with extensive weed growth have been modeled using the common hydrodynamic equations that describe lake circulation. The method has also been applied to a large stratified harbour. An approximate analytical procedure is presented for modeling the bottom roughness due to weed growth. As in meteorology, the Karman–Prandtl equation for the velocity distribution near the boundary of an open channel has been modified and applied in conjunction with empirical factors obtained from observed velocity profiles inside and above aquatic vegetation. Field experiments were carried out at Valens reservoir (a small recreational lake) and in Hamilton harbour. Surface currents, wind speeds and directions, and weed distributions were measured. Good agreement was found between the model results and observations. The model has especial application where spread of surface contaminants is a problem in lakes with heavy weed growth.

Alors que de nombreuses études ont été réalisées sur des lacs de grandes dimensions comme les Grands Lacs, peu de travail a été entrepris sur les petits lacs semblables à ceux qui servent à des fins récréatives et qui abondent en Amérique du Nord. De telles masses d'eau sont souvent affectées par la teneur en sels nutritifs et la croissance spectaculaire des plantes herbacées; il se dessine donc un besoin certain pour un modèle de simulation en ce domaine. Dans cet article, on établit un modèle de simulation des courants générés par le vent dans un lac de faible profondeur; on exploite alors les équations usuelles de l'hydrodynamique pour décrire la circulation dans le lac. La même approche a été appliquée à un port aux eaux stratifiées. Les auteurs exposent une méthode analytique approximative de simulation de la rugosité de fond due aux plantes. Comme on le fait en météorologie, on a modifié l'équation de Karman–Prandtl qui décrit la distribution des vitesses aux limites d'un canal à surface libre, pour ensuite l'appliquer en y introduisant des coefficients empiriques déduits des observations de profils de vitesses relevés dans et au-dessus de la zone de végétation aquatique. On a procédé, au réservoir Valens (petit lac à vocation récréative) et dans le port de Hamilton, à la mesure des courants de surface, des vitesses et des directions du vent et de la répartition des herbages. On a observé une bonne concordance entre ces observations et les résultats fournis par le modèle. Ce dernier trouve une application particulière dans le cas où des contaminants sont répandus à la surface de lacs qui sont le site d'une intense croissance de plantes herbacées.

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Introduction

In this paper we present a formulation of the boundary conditions at the bed, for a three-dimensional transport model. The approach is particularly suitable for shallow lakes with considerable roughness, and more especially for lakes with weed growth.

The boundary conditions (bed topography, shoreline configuration, and surface and bottom shear stress distribution) obviously more significantly influence circulation in small, shallow, and enclosed lakes (James and Eid, to be published).

The hydrodynamic equations commonly used to describe wind-induced lake circulation were used in this study. Flow is assumed to be incompressible and in hydrostatic equilbrium. The lake is considered to consist of a number of horizontal layers. The equations are integrated vertically over each layer and both vertical and horizontal eddy viscosities are introduced. The effect of temperature stratification has been neglected in this part of the study. For the bed boundary condition, the Karman–Prandtl logarithmic velocity distribution near the boundary has been modified to calculate bottom shear stresses. In this approach, the local bottom roughness has

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been taken to be related to local roughness parameters, e.g., weed length and density.

Model Hydrodynamics

The vertically integrated nonlinear equations of motion for a given layer in the two horizontal directions (x and y) are

$$[1] \quad \frac{\partial U}{\partial t} = fV - gH\frac{\partial \xi}{\partial x} + \frac{1}{\rho}(\tau_x^{(1)} - \tau_x^{(2)}) + A_h\left(\frac{\partial^2 U}{\partial x^2} + \frac{\partial^2 U}{\partial y^2}\right)$$

$$[2] \quad \frac{\partial V}{\partial t} = -fU - gH\frac{\partial \xi}{\partial y} + \frac{1}{\rho}(\tau_y^{(1)} - \tau_y^{(2)}) + A_h\left(\frac{\partial^2 V}{\partial x^2} + \frac{\partial^2 V}{\partial y^2}\right)$$

and the equation of continuity when integrated from the bottom to the water surface is

$$[3] \quad \frac{\partial \xi}{\partial t} = -\frac{\partial \bar{U}}{\partial x} - \frac{\partial \bar{V}}{\partial y}$$

where U and V are horizontal volume transport components in x and y directions for a layer

$$\left(\text{e.g., } U = \int_{z_1}^{z_2} u\,dz\right),$$

f is the Coriolis parameter (10^{-4} rad/s), g is the gravitational acceleration, h is the layer thickness, ξ is the fluctuation of the free-surface elevation from the still water level, ρ is the water density, A_h is the horizontal eddy viscosity coefficient, and τ_x and τ_y are the x and y components of shear stress. (1) and (2) denote the upper and lower interfaces of a layer such that: at the free surface,

$$\tau_x^{(1)} = \tau_{sx}; \quad \tau_y^{(1)} = \tau_{sy}$$

for interior interfaces,

$$\tau_x^{(1)} = \rho A_v \partial U/\partial z; \quad \tau_y^{(1)} = \rho A_v \partial V/\partial z$$

at the bottom,

$$\tau_x^{(2)} = \tau_{bx}; \quad \tau_y^{(2)} = \tau_{by}$$

where τ_{sx} and τ_{sy} are surface shear stress components in the x and y directions, τ_{bx} and τ_{by} are bed shear stress components, and A_v is the vertical eddy viscosity. Surface shear stress can be determined through the quadratic relationship:

$$\underline{\tau_s} = \rho_{air} C_d |W_a| \underline{W_a}$$

where C_d is the wind drag coefficient (2×10^{-5}), ρ_{air} is the air density, and W_a is wind speed at the anemometer level. The vertical eddy viscosity coefficient was calculated as a function of wind speed and lake depth through the equation $A_v = CHW$ (where $C = 2 \times 10^{-5}$, $H = 400$ cm, and W = wind speed (cm/s)). This relation is discussed elsewhere (Bengtsson 1973; Eid 1976). Bottom shear stress is discussed in more detail in the next section. $\bar{U}$ and $\bar{V}$ are total volume transport components:

$$\bar{U} = \int_{-H}^{\xi} u\,dz; \quad \bar{V} = \int_{-H}^{\xi} v\,dz$$

The above equations of motion and continuity are used to compute the three-dimensional wind-driven circulation for given initial and boundary conditions. These equations are solved numerically using finite differences in time and space. The numerical techniques are not discussed in this paper, but may be studied in many other publications (Eid 1976; Simons 1975).

Modeling of Bottom Roughness due to Weed Growth

The determination of flow within a vegetated shallow lake or coastal zone is a complicated problem, one that requires considerable research before the phenomena involved are completely understood. Very little, if anything, is known about how to calculate flow resistance inside and around aquatic weeds. The procedure outlined here may be considered as an initial attempt, to be modified as data become available. The approach is similar to that used to calculate flow resistance in vegetated open channels. Of course, flow in a lake is three-dimensional and hence more complicated.

Boundary layer theory (Prandtl 1904) has opened the way toward a rational understanding of some of the flow phenomena involving boundary roughness (Schlichting 1968). The application by G. H. Keulegan and others (American Society of Civil Engineers (ASCE) 1963) of Karman–Prandtl concepts and Nikuradse roughness to open channels has been quite successful in describing grain-type roughness in wide channels. However, it is not adequate for describing flow in vegetated channels. Recently, studies using flexible roughness strips attached to the bed of laboratory flumes were carried out (Kouwen *et al.* 1969; Kouwen and Unny 1973).

In meteorology, wind profiles inside and above vegetal cover (tall plants) have been studied in both laboratory and field (Cionco 1965; Pernir *et al.* 1972; Plate and Quraishi 1965). No investigations have been done to determine the velocity distribution inside and around an aquatic plant canopy. There exists a definite need for field measurements for the different types of aquatic weed commonly growing in shallow lakes. For example, no velocity or turbulence measurements are available at the top of a flexing plant canopy. Indeed, in view of the small velocities and probable large eddies in and around flexing weeds of varying length, such measurements would appear to be difficult to accomplish.

We propose a modification to the standard logarithmic boundary layer equation to account for the effect of weeds on the bottom. The parameter z' is used as a measure of the effects of the boundary on the profile and is not only a measure of the displacement from the true geometric boundary. We have calculated z' using a match of logarithmic and observed nondimensional profiles, and this same large value of z' is applied directly to the velocity profile. This may describe a turbulence field with relatively large scale eddies at the top of the weed field, which may accord with the actual mechanism, although such eddies would of course be slow-moving. Indeed we often observed such motions in preliminary tests with Rhodamine WT dye. However, in the absence of further evidence we cannot state that the eddies are *due* to the water movement over the plant canopy; we merely assert that we have observed large scale eddies with a vertical axial plane over the weed canopy and during typical wind-current test conditions.

A more justifiable approach to simulating the effect of the weed cover may be to determine the velocity at the top of the weed cover and to add to this a logarithmic velocity profile, with possible testing of the effect of adjusting the turbulence field at the interphase. But such work appears to be out of reach at present because of the low velocities, three-dimensional effects, and the difficult environment. Our approach has been to develop a simplistic bed boundary model based on recent meteorological–hydrodynamical literature, and to test the model in the field.

Methodology

Rouse (1965) summarized much of the reported work in open channel resistance using Karman–Prandtl concepts of turbulent flow near a rough boundary. Meteorologists have assumed that many of the parameters found to affect flow resistance in channels with rigid roughness could also be applied to flexible roughness such as that represented by grass and other vegetation (Plate and Quraishi 1965).

The velocity profile of a fluid moving near a rough boundary can be described by the logarithmic law:

$$[4] \qquad V/U_* = (1/k) \ln z/z'$$

where V is the velocity at distance z above the boundary (bed); U_* is the shear velocity, $(\tau_b/\rho)^{1/2}$; τ_b is the average boundary shear stress; k is Karman universal constant (= 0.4); z' is the magnitude of z at $v = 0$, i.e., the intercept of the velocity profile on the z-axis, as shown in Fig. 1, also known as roughness parameter (Rouse 1965; Sayre and Albertson 1961).

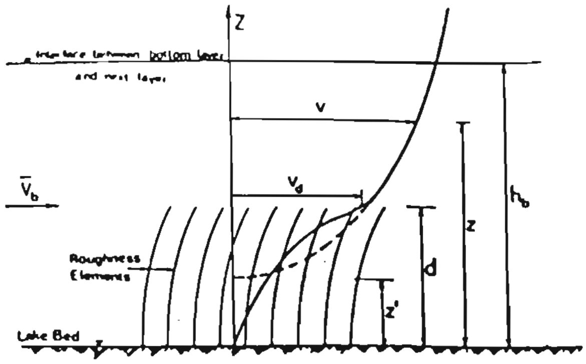

FIG. 1. Typical velocity profile inside and above plant canopy.

Plate and Quraishi (1965), and others (Cionco 1965; Pernir *et al.* 1972) have reported that the logarithmic velocity distribution can be applied at some distance above the plant cover. This distance is called 'zero plane displacement'. The velocity inside and around the plant cover cannot be defined by the logarithmic law. Derivation of an equation describing the velocity distribution both inside and above the weeds would require field and laboratory investigations (which were not available for this study). We have not dealt herein with the transverse velocity profile around locally high plants. We thus assume that flow within the submerged weeds is similar to meteorologic data for the velocity distribution of wind inside certain crops. These data have been summarised by Plate and Quraishi (1965). As the aquatic weeds generally formed a dense, flexible growth, the velocity distribution was assumed to have a shape similar to that for wind within wheat (the most flexible of the crops used) but with small velocities.

The approach used in this study is based on the assumption that the logarithmic law, [4], can be applied at a distance z' inside the weed canopy. This distance is calculated such that the area under the logarithmic velocity profile inside the weeds (from z' to d) is equal to that given by the wind data, as shown in Fig. 2. In other words, the discharge resulting from integrating the logarithmic velocity distribution (from z' to the weeds' height d) is set equal to the discharge (within the plants) given by the empirical data. This yields an equivalent sublayer of zero discharge with depth z', denoted the 'boundary layer displacement thickness'. This is equivalent to an imaginary new solid boundary, over which the

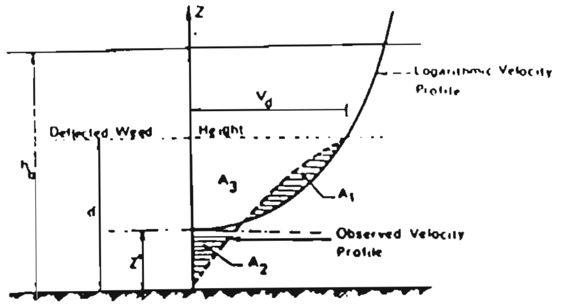

FIG. 2. Empirical and theoretical velocity distributions inside the canopy.

logarithmic velocity distribution is applied. Now, applying [4] at the tip of the vegetation,

$$[5] \qquad V_d/U_* = (1/k)\ln d/z'$$

where d is the deflected height of the weeds as shown in Figs. 1 and 2. z' can be determined from

$$[6] \qquad A_1 + A_3 = A_2 + A_3$$

where

$$A_1 + A_3 = \int_{z'}^{d} v\,dz = \frac{U_*}{k}\int_{z'}^{d}\ln\frac{z}{z'}\,dz$$

Thus

$$[7] \quad A_1 + A_3 = (U_*/k)[d(\ln d/z') - (d - z')]$$

A nondimensional presentation of field data for wind profiles inside plant cover (wheat and corn) obtained by Tan and Ling together with the wind tunnel results obtained by Plate and Quraishi (1965) are given in Fig. 3.

Assuming the dimensionless area under the observed profile (Fig. 3) is A, then

$$[8] \qquad A_2 + A_3 = AdV_d = (AdU_*/k)\ln d/z'$$

From equations [7] and [8], [6] becomes

$$[9] \qquad (1 - z'/d) - (1 - A)\ln d/z' = 0$$

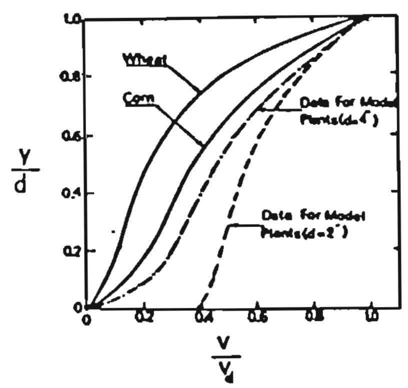

FIG. 3. Field and experimental velocity profiles in canopy (from Plate and Quraishi 1965).

For a given velocity distribution inside the plant cover, the boundary layer displacement thickness z' can be obtained from the dimensionless equation [9].

It is evident from [9] that the boundary layer displacement thickness z' depends on the local roughness conditions, i.e., size, shape, and spacing of the roughness elements. In other words, z' should completely describe the boundary roughness, as in Sayre and Albertson (1961). It will include the effect of turbulence at the interface.

Rewrite [9] with $z'/d = c$ as follows:

$$[10] \qquad (1 - c) - (1 - A)\ln 1/c = 0$$

By solving [10], c is obtained as a function of the normalized area A, which in turn depends on the

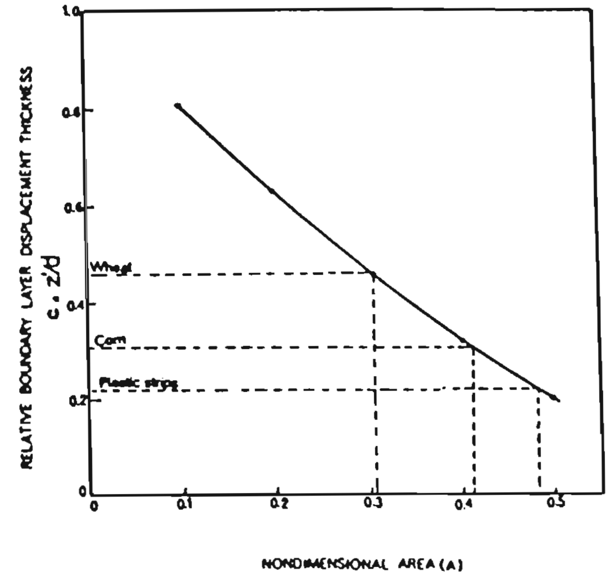

FIG. 4. Determination of boundary layer displacement thickness.

type of vegetation and its density. This relation is represented in Fig. 4. Then, once the velocity distribution inside the weeds is known, the roughness parameter ($z' = cd$) can be obtained from Fig. 4. Substituting back into [4] we can calculate the velocity distribution above the boundary layer displacement thickness in the bottom layer.

Calculation of Bottom Shear Stress (τ_b)

For the flow in the bottom layer, the discharge per unit width is

$$[11] \qquad V_b(h_b - z') = \int_{z'}^{h_b} v\,dz$$

Substituting [4] into the right hand side of [11] and integrating gives

$$[12] \quad \frac{\bar{V}_b}{U_*} = \frac{1}{k}\frac{h_b}{h_b - z'}\ln\frac{h_b}{d} + \frac{1}{k}\left[\frac{h_b}{h_b - z'} \times \ln\frac{d}{z'} - 1\right]$$

where, as shown in Fig. 1, $\bar{V}_b(\bar{u}_b, \bar{v}_b)$ is the average velocity in the bottom layer, h_b is the depth of the bottom layer, and z' is the boundary layer displacement thickness ($= cd$). Equation [12] can be rewritten in the simple form

$$U_* = (1/c_2)\bar{V}_b$$

Thus

$$\tau_b/\rho = (1/c_2)^2 \bar{V}_b^2$$

$$[13] \quad \tau_b/\rho = K\bar{V}_b^2$$

where

$$c_2 = 2.5\frac{h_b}{h_b - z'}\left(\ln\frac{h_b}{d}\right) + 2.5\left[\frac{h_b}{h_b - z'}\left(\ln\frac{d}{z'}\right) - 1\right]$$

and $K = (1/c_2)^2$ is called (Simons 1973) the nondimensional skin friction coefficient. In a study on Lake Ontario (Simons 1975), it was assumed to be a constant of the order of 2.5×10^{-3}. In the present study K is assumed to have the same value (2.5×10^{-3}) for the case of no weeds. It is convenient to rewrite [13] in a vectorial form:

$$[14] \quad \underline{\tau_b} = \rho K|\bar{V}_b|\underline{\bar{V}_b}$$

The shear stress components in x and y directions are calculated from

$$[15a] \quad \tau_{bx} = \rho K|\bar{u}_b|\bar{u}_b$$

$$[15b] \quad \tau_{by} = \rho V|\bar{v}_b|\bar{v}_b$$

Substituting back into the equations of motion [1] and [2] yields the complete set of equations to be used for calculating wind-driven currents in shallow lakes with prolific bed weeds. These equations are solved using a finite differences technique in time and space.

Field Observations

To verify the computer model, field observations were carried out in Valens reservoir (about 185 acres (75 ha) with a maximum depth of 4 m). Surface drift velocities were measured, using drogues, for various wind and bottom roughness conditions throughout two summers (1974 and 1975). Weed distributions throughout the lake were measured regularly. Wind speeds and directions and water levels were continuously recorded. These observations were used to check the validity of the computer model (Eid 1976). The actual computer model will be described in a further paper now under preparation. A summary of field observations is presented in Table 1. The model has also been applied to the Hamilton Harbour Study where the physiochemical processes are much influenced by wind generated motion (Palmer and Poulton 1976; James and Eid, to be published).

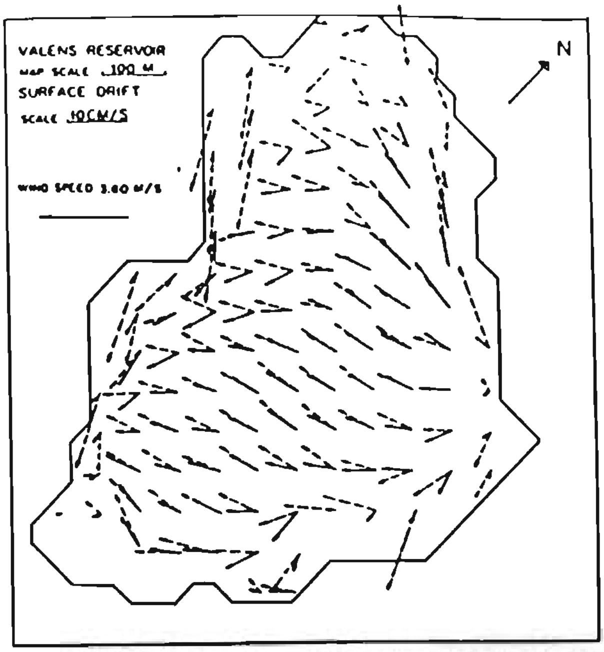

FIG. 5. Effect of weeds on surface currents: with weeds, solid lines; without weeds, dashed lines. Vertical eddy viscosity = 5 cm^2/s.

Analysis of the Results

The computer model was run for all valid observed wind events for two cases: (a) with the existence of bed weeds, and (b) without weeds, in which the nonlinear bottom roughness coefficient was assumed to be constant all over the lake and of the order of 2.5×10^{-3}. Samples of the model results are shown in Figs. 5–7. Figure 5 shows the computer plots comparing surface current patterns for the two cases of bottom roughness. Solid vectors show results if weed growth and roughness distribution is modelled. It is clear that bottom friction stresses due to weeds generally affect the flow in both magnitude and direction. This influence depends on the stage of weed growth. Modeling weed growth might decrease the computed surface velocities. This reduction may be as high as 20%, but in shallow parts the reduction is greater because extensive weeds cause a considerable retardance of the flow. The effect of bottom roughness on the velocity profiles is shown in Figs. 6 and 7. The velocity profile resulting from the assumption that the bottom roughness is areally constant was compared with that obtained when

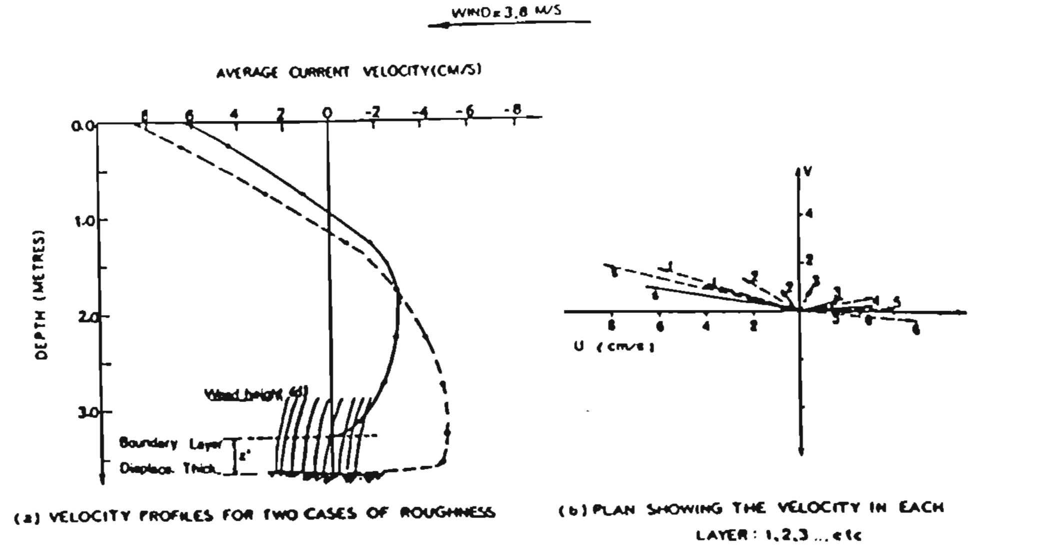

FIG. 6. Effect of bed weeds on velocity profiles: with weeds, solid lines; without weeds, dashed lines. $A_v = 5.0\ cm^2/s$.

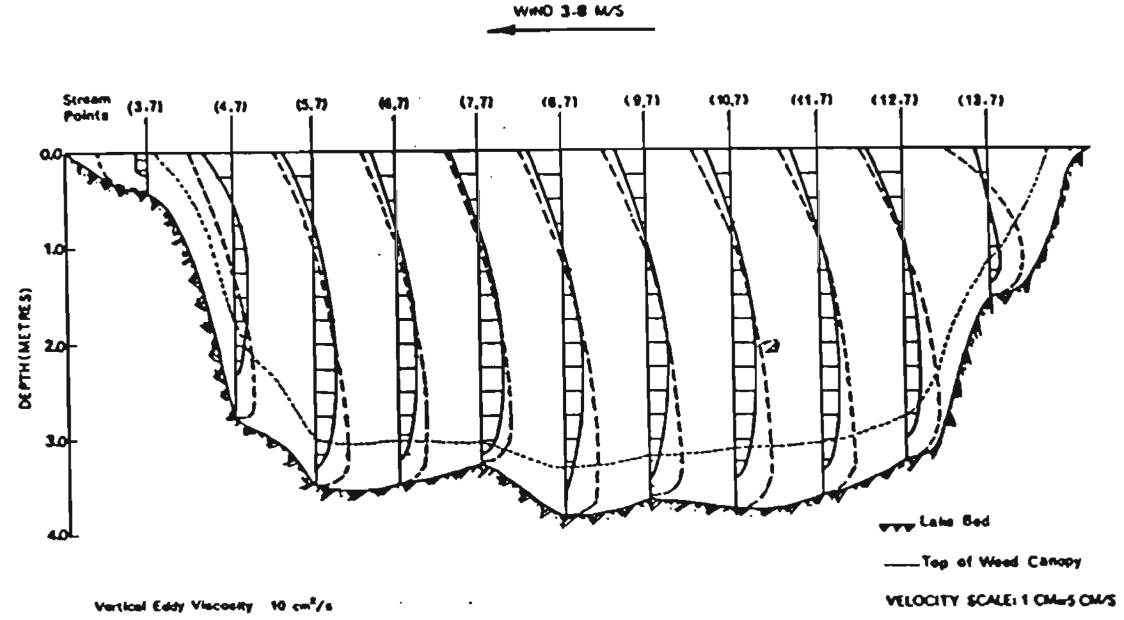

FIG. 7. Schematic cross section through Valens Lake and velocity profiles at stream points: with weeds, solid lines; without weeds, dashed lines.

computing local bottom roughness due to weed growth. It was found that bottom stresses due to weeds cause a reduction in both surface and return currents. The reduction of the return flow near the bottom is much bigger than that of the flow near the surface. A comparison was made between the observed and computed average surface drifts for areal strips. The results are shown in Table 1.

Conclusion

Bottom roughness due to extensive weed growth in shallow lakes can be modeled using the formulation developed herein. A bottom stress equation has been formulated to incorporate the effect of weed height and distribution throughout the lake. The Karman–Prandtl logarithmic law for the velocity distribution at the boundary of vegetated channels was suitably adapted. This formulation for bottom stresses can be conveniently incorporated into a three-dimensional transport model for wind-driven lake circulation. It appears to account for the observed flow pattern.

In shallow lakes with prolific weed growth, the boundary condition at the bed is very important. Bottom shear stresses due to weeds greatly affect the

TABLE 1. Computed and observed surface currents in Valens reservoir with weed growth

Survey number	Date	Wind speed (m/s)	Wind direction (deg. from North)	V_{obs} (cm/s)	V_{comp} (cm/s)	$\frac{V_{obs}}{V_{comp}}$
1	June 5, 1974	4.0	200 (S-W)	9.0	8.68	1.04
2	June 25, 1974	3.8	45 (N-E)	6.3	7.8	0.81
3	June 26, 1974	3.6	90 (E)	5.91	5.7	1.05
4	July 30, 1974	4.1	245 (S-W)	7.46	7.26	1.03
5	May 21, 1975	4.25	245 (S-W)	8.2	8.66	0.95
6	June 16, 1975	4.9	250 (S-W)	8.0	8.9	0.9
7	June 26, 1975	4.36	80 (N-E)	7.5	7.87	0.45
8	July 2, 1975	3.6	300 (N-W)	8.0	7.33	1.09
9	July 22, 1975	3.87	225 (S-W)	9.0	8.2	1.1
10	July 28, 1975	3.4	270 (W)	7.2	7.1	1.01
11	July 30, 1975	3.55	225 (S-W)	7.57	7.06	1.07
12	August 11, 1975	2.88	225 (S-W)	5.6	5.7	0.98
13	August 18, 1975	3.58	300 (N-W)	8.4	8.7	0.97
14	August 26, 1975	2.86	270 (W)	7.57	6.55	1.15

currents in both magnitude and direction. This influence depends on the stage of weed growth and its distribution over the lake bed. It causes a reduction of flow velocities (both surface and return current). The effect of bottom roughness on the current in shallow zones is much greater than that in deeper parts, as is to be expected.

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1983 International Symposium on Urban Hydrology, Hydraulics and Sediment Control
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STORM DYNAMICS MODEL FOR URBAN RUNOFF

By

Wm. James and R. Scheckenberger

ABSTRACT

Conventional municipal storm sewer design procedures have never accounted for the spatial variability and dynamics of storms. Present methodology uses complex hydrological program packages, but considers storms to be simple, even static events, which uniformly cover the drainage sub-areas under consideration. This simplification of real storm processes probably results from the paucity of rain data that engineers have conventionally had to deal with.

Consequently hydrological models are normally calibrated and validated using hyetographs measured by very few point raingauges located in or around the drainage area. Single, isolated point raingauges cannot provide information on storm dynamics and spatial variability. In fact thunderstorm cells frequently miss a raingauge entirely. Without proper quantification of causative rainfall, runoff models will not provide accurate estimates of resulting runoff and pollutant levels. For design inference, engineers then use composite, statistically derived design storms (based on historical records of point raingauges) as input to a model calibrated by poorly represented rainfall. This obviously reduces the reliability of the design.

This paper proposes the use of RAINPAK, a package of computer programs, to overcome these problems. Using rainfall data from a network of at least 3 raingauges, STOVEL estimates the speed, direction and growth/decay characteristics of a storm cell as it tracks across a catchment. This data is then input to the next program, THOR4D, which produces separate, unique hyetographs for each subcatchment. By discretizing the storm spatially over the subcatchment spaces and temporally over the hydrological integration time-step, THOR4D simulates the variable speed, variable direction, and growth/decay of spatially-limited, multi-cellular storm events. THOR4D generates hyetographs for calibrating the quality and runoff routines in the subsequent hydrological program package. THOR3D is a simpler version of THOR4D; it has only 2 spatial dimensions and approximates cells as line-storms similar to squall-lines. THOR3D also determines critical storm properties.

INTRODUCTION

Hydrological computer modelling of a drainage system requires careful discretization. The engineering hydrologist must identify important physical characteristics and properties of the system and then incorporate appropriate coding for these into his model. The required level of discretization is under constant debate and is normally considered a function of the cost of the project, amount of allocated or available resources, and available time.

Two basic types of data are required for the development of deterministic hydrological models. The first type defines the physical environment: subcatchment areas, percent impervious area, slope, infiltration rates, pipe dimensions etc. The second type is the hydro-meteorological input time series: rainfall hyetographs, potential-evapotranspiration, wind speeds, wind directions, temperatures, etc. The present paper focuses on the level of discretization and other important properties of the second type of data, specifically the input rainfall time series, or hyetograph.

Present design methodology does not account for the spatial and kinematic characteristics of rainstorms. Some rainstorms, particularly those of the MESO-BETA scale (2.0-20.0 Km breadth), tend to display rapid spatial variations in rainfall intensities as well as significant variations in the speed of cellular travel (Rogers, 1; McAnelly, 2; Maddox, 3). These properties are typical of high intensity, short duration summer thunderstorms. These summer thunderstorms often contribute to urban flooding, combined sewer overflows and highly polluted storm runoff events. By incorporating the general properties of thunderstorms into a practical design methodology, it will be possible to design safer, more efficient hydrological systems. The estimates of pollutant loadings in continuous modelling will also be improved through the inclusion of these storm properties. Finally, the process of model calibration, so important in hydrological simulation, will be improved and perhaps made easier by adopting such methodology (James and Robinson, 4).

BACKGROUND REVIEW

A design storm is supposed to provide an estimate of the hydrograph or pollutograph of specific recurrence interval, for a given catchment, for design or planning requirements. The development of a synthetic hyetograph for drainage system design may be based on IDF relationships, historic storms or engineering judgement. Important examples of this methodology include the Chicago Design Storm and the quartile hyetograph developed by the Illinois State Water Survey.

The Chicago synthetic hyetograph has been widely adopted in North America because of its relatively easy determination through the use of IDF relationships. This methodology takes into account the maximum rainfalls of individual durations, and the average amount of rainfall antecedent to the peak intensity. The intensity distributions on either side of the peak are obtained from IDF relationships using constants determined from local data. Different constants are derived for each storm duration.

The Illinois State Water Survey design storm preserves quartile rainfall distributions. Maximum hourly rainfall depths are obtained from observed local data or IDF relationships for a variety of return periods. The historical storms are then subdivided into a number of groups depending on the relative time of occurrence of the peak intensity. The time distributions are determined for the group of storms and their median distribution is used for the design storm (Huff, 5).

In our method, a package of four programs, RAINPAK (STOVEL, THOR4DPT, THOR4D, and THOR3D) has been developed to analyse point rainfall hyetograph records and produce a 3-dimensional numerical representation of the recorded storm event. This kinematic rainfall is then input to a hydrological program such as the SWMM. The method is superior for calibration and validation of rain-runoff models, and can also be included in the design storm method, although we caution against the design storm approach and recommend a continuous modelling approach.

Importance of Thunderstorms

All RAINPAK programs relate to thunderstorm-type rainfall events. Thunderstorms have very specific characteristics which are important for storm water management. In most North American communities, thunderstorms cause the most frequent and severe urban flooding. This is largely due to characteristic high rainfall intensities and volumes over a short duration, usually less than one hour. Water quality is also significantly affected by thunderstorms, since thunderstorms are usually preceded by hot dry spells of weather during which urban pollutants build up. The intense rainfall is usually strong enough to washoff the accumulated pollutants. Even in combined sewer systems, due to drainage system overloading during the thunderstorm, very little of these pollutants find their way to treatment facilities; most are directed by overflow structures to receiving waters, often near the urban population concentrations.

Characteristics of Thunderstorm Cells

Thunderstorms exhibit single and multi-cellular structures. Each cell is in itself a mini-storm produced from moist convective air parcels. Thunderstorm cells have been observed by radar and satellites to be approximately circular or oval in shape. They are normally shape-preserving throughout their life cycle and typically exhibit the areal intensity pattern shown in Figure 1. The most intense rainfall occurs almost without exception in the centre of the cell. As the cell develops, the intensity distribution shown in Figure 1 varies in intensity and slightly in areal dimension.

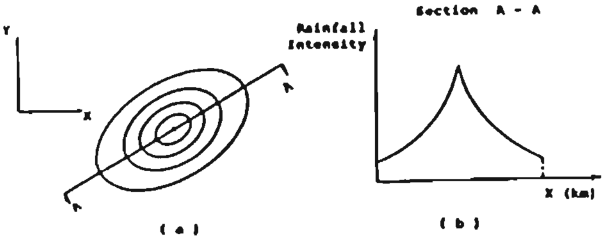

Figure 1. Typical thunderstorm cell cross-section

Cells experience three basic life stages: growth, maturity and death. The entire life cycle lasts approximately 30 minutes; each stage lasts about 10 minutes (the death stage can occassionally last considerably longer, up to 30 minutes). Each stage has recognizable features which can be observed on a point-hyetograph. During the growth stage, the cell grows in size and moisture content. Rainfall during this stage is of low intensity and short duration (eg. 5 min). The next stage, maturity, exhibits rapid fluctuations in updraft velocities as well as sudden intense downpours. Again depending on cell size, this stage can last from 5 to 20 minutes and always causes the most severe rainfall. The final, death stage is the 'rainout' period when the source of moist air is cut off and all that remains are isolated pockets of wet air. The rainfall rates during this stage fluctuate rapidly from medium intensities to nothing. Actual rainout, depending on cell size, can last from 10 to 30 minutes. Figure 2 indicates a hyetograph produced by a typical mature thunderstorm cell (Drufuca and Rogers, 6).

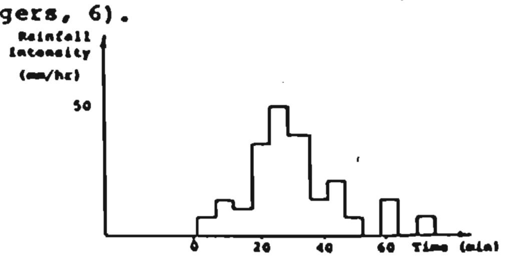

Figure 2. Typical mature thunderstorm cell hyetograph

As mentioned at the outset, thunderstorms can be multicellular and hence more than one cell can pass over a raingauge during a storm event. Cells can merge with each other so that the leading edge of a trailing cell can overlap the trailing edge of a leading cell. In this case the start/end of a cell is indistinguishable on a point hyetograph record. The peak intensities of each respective cell remains largely undisturbed by this merging effect. Figure 3 depicts a hyetograph for merged thunderstorm cells.

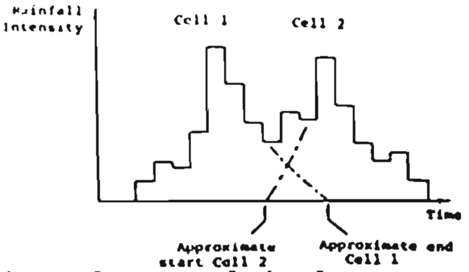

Figure 3. Merged thunderstorm cell hyetograph

RAINPAK

STOVEL

STOVEL (STOrm VELocity) is the first program in RAINPAK. Based on point hyetographs, STOVEL determines a storm velocity vector, consisting of a direction and speed. The velocity vector is necessary for further analysis of the cell motion and the growth and decay mechanisms. STOVEL will usually be used in conjuction with one or more of the RAINPAK programs.

The observed peak rain intensity at a point gauging site is closely correlated with the observed volume of rainfall at that site for individual storm events. This relationship is considered strong enough to form one of the basic concepts used in STOVEL.

Since the thunderstorm cell structure is basically shape-preserving (especially over short distances), the direction and speed can be estimated from the relative time of peak between any three gauging station locations.

The minumum number of raingauge hyetographs required for STOVEL is 3. The direction and speed of a storm cell is dermined from the Cartesian co-ordinates of the three raingauges and the relative time-of-peak. If more than 3 gauges are available, every combination of three raingauges from the total number of raingauges is chosen (455 for the maximum number of gauges, 15). When more than one combination is possible an overall arithmetic and weighted average of the storm cell velocity vectors is determined. The weighted average is based on the distance separating raingauges, or the perimeter of the triangle formed by a triad of gauges.

Growth-Decay Mechanism

The arithmetic and weighted average velocity vectors are used for two separate growth and decay analyses. Using the newly determined average vectors, the relative time-of-peak at each gauge site is re-calculated. A non-linear curve is fitted to the computed and observed time-of-peaks. This curve is in the form of a parabolic-exponential:

```
PEAK = EXP(A + B1 * ln(t) + B2 * ln(t**2)
where t = time
      PEAK = peak rainfall intensity
      A,B1,B2 = constants of determination
```

The most intense rainfall is assumed to fall in the centre of the storm cell; a typical cell foot-print is shown in Figure 4. The track of a storm cell centre-line is determined by a least squares fit of the distance perpendicular to a possible centre-line from a gauge location, weighted by the peak intensity at that gauge site.

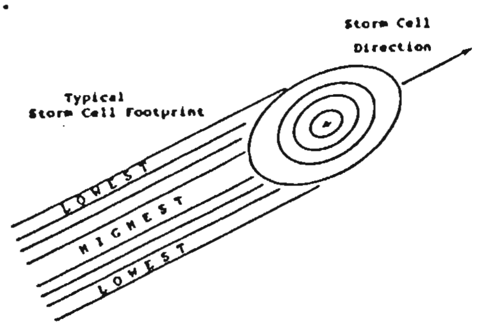

Figure 4. Ground-based rainfall accumulations and intensities

THOR4DPT and THOR4D

These programs use the output directly from STOVEL requiring only minimal additional data, mainly user control data (ie. hyetograph timestep, length of hyetograph, number of cells). The two programs are virtually identical, the major difference being that THOR4DPT produces an instanteneous hyetograph at a point (assumed to be a gauging site) whereas THOR4D produces a time and spatially averaged hyetograph for a subcatchment. In a standard RAINPAK application the THOR4DPT program would be the first to follow STOVEL. Through a variety of numerical techniques, THOR4DPT simulates rainfall at the active gauging stations. The user must then compare the simulated and observed hyetographs, focusing on key objective functions such as:

(i) total precipitation,
(ii) peak rainfall intensity,
(iii) shape (timing of peak),
(iv) duration.

Several parameters can be used to calibrate the storm so that simulated hyetographs better fit the observed hyetographs. A perfect fit for all gauges will not be possible, since THOR4DPT merely fits the best possible circular or elliptical exponential cone to the observed data. This approach will smooth out the stochastic noise, inherent in field measurements. The user subjectively determines when a simulated storm cell best fits observed data. Once this fit has been adequately determined, the parameters determined by THOR4DPT are input to THOR4D.

The basic assumptions behind THOR4DPT are:

(i) storm cells are either circular or elliptical,
(ii) the cell is assumed to remain circular or elliptical throughout its life cycle (however its dimensions may change with time),
(iii) the peak rainfall intensity is located at the centre of the specified shape,
(iv) the rainfall distribution varies exponentially away from the centre, decreasing to a user specified intensity at the cell boundary,
(v) storm events can consist of more than one cell, and these cells can merge and overlap.

Using these concepts, a storm system can be created which exhibits virtually all the characteristics of a real storm, except stochasticity. The input data required to execute THOR4DPT can be divided into two basic categories: meteorologic data describing the cell, and user control data. The latter specifies the duration in minutes of the hyetograph and timestep used in the hyetograph. It is imperative that the user specify enough time for the cell to reach the gauges from its assumed position at the start time. The meteorologic input data includes the following parameters:

(i) peak intensity,
(ii) boundary intensity,
(iii) cartesian co-ordinates at start time of cell centre,
(iv) speed,
(v) direction,
(vi) shape and dimensions,
(vii) growth/decay function.

The dimension of the cell parallel to the storm cells' motion can be obtained from the hyetograph record. For a given cell, the hyetograph observed at any number of raingauges will have approximately the same duration (exceptions include gauges located on the periphery of a storm cell). Using this duration and the storm cell speed, the longitudinal dimension is estimated by:

Cell Dimension=(Mean Hyetograph Duration) *(Cell Speed)

THOR3D.

THOR3D, is useful in design applications as well as preliminary examination of drainage system response to dynamic rainfall. THOR3D has one less spatial dimension than THOR4D; rather than simulating an elliptical cell, THOR3D simulates a line storm which has an infinite dimension transverse to its motion. THOR3D also requires two basic types of data: subcatchment data and storm data.

The subcatchment data required depends on the areal centroid of a subcatchment and the weighted subcatchment widths in the N-S, NE-SW, E-W and SE-NW directions passing through the subcatchment centroid. The outermost boundary, internally determined from user input, represents the starting line for the storm. Additional subcatchment data required are the weighted subcatchment widths, four for each subcatchment. This distance is taken through the centroid. The data for each subcatchment numerically represent their physical size and configuration within the catchment area. The storm characteristic data describes the temporal distribution of rainfall associated with the storm. Model input parameters required include:

(i) total precipitation,
(ii) storm speed,
(iii) storm direction,
(iv) time of rising limb: time of receding limb,
(v) initial 1-min intensity: peak 1-min intensity,
(vi) time-step;

and one of either:

(vii) one-minute peak intensity, or
(viii) time of rising hyetograph limb

These hyetograph parameters can be variously determined depending primarily on the application of the rainfall model. The number of parameters available allow the user to calibrate the rainfall model to match an observed event using data from either raingauges or radar. Many alternative approaches exist and at present various applications are being examined.

THOR3D converts the instantaneous-point hyetograph into a time and areally averaged hyetograph. Based on the time required to cross an individual catchment, the effective size of the cell in the line of motion is considered.

At present THOR3D writes the hyetograph into SWMM (Stormwater Management Model) compatible format.

RAINPAK APPLICATION

RAINPAK has been applied to a previously tested and calibrated hydrological model in the Chedoke Creek, Hamilton, Ontario (see Figure 5) (Robinson and James, 7).

The objective functions tested included:

(i) peak discharge,
(ii) time of peak discharge,
(iii) volume of flow,
(iv) surcharge volumes,
(v) peak suspended solids concentration.

The sensitivity of the hydrological model to the following rainfall model parameters was tested:

(i) direction of storm,
(ii) speed of storm,
(iii) hyetograph timestep,
(iv) hyetograph shape (time-base),
(vi) stationary model vs. moving storm model.

The storm used as a standard for sensitivity testing by THOR3D is considered to be

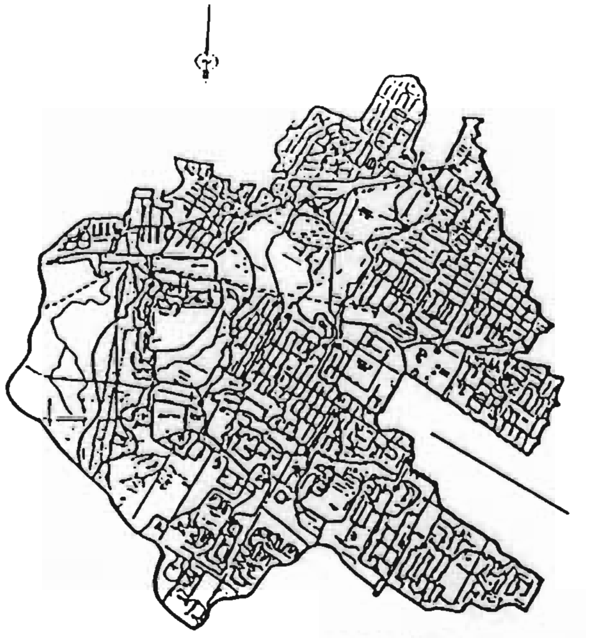

Figure 5. Chedoke Creek Tributary Area, Hamilton, Ontario, Canada; Divided into 24 subcatchments

representative of a summer thunderstorm in Southern Ontario and has the characteristics shown in Table 1. The resultant one-minute peak of 3.78 in./hr is typical. By varying one of the meteorological parameters within their most expected value, and keeping all others constant, one can estimate the rainfall model sensitivity as well as the catchment model sensitivity to perturbations of that parameter. The results of this analysis are tabulated in Tables 2 to 6.

Table 1. Meteorological Characteristics of Standard Storm

Total Precipitation	1.0"
Storm Speed	10km/hr
Storm Direction	from SW
Time of Rising Hyetograph Limb	20 min
"Resultant" One Minute Peak	3.78"/hr
Time of Rising Limb: Time of Falling Limb	1:1.5
Percent of Initial Intensity to Peak	5%
Timestep and Averaging Period	5 min

Direction

Results for storm direction sensitivity indicate the importance of storm track orientation in an urban environment. The Chedoke Creek drainage pattern runs primarily in a SW to NE direction, the prevailing storm direction in Hamilton. In fact, a storm from the north produced a peak discharge of 250 cfs less than that from the southwest. (See hydrograph, Figure 6). Even more dramatic are the peak surcharge volumes in two of the most important pipes in the catchment. The maximum surcharge is 174,000 cu. ft. for SW; and a minimum of zero for North storms.

Table 2. Direction Comparison

Test Type	Inches Over Basin	Total Flow at Inlets, (cfs x 10^7)	Hydro Substation Peak Discharge (cfs)	Time to Peak (hr:min)	Peak SS Concentration (mg/l)	Max. Surcharge Vol Gutter no. (ft^3) 7	10
Standard Storm from SW	1.05	.999	1214	0:50	947	239,000	174,000
Storm from West	1.05	1.009	1157	0:50	1099	110,000	119,000
Storm from NW	1.05	1.002	1023	0:55	1300	95,000	21,000
Storm from North	1.05	.990	944	1:05	1172	140,000	-
Storm from NE	1.05	.989	1002	1:10	1089	129,000	5,000
Storm from East	1.05	.985	1090	1:10	1145	144,000	66,000
Storm from SE	1.05	1.021	1145	1:10	1070	242,000	104,000
Storm from South	1.05	1.009	1170	1:00	1009	275,000	120,000

Table 3. Storm Speed Comparison

Test Type	Inches Over Basin	Total Flow at Inlets, (cfs x 10^7)	Hydro Substation Peak Discharge (cfs)	Time to Peak (hr:min)	Peak SS Concentration (mg/l)	Max. Surcharge Vol Gutter no. (ft^3) 7	10
Standard 10 km/h	1.05	.999	1214	0:50	947	239,000	174,000
5 km/h Storm	1.05	.994	1077	1:05	750	76,000	49,000
20 km/h Storm	1.04	1.036	1220	0:45	1049	207,000	172,000
40 km/h Storm	1.04	1.021	1194	0:45	1057	277,000	140,000
80 km/h Storm	1.03	1.001	1163	0:45	1001	226,000	112,000

Speed

If the direction is kept constant (say Southwest) and the storm speed is varied, the influence of speed can be examined. In this study, time of rise is kept constant even when speed is changed; only storm travel time through the catchment was varied. An extremely fast moving storm, such as the 80 km storm in Table 3, has the same effect as a stationary storm, since there is little or no lagging effect and the hyetographs are almost the same for all subcatchments. The 5 and 20 km/hr storms produce consistent results. The slower storm provides a lower peak discharge, ten minutes after that of the standard 10 km/hr storm (see hydrograph, Figure 7). Maximum surcharge volumes are also about one-third those of the standard, which is reasonable since stormwater on the upper catchments runs off before the major part of the storm reaches the lower catchments near the outfall. Total volume of flow is also decreased slightly due to added infiltration. The faster 20 km/hr storm produces a contrasting result - higher peak discharge, ten minutes before that of the standard, and surcharge volumes that are comparable to the standard. This again is to be expected: a critical storm speed will exist for a catchment when storm travel time is comparable to the characteristic catchment response time.

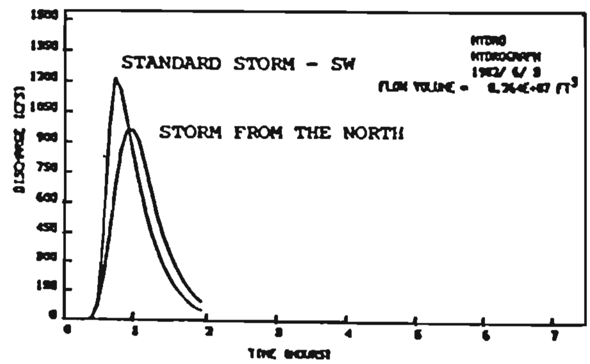

Figure 6. Directional comparison

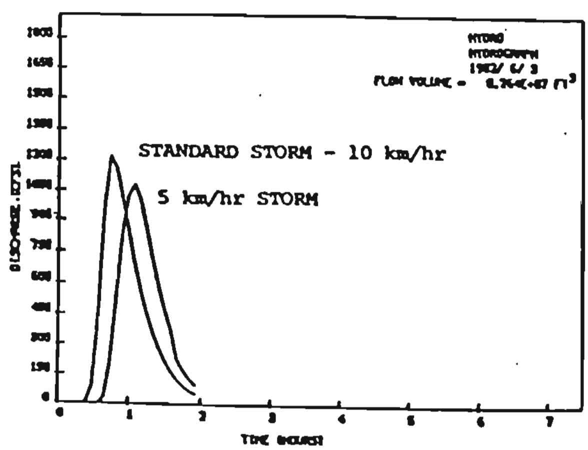

Figure 7. Speed comparison

Table 4. Timestep Comparison

Test Type	Inches Over Basin	Total Flow at Inlets, (cfs × 10^7)	Hydro Substation: Peak Discharge (cfs)	Hydro Substation: Time to Peak (hr:min)	Hydro Substation: Peak SS Concentration (mg/l)	Max. Surcharge Vol Gutter no. (ft^3): 7	Max. Surcharge Vol Gutter no. (ft^3): 36
Standard 5 min.	1.05	.999	1216	0:50	947	219,000	174,000
One Minute Hyetograph	1.05	1.068	1247	0:50	951	242,000	192,000
Ten Minute Hyetograph	1.05	.990	1124	0:55	955	106,000	125,000
Thirty Minute Hyetograph	1.05	.851	779	1:05	800	-	-
Sixty Minute Hyetograph	1.01	.659	515	1:10	769	-	-

Table 5. Storm Timebase Comparison

Test Type	Inches Over Basin	Total Flow at Inlets, (cfs × 10^7)	Hydro Substation: Peak Discharge (cfs)	Hydro Substation: Time to Peak (hr:min)	Hydro Substation: Peak SS Concentration (mg/l)	Max. Surcharge Vol Gutter no. (ft^3): 7	Max. Surcharge Vol Gutter no. (ft^3): 36
Standard Storm time 50 min	1.05	.999	1216	0:50	947	219,000	174,000
Storm Time 25 min	1.05	1.242	1510	0:35	876	441,000	691,000
Storm Time 75 min	1.04	.905	845	1:05	945	-	-
Storm Time 115 min	0.99	.720	549	1:25	880	-	-

Time Step

Rainfall observations are difficult to obtain at small time increments and computing costs become higher at smaller timesteps as well. The question that arises is: when are the timesteps small enough ? 1-hr, 30 min., 15 min., 5 min., 1 min., or less ? By using the exact same storm time-averaged into various timesteps this effect can be illustrated. Table 4 indicates the rather obvious trend for the range of integration periods. The finer the timestep, the higher and earlier is the peak discharge and the greater are the surcharge volumes. Figure 8 compares the effect of a 5-minute hyetograph integration period to a 30 minute hyetograph integration period. The hydrograph is produced at a key location in the Chedoke Creek Catchment. The peak discharge varies from 1247 cfs for a 1-min. hyetograph to 515 cfs for an hourly hyetograph.

For urban environments where the typical time of concentration is short, hyetographs with timesteps larger than 5-minutes produce inconclusive results. Thus inexpensive, highly-accurate instrumentation is desirable. In 1981 a microprocessor based raingauge data acquisition and processing system was developed by our group (James et al, 8) for collecting data at one-minute time steps. The data retrieval and plotting system is also efficient and inexpensive. Further development of this system is on-going with research focusing on real-time control of flooding.

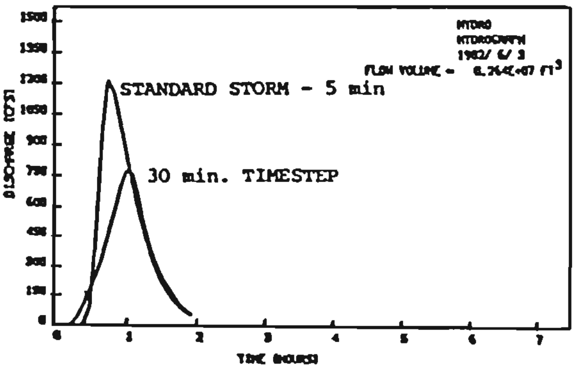

Figure 8. Timestep comparison

IMPLICATIONS FOR DESIGN

Present design methodology does not permit storm motion. In our study we applied the standard storm, Table 1, Figure 9, to the entire catchment area simultaneously, thereby eliminating the effects of storm speed and direction. Present design procedure effectively simulates a stationary storm which grows and dies uniformly over the entire catchment area, an unlikely event. The same hyetograph was applied to all 24 subcatchments in the Chedoke Creek study area, and the results are shown in Table 6 and Figure 10. Computed peak discharge does not vary significantly from computed results from our moving standard storm, but time to peak and surcharge volumes differ considerably. The pseudo stationary storm discharge peak occurs 15 minutes earlier and its peak surcharge volumes are 70,000 and 50,000 cu. ft.

Table 6. Stationary versus Kinematic Storm

			Hydro Substation				
Test Type	Inches Over Basin	Total Flow at Inlets, (cfs x 10^7)	Peak Discharge (cfs)	Time to Peak (hr:min)	Peak of Concentration (mg/l)	Max. Surcharge Vol Gutter no. (ft^3) 7	30
Standard 24 Hyetographs	1.05	.999	1216	0:50	947	239,000	174,000
Homogeneous Stationary 1 Hyetograph	1.03	.999	1149	0:40	1100	300,000	224,000

larger than the standard moving storm. Thus the design of super-sewers or detention tanks would have been significantly over-estimated had conventional design procedure been used, yet peak discharge would have been underestimated.

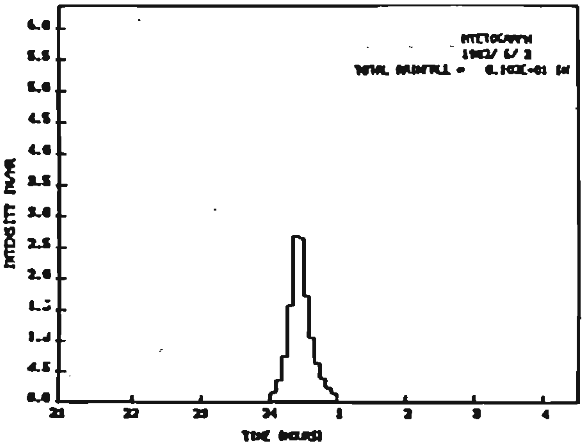

Figure 9. Standard hyetograph

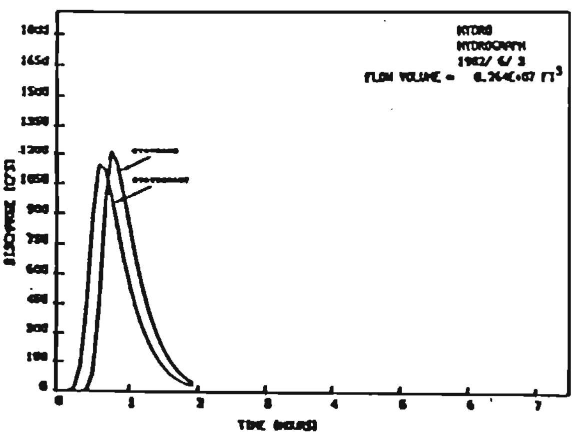

Figure 10. Kinematic versus stationary storm

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ATMOSPHERIC AND LAND-BASED LOADINGS FOR STORMWATER RUNOFF QUALITY MODELS

by

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Abstract. The deterioration of natural water quality by non-point source pollution continues to stimulate research in a broad range of disiplines. Urban runoff is one of the identified sources of heavy metals and toxic substances. Many runoff models are currently in use to predict both the quantity and quality of stormwater runoff. In most models, the quality algorithms need modification to gain the confidence of model users. The writers have attempted to discretize the accumulation process and to develop improved algorithms for buildup. A multiregression model ATMDST is developed to predict atmospheric dustfall considering meteorological information. This data is then used in NEWBLD to calculate pollutant accumulation on individual subcatchments. Both ATMDST and NEWBLD are interfaced with SWMM3 RUNOFF and applied to the Chedoke Creek catchment in Hamilton. The pollutographs for SS, BOD, total nitrogen and total phosphorous computed by both the modified and unmodified SWMM3 algorithms along with observed data are presented in this paper.

Introduction

Urban runoff is one of the identified major nonpoint sources contributing pollutants to natural waters. The type and quantity of urban pollutants generated are also widely different from one area to another. The impact of heavy metals and toxic substances on lakes and rivers has interested many researchers. During the last decade, many theories and computer models have been advanced, aimed at restoring the quality of natural waters. Among the available quantity and quality algorithms, the quality processes are yet to gain the confidence of model users.

Model Development

Pollutants from pervious and impervious areas are governed by three major processes: accumulation, washoff and transport. Materials scavenged during rainfall, and materials previously deposited in storm sewers and catchbasins, also add to the total quantity. The mass balance for pollutants in stormwater for a single storm event is:

$$Pru = Pim + Psc + Ppe + Psd \qquad (1)$$

where Pru = Total mass of pollutants in the runoff water
Pim = Total mass of accumulated pollutants washed off from impervious areas.
Psc = Total mass of pollutants scavenged by rain during its motion through the atmosphere,
Ppe = Total mass of pollutants washed off pervious areas,
Psd = Total mass of previously deposited pollutants scoured from storm sewers and catchbasins.

Pollutant accumulation is the net addition or removal due to various processes in the catchment between storms. Previous work on accumulation indicates that it might be linear or exponential over a dry period. Available runoff quantity and quality models use either linear or non-linear accumulation without examining the physical processes involved. The standard approach is based on land use classifications to differentiate between the subcatchments. In the following, we discretize the accumulation process and develop algorithms for various processes. These new algorithms have been interfaced with the Storm Water Management Model (SWMM3).

The accumulation process can be expressed:

$$Pi = (Pa+Pv+Ps+Pp) - (Rv+Rb+Ri) \qquad (2)$$

where Pi =Mass of pollutant accumulated per day
Pa =Mass of atmospheric pollutant fallout per day
Pv =Mass of pollutants from vehicles per day
Ps =Mass of pollutants from special activities such as construction demolition of stuctures etc.
Pp =Mass of pollutants due to population
Rv =Mass of pollutants removed by vehicle generated eddies per day
Rb =Mass of pollutants removed by biological decomposition
Ri =Mass of pollutants removed due to street cleaning

Each of these processes is now discussed in turn.

1. Atmospheric dustfall

Industries, vehicle exhausts and wind blowing over unprotected pavements introduce pollutants to the atmosphere. Redistribution of this material on the ground depends mainly on prevailing meteorological and geological conditions. Recent studies have revealed that the contribution of solids and nutrients to urban runoff from the atmosphere is significant, and cannot be assumed to be negligible(1,2,3,4). Runoff models do not consider the physics of atmospheric fallout. A new model ATMDST has been developed to predict atmospheric fallout on individual subcatchments using prevailing meteorological conditions, and based on statistical methods.

ATMDST has been developed for an industrial city (Hamilton), using eighteen dustfall monitoring stations operated by the Ontario Ministry of The Environment. Dustfall data have been collected for the last two decades, but implementation of pollution prevention measures to point sources has reduced dustfall such that data from only 1977 on can be used for model development. In fact data from 1977-1980 was used to develop the model and data for 1981 for validation. Multiregression analysis using monthly average wind velocity and monthly total precipitation as independent parameters and monthly total dustfall as dependent parameter, was carried out for all the eight principal compass directions.

The standard deviations between observed data and predicted dustfall data for all the stations were determined for all eight directions. It was found that the NE wind velocity yielded the lowest standard deviation and a better fit to the observed data for Hamilton. The observed and ATMDST-computed dustfall isopleths in grams/sq.m/30days are plotted in Figure 1. The regression constants are given in Table 1. These results are linearly interpolated to obtain loadings on the subcatchments, by joining the dustfall stations by straight lines as shown in Figure 2. The calculated dust quantity on discretized subcatchments on an effective hourly basis is tabulated in Table 2.

2. Vehicle input

Traffic byproducts, gasoline and oil drippings, tire residuals, brakeware, vehicle exhaust and metallic corrosion are also major contributers to pollutant accumulation. Many studies have focussed on vehicle input and developed algorithms for the prediction of runoff quality(5). These vehicle produced pollutants could be quantified:

$$VEHI = NVEH * VEHL * MILE \qquad (3)$$

where VEHI = mass of pollutants produced by vehicles
NVEH = number of vehicles in a subcatchment
= total population/PVR
PVR = population to vehicle ratio = 2.5 to 3.0
VEHL = mass of pollutants produced per vehicle per mile travelled (0.00288lb/axle-mile).
MILE = total length of road in miles

3. Population input

Compared to the sources discussed above, pollution due to population density is not significant. This could be quantified as follows:

$$POPI = POPU * POPL \qquad (4)$$

where POPI= total mass of pollutants due to population
POPU= total population in a subcatchment
POPL= mass of pollutants/capita-day (200mg/cap-day)

4. Special activities

This component is added to accumulation only in the case of construction, demolition and any other pollutant producing activity in the subcatchment. Pollutant due to this is estimated:

$$SPI = SPL * AREA \qquad (5)$$

where SPI = total mass of pollutants produced by special activities
SPL = pollutant loading/acres-day
AREA = special activity area in acres.

5. Biological removal

Organic pollutants decompose into simple substances in the presence of organisms, with or without oxygen. This reduces the total quantity of pollutants available at the time of washoff. Nitrate and phos-

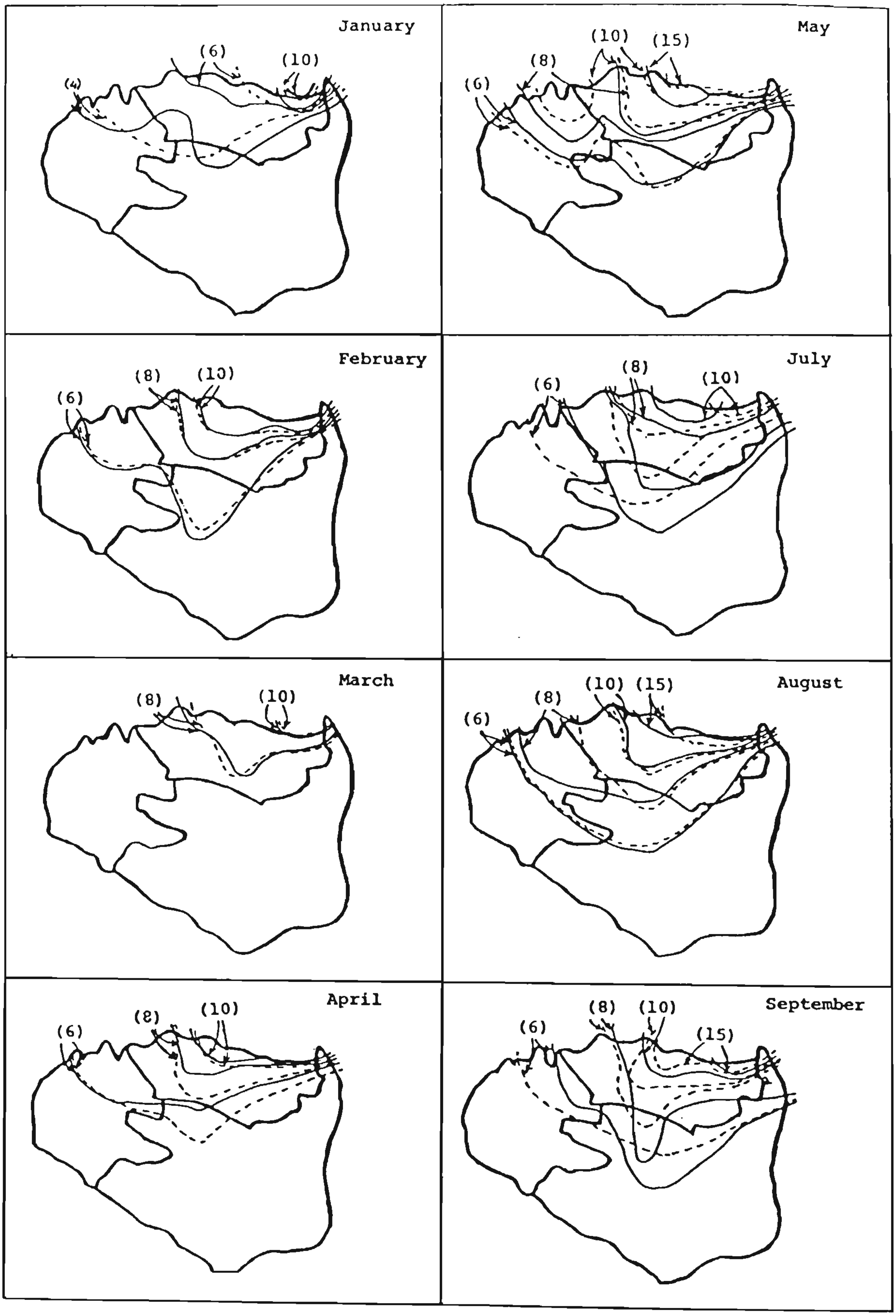

Figure 1. Observed and ATMDST computed dustfall isopleths,1981 data
(▬ Catchment boundry, —— Observed, ---- Computed)

Table 1. Multiregression coefficients

Stat.	Dire.	Const-A	Const-B	Const-C
9001	NE	5584.695	3.092	4.088
9006	NE	4677.929	6.449	-0.750
9008	NE	8058.933	14.286	-0.020
9009	NE	4004.965	5.310	-2.712
9011	NE	11997.335	8.688	3.490
9012	NE	5265.253	2.398	49.905
9017	NE	13110.332	-10.602	17.028
9019	NE	3553.187	-7.462	19.451
9025	NE	8929.253	-2.649	26.513
9026	NE	5831.605	-0.599	-2.191
9030	NE	7962.386	-7.282	6.993
9031	NE	4319.199	7.230	5.948
9036	NE	12053.507	-7.755	44.986
9037	NE	24161.562	-25.879	77.735
9044	NE	7249.218	9.962	1.332
9046	NE	3432.210	-9.443	0.214
9067	NE	4689.531	2.138	7.063

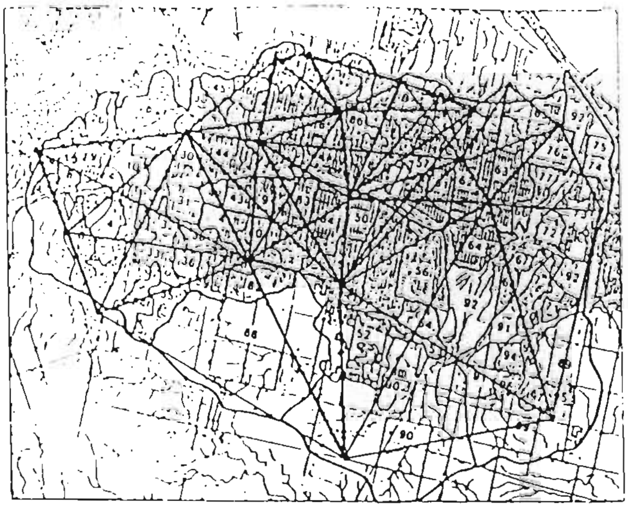

Figure 2. Dustfall distribution network.

Table 2. Predicted dustfall on subcatchments,mg/sq.m/eff.hr

Subcat. #	Jan.	Feb.	March	April	May	July	August	Sept.
1	15.27	22.32	24.06	10.62	6.33	11.24	23.49	21.93
2	37.73	54.96	63.49	27.32	14.97	28.96	59.62	57.06
3	44.02	64.71	72.49	30.93	17.23	32.83	69.27	65.54
4	33.32	43.72	55.19	23.77	13.23	25.20	52.40	49.81
5	40.05	57.96	68.79	29.66	15.96	31.41	63.63	61.48
6	5.19	7.50	7.00	3.44	2.51	3.62	7.43	6.65
7	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8	9.38	14.00	12.99	5.83	4.01	6.23	13.86	12.31
10	66.05	95.93	113.99	48.76	25.94	51.68	105.38	101.81
24	9.38	14.00	12.99	5.83	4.01	6.23	13.86	12.31
25	27.03	38.97	46.19	20.10	10.97	21.27	42.75	41.32
26	53.06	76.94	91.39	39.21	20.95	41.55	84.50	81.65
27	53.06	76.94	91.39	39.21	20.95	41.55	84.50	81.65
28	105.12	152.89	181.78	77.42	40.90	82.09	168.01	162.29
29	98.62	143.39	170.45	72.64	38.40	77.02	157.57	152.21
30	107.54	153.63	196.73	84.08	42.19	89.01	174.65	173.13
31	56.60	84.21	90.48	38.30	21.75	40.72	88.57	82.51
32	21.96	33.49	30.99	13.20	8.53	14.08	33.16	29.27
33	21.96	33.49	30.99	13.20	8.53	14.08	33.16	29.27
34	9.38	14.00	12.99	5.88	4.01	6.23	13.86	12.31
35	13.58	20.50	18.99	8.32	5.52	8.85	20.30	17.96
36	26.15	39.99	36.93	15.64	10.03	16.69	39.59	34.93
37	26.15	39.99	36.93	15.64	10.03	16.69	39.59	34.93
38	26.15	39.99	36.93	15.64	10.03	16.69	39.59	34.93
39	26.15	39.99	36.93	15.64	10.03	16.69	39.59	34.93
40	32.44	49.74	45.98	19.30	12.29	20.81	49.24	43.41
42	87.04	128.42	143.97	60.96	33.46	64.76	137.54	130.05
43	109.12	159.19	187.33	79.60	42.26	84.45	174.14	167.58
44	106.65	153.19	192.13	82.00	41.71	86.85	172.45	169.74
79	84.59	121.97	150.63	64.22	33.00	68.05	136.34	133.47
82	32.44	49.74	45.98	19.30	12.29	20.61	49.24	43.41

phate are exceptions, normally increasing by the addition of decomposed end products. This removal of solids by biological decomposition is calculated:

REMB = POLL*FRA*(1-EXP(-DECAY*DAY)) (6)

where REMB = total mass of pollutants removed by biological decomposition
POLL = average mass of pollutant
FRA = fraction of decomposable pollutants
DECAY = decay coefficient
DAY = number of days.

6. Vehicle removal

Vehicle generated eddies transport pollutants from streets to ineffective impervious and pervious areas from which they may not be washed off in the stormwater. The pollutant lost from the system is calculated:

REMV = NVECH * FR * POLL (7)

where REMV = mass of pollutant removed due to vehicle generated eddies,
NVEH = number of vehicles in a subcatchment,
POLL = pollutant available for eddy-transport,

FR=THETA1*(1-EXP(-THETA2*NSPEED)) (8)

THETA1 = maximum fraction of pollutants that could be removed
THETA2 = decay coefficient
NSPEED = vehicle speed in miles/hr.

7. Intentional removal

Regular street cleaning enhances aesthetics and minimises pollutant loadings on natural water. The frequency and efficiency of cleaning depend on traffic density and cleaning equipment respectively. Solids remaining on the pavement after street cleaning finally gets washed off by rain:

REMAIN = 1.0 - AVSWP * REFF (9)

where REMAIN = fraction of constituent load remaining on subcatchment surface

AVSWP = 0.6 * PD ** (-0.2) (10)
AVSWP = availability factor
PD = population density, persons/acre
REFF = removal efficiency.

The net accumulation on an impervious area is then calculated on a daily basis considering both addition and removal quantities. Subroutine NEWBLD was developed using the algorithms discussed above for the prediction of pollutant accumulation on individual subcatchments. A provision to input the sweeping time series for all the subcatchments is made in the subroutine. Fraction of dust and dirt arrested by vegetation and buildings due to canopy effect is also added to accumulation. NEWBLD has been inserted in SWMM3-RUNOFF.

Results and discussions

ATMDST and NEWBLD were calibrated using 1980 data from the Chedoke creek catchment in Hamilton. The discritized catchment is shown in Figure 3. It consists of single

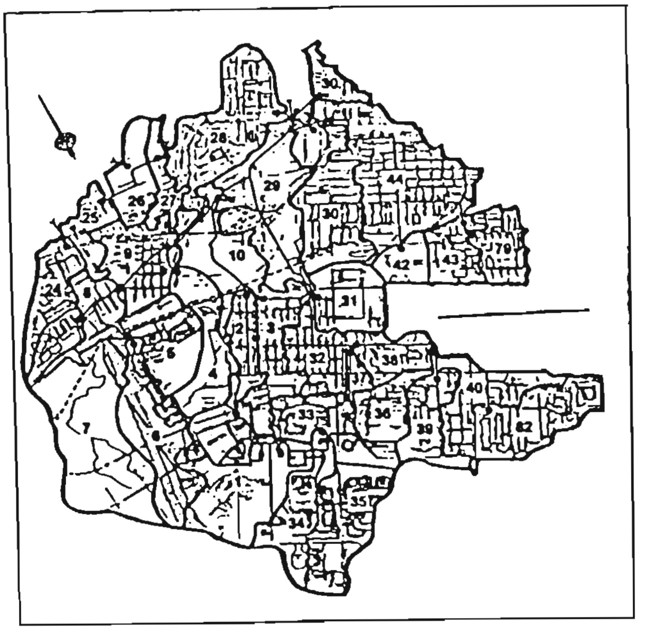

Figure 3. Discretized Chedoke creek catchment.

and multi residential areas and open land. No business districts or industrial areas lie in the catchment, although a heavily industralised area lies to the North-East. Dustfall computed by ATMDST was then input to SWMM3-NEWBLD and individual buildup was estimated for BOD,COD,SS,TSS, total nitrogen and total phosphorous during the interstorm periods. The pollutographs for BOD,SS, total nitrogen and total phosphorous computed by modified and unmodified SWMM3-RUNOFF along with the observed data are shown in Figure 4.

Conclusions

The conclusions that may be drawn are:

1) Discretization of the accumulation process and use of algorithms for individual processes yield better pollutographs.

2) In multiregression analysis, NE winds yield minimum standard deviations between observed and predicted dustfall over Hamilton.

3) Addition of ATMDST and NEWBLD to SWMM3-RUNOFF improved the computed pollutographs.

At present, pollutant prediction is gener-

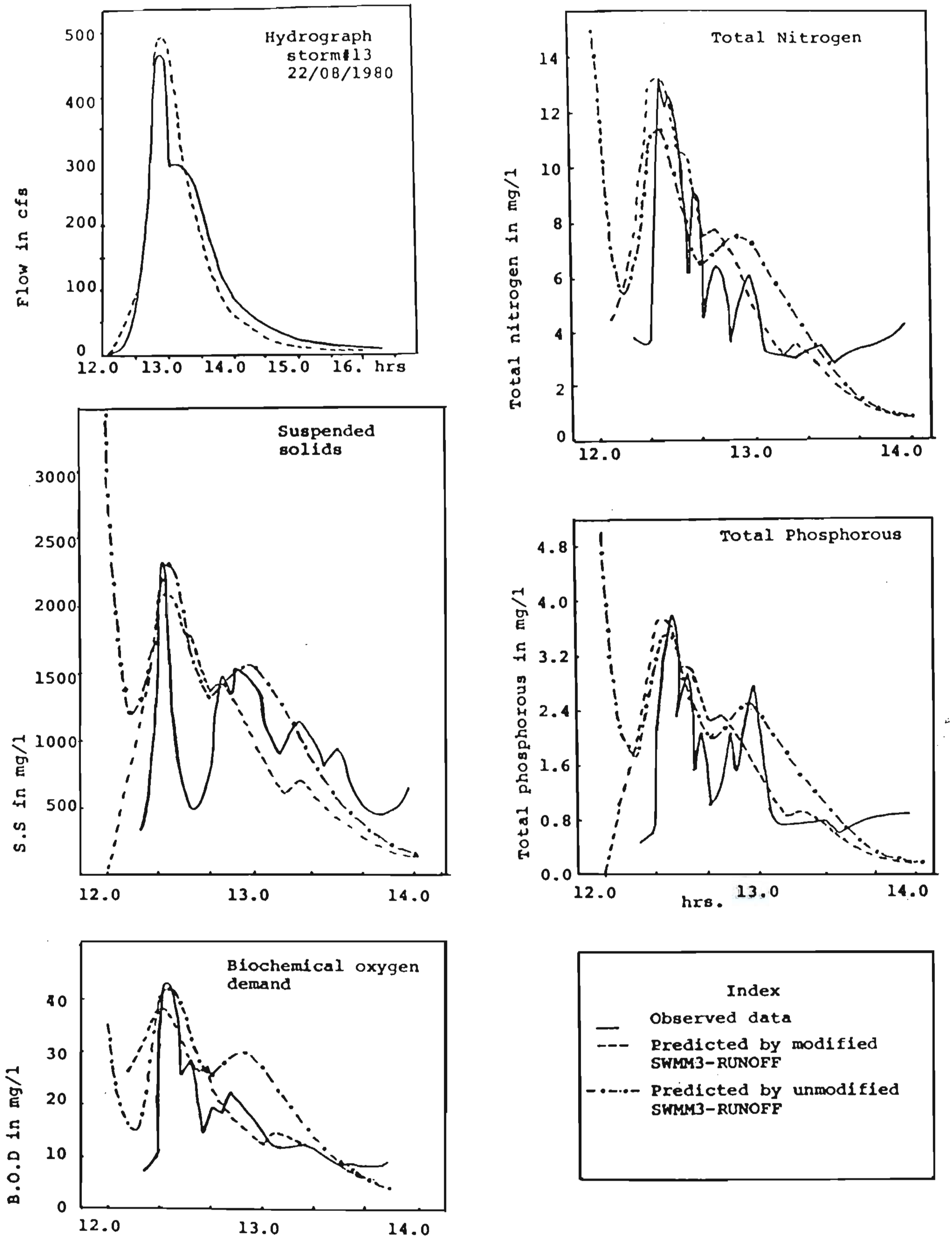

Figure 4. Computed pollutographs by modified and unmodified SWMM3-RUNOFF along with the observed data (Hotpoint)

ally based on land use classifications with linear or non-linear buildup functions. This approach does not depict the processes nor is it sensitive to meteorological parameters. NEWBLD gives more flexibility to users to model prevailing conditions in the subcatchments. The comparision of calibrated pollutographs between NEWBLD and unmodified SWMM3 shows that NEWBLD provides a better fit. Routing algorithms in GQUAL have also been modified to improve predictions. The overall performance of these routines is better than unmodified SWMM3, at least for our study conditions.

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STORMWATER QUALITY AND ATMOSPHERIC POLLUTION MODELLING IN AN INDUSTRIAL CITY

by

WILLIAM JAMES(1) and SHIVALINGAIAH, B.(2)

ABSTRACT

The location of the industrial areas, the direction, velocity and duration of wind and the source concentrations are important parameters in the prediction of atmospheric fallout, a component of surface pollutant loadings. In the paper, observed isopleths of dustfall are correlated with these parameters and superimposed on the discretized subcatchments for a comprehensive stormwater quality management model, SWMM3. The water quality algorithms in SWMM3 have been enhanced to include atmospheric fallout, street sweeping time series data, separation of pollution source areas, variable time step hydrology and moving storm analysis. Model results are presented for some of these enhancements and compared with observed loadings in Hamilton.

KEYWORDS

Urban Runoff water quality; atmospheric fallout; Chedoke creek catchment.

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RESUME

Le site de la zone industrielle, la direction, la vitesse et la durée des vents et la concentration des sources sont certains des paramètres importants dans la prévision des rejets atmosphériques, une des composantes principales de la pollution. Dans cette communication les "isopleths" observés du tourbillon de poussière sont mis en corrélation avec ces paramètres et surimposés sur les bassins de réception discrétisés par un modèle compréhensif de gestion des eaux pluviales SWMM3. Les algorithmes de la qualité d'eau en SWMM3 ont été améliorés afin d'inclure les rejets dans l'atmosphère, les données sur le balayage des rues, l'hydrologie à temps variable et l'analyse du parcours des eaux pluviales. Les résultats de ce modèle sont présentés pour certaines de ces améliorations et comparés avec les valeurs prototypes d'Hamilton.

INTRODUCTION

Urban runoff contributes considerable quantities of pollutants to lakes and streams. Growing population, increasing motorization and industrial production augments pollutant loading in an urban environment. In spite of laws governing the safe disposal of point source pollutants, the deterioration of surface water quality is evidently continuing. This in turn has stimulated investigation of nonpoint pollutant sources. Starting in the early 1970's, much work has been done to uncover the mechanism of pollutant build-up and wash-off due to rainfall.

Various human activities initiate pollutant accumulation on the ground surface and in the atmosphere. Atmospheric pollutants ultimately reach the ground, either by gravitational settling or by precipitation scavenging. The contribution of atmospheric pollutants to runoff water quality was initially assumed to be negligible, but recent studies have shown that this is the source of considerable quantities of solids and nutrients (Randall et al 1978, per Arne Malmquist 1978, Vladimir Novotny 1981, Albert Goettle 1978, Dean Stuart 1978, Chan and Kuntz 1981). Most runoff water quality models use dust and dirt build-up equations without considering the physical processes of pollutant generation from various sources. The accumulation processes are obviously complex in an industrial urban situation, yet urban runoff models such as STORM and SWMM lump all the sources together, estimating the build up rates under a conventional land use classification. Atmospheric fallout and the contribution from scavenging is a local phenomenon which is mainly dependent on wind velocity, frequency and direction, location of various industries, total precipitation, topography of the catchment and frequency of temperature inversions. A comparison between build up rates cited in the literature for various land uses on one hand and observed atmospheric fallout in Hamilton on the other, indicates that atmospheric fallout contributes a considerable portion to dust and dirt build-up (Shivalingaiah and James, 1982). A model ATMDST which could be interfaced with the SWMM3 RUNOFF Block for the estimation of loading rate is developed as part of our ongoing research on urban runoff water quality algorithms. The model ATMDST predicts the atmospheric fallout on individual subcatchments located downwind.

GEOGRAPHICAL AND METEOROLOGICAL FEATURES OF HAMILTON

The city of Hamilton is located on the south western shore of Lake Ontario. It is one of Canada's major industrial cities. The iron and steel industry, the major activity in this city of 306,640 (1980), occupies most of the southern shore of Hamilton Harbour, with the downtown area to the southwest. The Niagara Escarpment divides the city into upper and lower sectors having an average height difference of 325 ft. (Fig.1). The terrain of both the upper and lower city is essentially flat. The escarpment is cut by a number of deep valleys of which the most important is the southwest- northeast aligned Dundas Valley. The main morphological features of the city with respect to pollution are the heavy industrial sector concentrated along the southern side of Burlington Bay and the central business district. The major steel plants are located in the former, together with associated industries including machinery, electrical and chemical manufacturing. The business district is made up of a core of multistorey commercial buildings of limited areal extent surrounded by a lower level of mixed commercial and residential properties.

The prevailing wind is dominantly west and southwest which helps advect the pollutants away from Hamilton. Northeast and easterly winds play an important role in distributing pollutants over the city. The meterological data from the RBG station indicates an increase in the frequency of northeast and easterly winds between March and September (Shivalingaiah and James, 1982). Increased frequency of northeast and easterly wind was found in daytime compared to night in the lower part of the city (A.C. Farhang 1982). This effect may be due to the influence of lake breezes. A clockwise shift in the northeast and easterly wind is related to the Niagara Escarpment and Dundas valley (Rouse et al 1972, A.C. Farhang 1982). Hamilton is highly susceptible to atmospheric inversions because of differential heating of the lake and the land surface. A persistant sharp elevated inversion anywhere between 1070 and 1080 m height over the heavy industrial zone has been observed (Rouse et al, 1972). The height of inversion depends upon the season, but is strongly developed in the morning, diminished in magnitude towards solar noon and strengthens again towards sunset.

The city is divided into three major catchments for storm runoff quantity and quality modelling. The catchments are: the central business district, Chedoke Creek, and Redhill Creek. These catchments are further discretized into a number of subcatchments for detailed analyses. The location of the industrial area with respect to the catchments is shown in Fig. 1.

The central business district catchment consists of multistorey buildings with the highest degree of imperviousness. It also includes a lower level of mixed, commercial and residential areas, as well as the principal industrial areas.

The Chedoke Creek catchment, extensively used in developing our water quality algorithms, consists of mainly single and multistorey residential areas, schools, parks and open areas. The Chedoke creek runs almost through the middle of the catchment. The catchment is further discretized into a number of subcatchments for detailed study (Fig. 2).

The Redhill Creek catchment is the largest, but is mostly undeveloped. Only about one fourth of the catchment is residential and the remaining area, farm land.

AVAILABILITY OF DATA

The Ontario Ministry of Environment maintains a dustfall sampling network of 18 stations over Hamilton, and has collected data over the past decade. Dustfall is collected in plastic open jars 1 to 2 m. above ground over a period of 30 days. This includes both dry and wet deposition.

Atmospheric pollutants scavenged by both snow and rain also accumulates in the jar. Interstorm period dry deposition also accumulates in the same jars, thus the data reflects bulk deposition. Accidential contamination (leaves, birds, etc) is removed before calculating the total dust deposition. The grain size distribution of accumulated dust is determined. Since dust accumulation data on ground surfaces is not available, it was decided to use the above dust collection jar data in the development of our ATMDST model.

MODEL DEVELOPMENT

Pollutants from the atmosphere are removed by natural processes generally referred to as either precipitation scavenging (wet process) or dry deposition (dry process). Precipitation scavenging or washout is the removal of pollutants from the atmosphere by precipitation. Dry deposition is the removal of pollutants due to gravitational, brownian and eddy diffusion in the absence of precipitation. Distinction is also made between below-cloud scavenging (washout) and incloud scavenging (rainout), depending on the elevation of the pollutant with respect to the cloud base.

The collection efficiency, with which pollutants are removed from an air stream by obstacles such as rain, snow, grass, leaves or water surfaces, is usually written as the product of collision efficiency and retention efficiency. After the collection efficiency has been determined, it is summed over all collecting elements to obtain an overall removal rate. Usually these integrals are complex in nature and must be rather severely approximated (Slinn, 1976). We have rather adopted a statistical approach to estimating the atmospheric fallout for a given collection station and sub-catchment.

The available dustfall record was divided into two parts, the first group being used to develop the model and the other used to test the model. The complete record covers 1970-1981, of which data for 1978-1980 was used for model development and 1981 data for model validation.

Sample wind rose and dustfall isopleths for Hamilton are presented in Figure. 4. Using wind velocity in km/hr and total precipitation (rain and snow) in mm water equivalent as independent parameters, a multi-regression model was developed. Wind velocity and direction frequency fluctuate widely, yet seasonal variations are evident. Only one set of regression constants per station is used to predict dustfall for each month of the year:

$$\sum_{1}^{n}(\mathrm{DUSTFALL})_{m,s} = a_{m,s} + b_{m,s}\sum_{1}^{n}(\mathrm{DAWINVEL}) + c_{m,s}\sum_{1}^{n}(\mathrm{TOTPREC})_{m}$$

Where:

DUSTFALL	=	30 day mean bulk dustfall per day (gram/day-sq.m).
DAWINUEL	=	daily average wind velocity (km/hr)
TOTPREC	=	total precipitation per day (mm water equivalent).
N	=	number of days (30 days)
a,b and c	=	multi-regression constants.

m,s = subscripts denoting month and sampling station.

Predicted values are compared to observations in Figure 5 for all dust collection stations.

Multiple regression constants are given in Table 1. Standard diviations have been minimized for all the directions of wind velocities for all the dust collection stations and a particular direction wind velocity for all the stations. The results show that the NE wind direction correlates best with dust collection, having the lowest standard deviation for most collection stations. The multi-regression model (ATMDST) is also compared to average and linear regression models.

The dustfall rate produced by ATMDST at the collection stations is ultimately to be used to predict mean subcatchment dust accumulation rate on the ground surface. In doing so, the wet deposition component was deducted from the predicted total dustfall and units converted from gram per sq.m per 30 days to grams per sq.m. per effective hour (effective hour based on wind direction and frequency). The model assumes that dust falls on the subcatchment only during the effective hours and that the rate of dustfall accumulation on the ground surface is equal to the observed dust collection rate 1 to 2 m above the ground surface. Those assumptions allow the direct use of available dust collection data to calculate the dust flux on a given subcatchment.

To interpolate catchment mean dustfall from data generated at the stations, all the dust sampling stations were connected by straight lines (Figure 3). Assuming linear distributions between two collection stations, the dustfall rate was interpolated on the subcatchments. Since all dust collection stations are located within the catchments, six imaginary stations with observed background concentrations were placed at the outskirts of the catchments. The computed results for various subcatchments are tabulated in Table 2. These dustfall results are finally added to a local dust component contributed by various physical, chemical and biological surface activities to obtain the net dust accumulation rate. Preliminary results indicate that utilization of these dustfall data in the SWMM3 RUNOFF block with the recorded street sweeping time series and moving storm analysis improve runoff water quality prediction for Hamilton catchments (detailed results will be communicated in the near future). Further study on individual pollutants based on dustfall analysis data is now also under investigation.

CONCLUSIONS

The following conclusions may be drawn at this stage of our study:

1. The application of atmospheric pollutant algorithms to generate input for industrial urban runoff quality models appears to be useful.

2. The northeast wind direction yielded the best fit and minimum standard deviation between observed and predicted dustfall results for most dustfall collection stations in Hamilton.

3. A multi-regression model based on wind velocity and total precipitation as independent parameters yielded better results than average and linear models based on the same data.

4. The ATMDST model can be used to generate more meaningful input to the SWMM3 RUNOFF block for the prediction of industrial urban runoff water quality.

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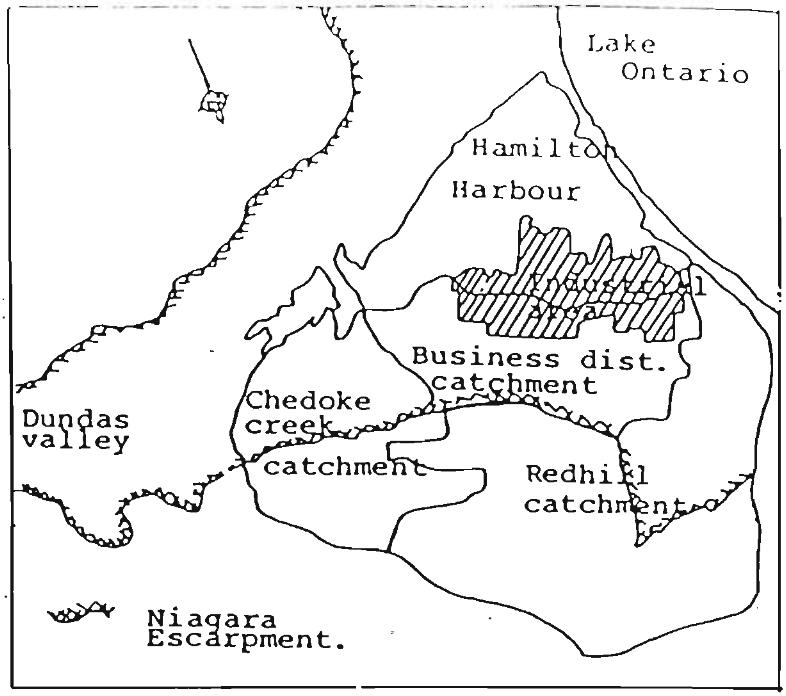

Fig.1. Geographical features of Hamilton.

Fig.2. Discretized Chedoke creek catchment.

Fig.3. Dustfall collection stations network.
(. Real stations X Imaginary stations)

Table 1. Multiregression coefficients

Stat.	Dire.	Const-A	Const-B	Const-C
9001	NE	5584.695	3.092	4.088
9006	NE	4677.929	6.449	-0.750
9008	NE	8058.933	14.286	-0.020
9009	NE	4004.965	5.310	-2.712
9011	NE	11997.335	8.688	3.490
9012	NE	5265.253	2.398	49.905
9017	NE	13110.332	-10.602	17.028
9019	NE	3553.187	-7.462	19.451
9025	NE	8929.253	-2.649	26.513
9026	NE	5831.605	-0.599	-2.191
9030	NE	7962.386	-7.282	6.993
9031	NE	4319.199	7.230	5.948
9036	NE	12053.507	-7.755	44.986
9037	NE	24161.562	-25.879	77.735
9044	NE	7249.218	9.962	1.332
9046	NE	3432.210	-9.443	0.214
9067	NE	4689.531	2.138	7.063

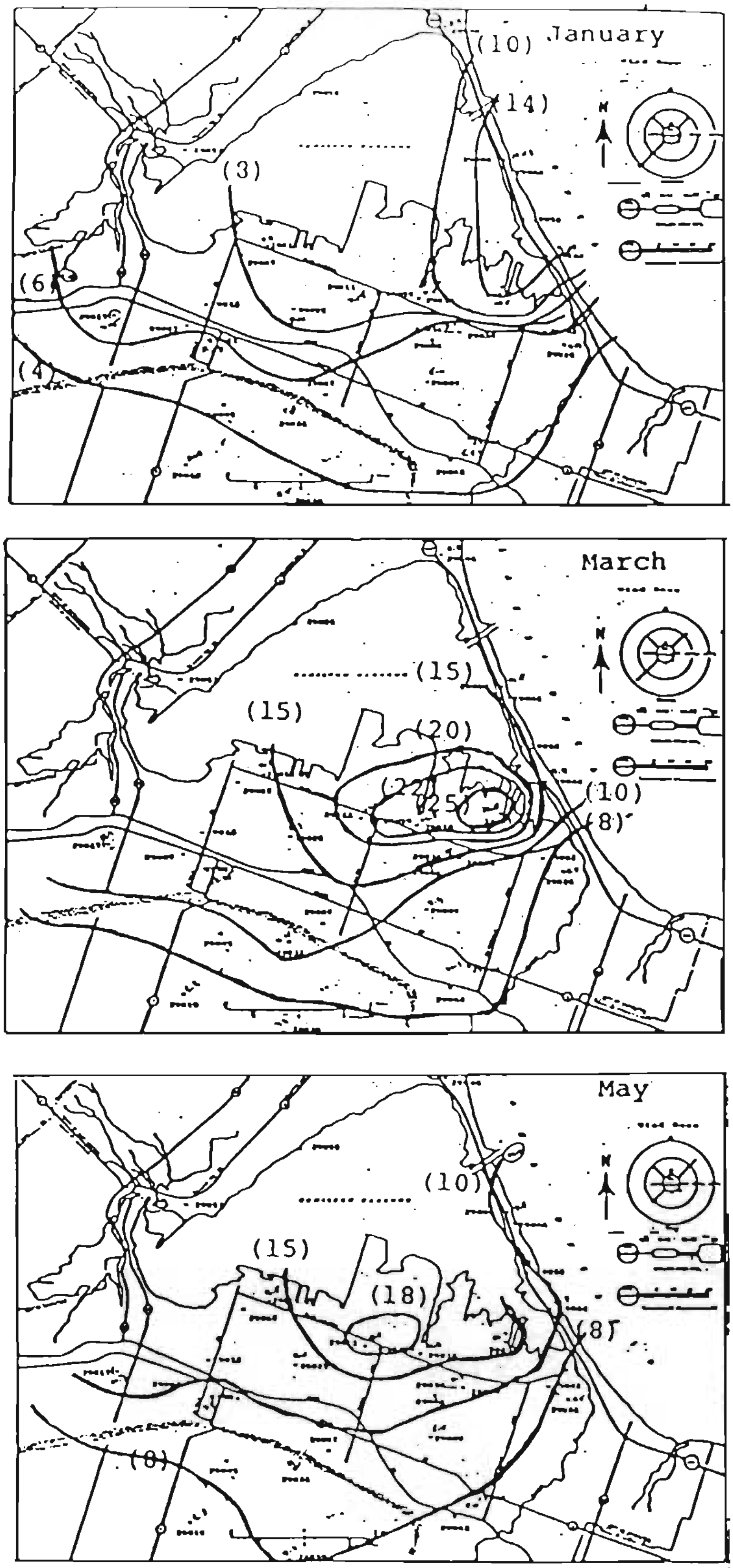

Fig.4. Average monthly dustfall isopleths,grams/sq.m-30days,1977-1980.

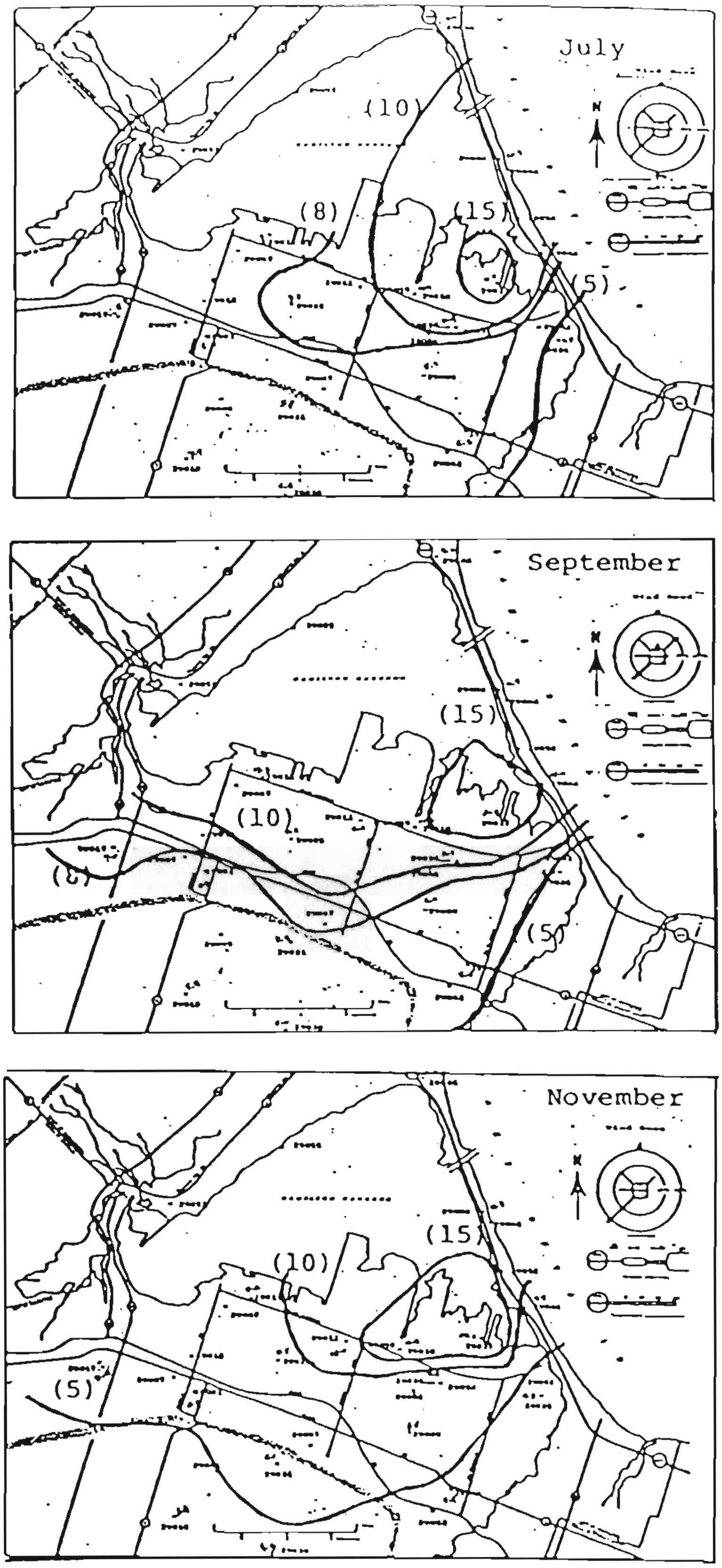

Fig.4.continuation

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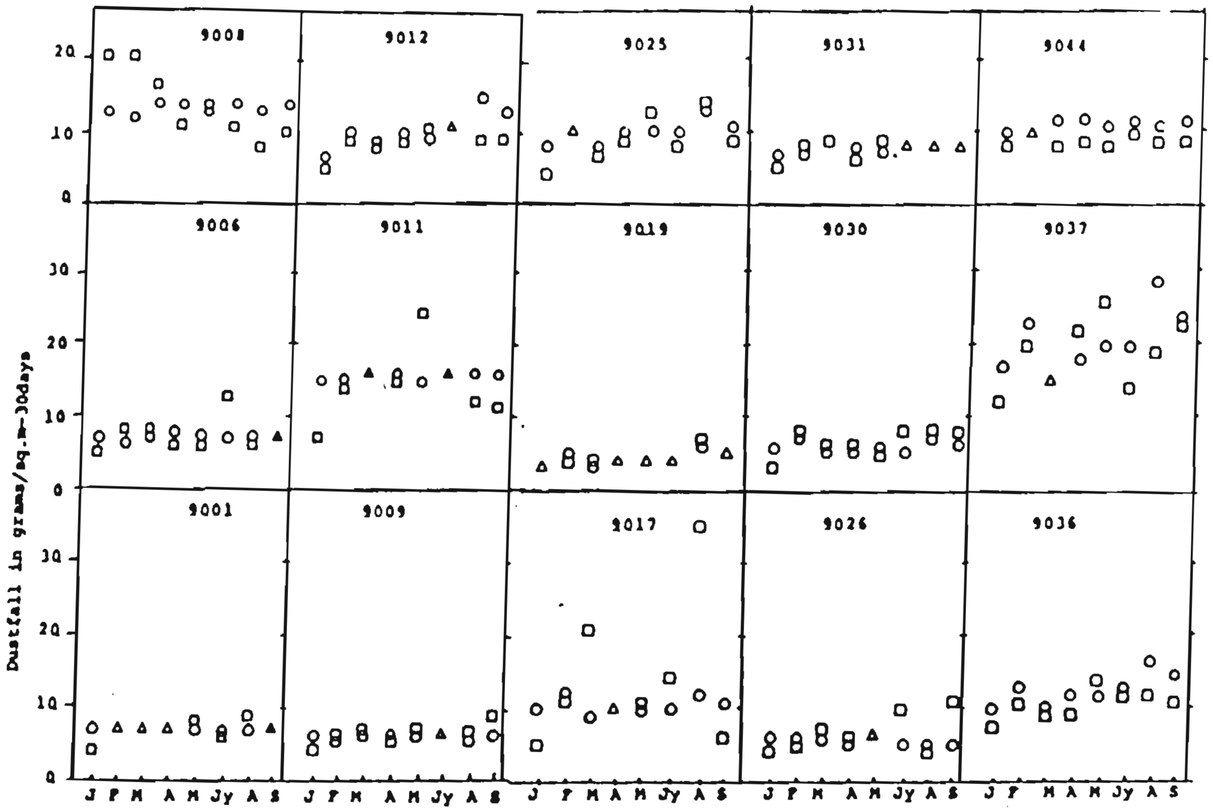

Fig.5. Observed and predicted dustfall at dust collection stations.
(O predicted, □ observed, △ common)

Table 2.Predicted dustfall on subcatchments, in mg/sq.m-eff.hr.

Sub-cat. #	Jan	Feb	Mar	April	May	Jul	Aug	Sept
1	15.27	22.37	24.06	10.62	6.33	11.24	23.49	21.93
2	37.73	54.96	63.49	27.32	14.97	28.96	59.62	57.06
3	44.02	64.71	72.49	30.93	17.23	32.83	69.27	65.54
4	33.32	43.72	55.19	23.77	13.23	25.20	52.40	49.81
5	40.05	57.96	68.79	29.66	15.96	31.41	63.63	61.48
6	5.19	7.50	7.00	3.44	2.51	3.62	7.43	6.65
7	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8	9.38	14.00	12.99	5.83	4.01	6.23	13.86	12.31
10	66.05	95.93	113.99	48.76	25.94	51.68	105.38	101.81
24	9.38	14.00	12.99	5.83	4.01	6.23	13.86	12.31
25	27.03	38.97	46.19	20.10	10.97	21.27	42.75	41.32
26	53.06	76.94	91.39	39.21	20.95	41.55	84.50	81.65
27	53.06	76.94	91.39	39.21	20.95	41.55	84.50	81.65
28	105.12	152.89	181.78	77.42	40.90	82.09	168.01	162.29
29	98.62	143.39	170.45	72.64	38.40	77.02	157.57	152.21
30	107.54	153.63	196.73	84.08	42.19	89.01	174.65	173.13
31	56.60	84.21	90.48	38.30	21.75	40.72	88.57	82.51
32	21.96	33.49	30.99	13.20	8.53	14.08	33.16	29.27
33	21.96	33.49	30.99	13.20	8.53	14.08	33.16	29.27
34	9.38	14.00	12.99	5.88	4.01	6.23	13.86	12.31
35	13.58	20.50	18.99	8.32	5.52	8.85	20.30	17.96
36	26.15	39.99	36.93	15.64	10.03	16.69	39.59	34.93
37	26.15	39.99	36.93	15.64	10.03	16.69	39.59	34.93
38	26.15	39.99	36.93	15.64	10.03	16.69	39.59	34.93
39	26.15	39.99	36.93	15.64	10.03	16.69	39.59	34.93
40	32.44	49.74	45.98	19.30	12.29	20.81	49.24	43.41
42	87.04	128.42	143.97	60.96	33.46	64.76	137.54	130.05
43	109.12	159.19	187.33	79.60	42.26	84.45	174.14	167.58
44	106.65	153.19	192.13	82.00	41.71	86.85	172.45	169.74
79	84.59	121.97	150.63	64.22	33.00	68.05	136.34	133.47
82	32.44	49.74	45.98	19.30	12.29	20.61	49.24	43.41
45	107.54	153.65	196.73	84.03	42.19	89.01	174.65	173.13
46	95.73	135.10	181.61	77.86	37.85	82.32	157.09	158.38
47	83.93	116.51	166.49	71.64	33.51	75.64	139.54	143.64
48	40.22	54.95	81.92	35.62	16.48	37.55	67.20	70.21
49	41.35	55.89	36.79	37.80	17.06	39.82	69.77	73.89
57	85.73	113.97	139.11	82.02	35.09	86.36	147.29	159.26
58	163.92	221.38	343.53	150.77	66.50	158.89	278.37	295.93
59	117.03	164.83	223.25	95.75	46.33	101.23	192.35	194.44
60	88.55	117.61	195.98	85.02	36.28	39.21	152.34	164.94
61	81.71	108.80	179.60	77.88	33.41	82.01	140.23	151.38
62	87.40	117.61	187.45	81.14	35.53	85.49	148.83	158.85
63	76.57	107.82	145.28	82.48	30.49	66.05	125.51	126.67
65	110.65	158.46	201.20	85.94	43.36	91.00	179.40	177.33
76	80.17	109.61	165.39	71.53	32.30	75.29	134.88	141.40
85	75.96	105.59	150.43	64.84	30.45	65.46	126.14	129.83
86	142.05	193.31	296.48	128.07	57.38	135.05	239.84	252.79
87	105.02	144.22	214.25	92.39	42.20	97.48	176.08	183.64
41	78.22	115.93	127.37	53.85	29.98	57.24	123.10	115.52
50	83.82	114.72	172.50	74.44	33.75	78.52	140.92	147.56
51	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
52	46.04	67.42	77.12	32.94	18.00	34.94	72.83	69.42
53	38.54	56.35	64.43	27.61	15.17	29.29	60.86	58.02
54	46.04	67.42	77.12	32.94	18.00	34.94	72.83	69.42
55	78.98	111.03	151.23	64.89	31.29	68.59	129.95	131.59
56	83.18	115.48	164.97	70.98	33.21	74.95	138.28	148.34
64	72.43	95.61	161.85	70.33	29.86	74.07	124.88	135.91
66	63.50	83.75	141.74	61.71	26.26	64.93	109.39	119.05
67	54.57	71.96	121.64	53.04	22.65	55.80	93.91	102.15
68	54.57	71.96	121.64	53.04	22.65	55.80	93.91	102.15
70	36.71	48.30	51.43	35.69	15.43	37.53	62.94	68.46
73	18.86	24.65	41.21	18.35	8.22	19.27	31.97	34.73
74	38.42	53.24	75.34	32.69	15.62	34.49	63.45	65.15
75	45.90	63.69	90.21	39.03	18.55	41.19	75.94	77.98
77	83.06	112.37	175.87	76.05	33.66	80.16	140.87	149.47
78	76.91	108.89	144.56	61.83	30.35	65.50	125.87	126.36
80	54.24	79.32	82.41	39.24	20.97	41.64	86.39	52.83
81	52.14	76.14	89.23	37.91	20.13	40.22	83.14	79.89
83	80.53	115.06	147.43	63.01	31.60	66.70	130.81	129.72
84	79.20	109.40	159.17	68.56	31.71	72.37	132.19	136.91
88	21.96	33.49	30.99	13.20	8.59	14.08	33.16	29.27
89	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
90	23.52	34.21	39.06	16.97	9.50	17.97	36.92	35.21
91	31.03	45.25	31.75	22.29	12.33	23.63	48.89	46.61
92	43.84	60.52	37.14	37.18	17.81	39.86	72.74	75.12
93	18.86	24.65	41.21	18.35	8.22	19.27	31.97	34.73
95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
96	16.01	23.14	26.37	11.65	6.67	12.31	24.94	23.81
97	16.01	23.14	26.37	11.65	6.67	12.31	24.94	23.81

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CONTINUOUS MODELS ESSENTIAL FOR DETENTION DESIGN

by

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ABSTRACT

Traditional design methodology for stormwater detention ponds is based on primitive methods such as design storms derived from intensity-duration-frequency analysis. Some of these methods have been enhanced in recent years and discrete-event hydrograph routing is typical. Modern design requires better estimates of initial conditions in the pond and the contributing catchment. This indicates a need for continuous modelling.

The paper describes techniques to account for general characteristics of real summer thunderstorms which, if ignored, lead to substantial errors. Difficulties associated with continuous modelling include smoothing due to coarse time-steps, input data deficiencies, and the management of large amounts of output data. Calibration of our continuous models requires inexpensive field instrumentation, specifically designed for use with continuous models running on micro-computers.

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INTRODUCTION

Traditional design methodology for stormwater detention basins is based on older, necessarily primitive methods suitable for slide-rule calculations, such as the SCS unit hydrograph approach and design storms derived from intensity-duration-frequency analysis. The I-D-F concept itself forms part of the rational formula school. These methods have, of course, been in use for a considerable period of time, and now some of them have been enhanced to a level of complexity suited to pocket-calculators; discrete event hydrograph routing is typical.

Modern design however, needs reliable estimates of risk and impacts downstream. This in turn requires a better estimate of initial storage in the pond and better estimates of initial moisture and initial pollution buildup in the contributing catchment. Since the associated inter-event dry periods are not generally sufficiently well-established to relate to the resulting flood frequency level, the initial conditions must be continuously simulated.

This indicates a need for new design methods based on continuous modelling, fortunately easily achieved using modern micro-computers. In fact the evolution of powerful, inexpensive micro-computers and their associated software makes detailed simulation widely available. The old complaints about computing cost are no longer relevant.

Thus it is now appropriate to re-evaluate storage design methodology in the light of the new micro-computer environment. For example, real summer thunderstorms have general characteristics which, if ignored, can lead to substantial errors. Such storms are of limited spatial extent, grow and die, and travel at substantial speeds over the ground. Better storm modelling is easily accomplished and the paper describes techniques to account for these general characteristics.

Storage design is more reliable if based on continuous modelling rather than discrete event modelling. Difficulties associated with continuous modelling include smoothing due to commonly-used coarse time-steps, input data deficiencies, and the management of large amounts of output data.

Calibration of a continuous model also presents certain difficulties. The key here is sensitivity analysis, and the use of special purpose inexpensive field instrumentation. Our instruments have been specifically designed for use with continuous models running on micro-computers. The design and construction of these instruments is briefly covered in this paper.

DESIGN STORMS

At a seminar on the design storm concept in Montreal in 1979, design storms were defined (1):

> "The overall idea of a design storm is to provide a means of estimating a discharge or volume of specified recurrence interval for planning or design

> purposes. That is, a recurrence interval is assigned to the design storm, a rainfall-runoff procedure is used to convert the rainfall to runoff, and the recurrence interval of the design storm is transferred to the resulting runoff discharge or volume."

When developing a design storm, the time distribution, the storm position and movement should all be considered. The resulting time and spatially varying design storm patterns are supposed to represent a statistical summary of historical precipitation records.

The following steps are usually involved:

- derive a set of rainfall depth-duration-frequency curves
- derive area-depth-frequency curves
- establish a temporal rainfall intensity distribution
- establish a spatial distribution of rainfall intensity.

To determine the temporal rainfall patterns during the storm one may either utilize some of the rainfall patterns reported in the literature, or derive such patterns from available precipitation data. Among the temporal rainfall patterns the best known appear to be the Chicago design storm, the U.K. Meteorological Office design storm, and distributions reported by Huff. General applicability of these three distributions has not been studied and therefore it is considered best to derive a distribution by a statistical analysis of local data.

Generally a prescribed return period is specified for a design project and then a design storm that has the same return period is selected. The project is designed not to fail when subjected to a calculated discrete flood produced by the discrete design storm event; it is assumed that the capacity of the design drainage system has a return period equal to that of the design storm, and that initial key storage conditions are correctly modelled in a statistical sense.

DRAWBACKS OF DESIGN STORM METHODS

The obvious drawback is that the probability of the computed peak runoff is not the same as that of the input total precipitation of the design storm (storms are usually ranked according to total precipitation, rather than peak 5-minute intensity). In a recent study (2), Sieker computed the frequency distribution of annual peaks of runoff synthesized from natural rainfall, and another frequency distribution computed from equivalent derived design storms of constant intensity. Both computations used a linear discrete event model. He concluded that the postulate "total rainfall frequency = flood peak frequency" is obviously not accurate and that, with this postulate, designs remain on the uncertain side.

The probabilities of storm and discharge are not equal because initial conditions, e.g. the soil moisture, surface loss rates, storage levels (even in detention ponds) immediately antecedent to the storm-flood event are simply unknown. Design storm analysis yields no information about the antecendent interevent dry weather period. The joint probability of both these initial conditions and the storm event must be known to compute a discrete flood discharge of a given frequency. We have shown that optimal detention storage is most likely to incur a highly variable initial storage level for design storms of typical frequencies (3).

Design storms are usually either developed by a simple statistical analysis of point rainfall records that include rain of all types, or an historic storm is used (particularly for rare events). In the former case the resulting temporal distribution of rain may be quite unlike the rainstorms occuring in nature (Urbonas, 1979) and thus result in runoff peaks that can vary significantly from peak flows calculated statistically from long-term simulation using recorded rainfall and calibrated models. Synthetic storms attempt to aggregate intensity duration stations from many storms of all types, thunderstorms and cyclonic rains, into a single and hence impossible storm event. the general design hyetograph shape is based on data from many geographic regions, some involving orographic and other local effects.

ALTERNATIVES TO DESIGN STORMS

Apart from the obvious classic statistical analysis of a long term discharge record, carefully applied to both the "virgin" catchment and the catchment incorporating the desired storage, there is no widely adopted alternative to design storms. Long-term discharge records are not available especially for rapidly urbanising areas.

A promising method has been proposed by Walesh et al. (4): hyetographs of major rainfall events are assembled from a long historic record and applied to a calibrated event model. Rain intensities thus still relate to actual precipitation types. Unfortunately, the historic record is usually based on independent single point rainfall observations, incapable of accounting for storm dynamics.

Marsalek (5) also used a discrete event model (SWMM) in a study of two other design storm models, including the Chicago design storm. Marsalek concluded that the Chicago storm produces runoff flows 80% larger than those produced by the historical storms of corresponding return period.

In these studies, and of course many others (6), the conclusions are subject to great doubt, because of the use of discrete models. Wenzel and Voorhees (7) used a continuous model to study design storms of uniform intensity as well as the Huff intensity distribution. They found that surcharging and antecendent conditions dominated, tending to render the design storm concept of doubtful value.

The trend seems to be increasingly critical of the use of a simple static, spatially-uniform design storm aggregating all kinds of storm types, and towards the use of a number of historic storms selected from the long-term record, or even continuous modelling using the entire long-term record.

PRECIPITATION TYPES SIGNIFICANT TO STORMWATER MODELLING

Stormwater management in urban areas typically involves subcatchments with short characteristic response times, for which short, sharp rain events are appropriate. This indicates summer convective thunderstorm rain cells of limited life and spatial extent. Such rain storms are always associated with distinct meso-scale motion over the ground (8). Expected synoptic characteristics of such rain cells are presented here, comparisons drawn with the Chicago storm, and a storm model suggested.

Cyclonic precipitation, associated with frontal systems, has two components: a broad belt of relatively low intensity rainfall lying along the warm front, and a narrower belt of relatively high intensity rainfall lying along the following cold front. Convective precipitation in summer may be caused by differential local surface heating and generate small intense rain cells, or the cold front rainband may in fact be composed of a linear set of convective cells associated with the air mass moving over the land. Occasionally, very intense, widespread convective rainfall may be associated with the incursion of a tropical storm into the Hamilton area (e.g. Hurricane Hazel in 1954).

Thunderstorms begin as cumulus clouds characterized by strong updrafts that reach 25,000 feet. During development of the storm additional moisture is provided by a considerable horizontal inflow of air. The storm enters a mature stage when the strong updrafts produce precipitation. Gusty surface winds move outward from the region of rainfall and heavy rainfall occurs for a period of 15 to 30 minutes. In the final dissipating stage of the storm the downdrafts predominate and precipitation tails off and ends.

Thunderstorms comprise one or more such cells in varying stages of development, the life cycle of which is usually completed in an hour or less. However, such storms tend to be self-propagating by the formation of new cells and in the Hamilton area generally move from the west (between, say, northwest and southwest) at speeds of 35-50 km/hr and in broken lines or bands up to 80 km in width. Severe storms may produce 5 cm of rain in less than half an hour, while slowly moving storms may appear to remain in one locality for an hour or more and produce a total point rainfall as great as 30 cm.

Although major structures may be designed on the basis of an exceptional recorded event, urban stormwater structures are designed on the basis of a composite synthetic storm, derived from raingauge data that is assumed to begin, peak and end simultaneously over the whole catchment. In fact, the greatest rates of runoff are usually associated with a linear set of convection cells containing individuals that continually generate and dissipate, and which moves across the area.

In summary, rain storms travel in preferred directions; they do not spontaneously grow and die over one spot, as suggested by current practice in the analysis of point rainfall data. Cells have substantial speeds and intensity variations across areas typically appropriate to urban runoff studies (e.g. 5-5000 acres). A substantial body of information is available and the general characteristics of stormcells can be described.

It is preferable to specify the expected speed and direction of movement of cells, and even cell size and rainfall intensity distribution, rather than to assume no speed or direction, and excessively large cells with uniform rainfall intensities (9).

Finally, thunderstorms have different characteristics from cyclonic events. Point rainfall data does not distinguish between rainfall types, and statistical analyses of rain data includes intensities from all types. Point rainfall data cannot generally provide information on storm cell kinematics.

THE STORM MODEL "THOR"

Radar studies of summer rain events resulting from moving clusters of sub-circular convective cells have shown that the cells are relatively short-lived, that they tend to have an exponentially-decreasing intensity away from the cell centre and that their statistical properties can be matched to those of ground-based precipitation records.

In our previous work (8) we propose a storm model THOR based on logarithmic rising and falling hyetograph limbs, similar to the Chicago storm. The Chicago storm attempts to distribute rain such that for any time interval the maximum average intensity is equal to that of the intensity-duration-frequency curves. The position of the peak intensity within the storm is based on observed storm characteristics (10).

To use the Chicago design storm in stormwater modelling, it is necessary to reduce the storm-hyetograph to a set of discrete values. The time-to-peak ratio, r, can be determined from an analysis of local storm distributions and depends on the depth of rainfall that can be expected antecendent to the peak intensity.

To assess the adequacy of the Chicago storm for design calculations, a comparison with flood frequency data is necessary. Synthetic flood frequency data can be generated by modelling.

Clarke and Bishop (11) conducted a study comparing simulation results for three interconnected catchments in Edmonton, Canada. The Chicago storm predictions agreed closely with those based on real storms. A similar study was conducted in Winnipeg, Canada (12) and again it was concluded that the theoretical storms resulted in runoff frequencies similar to those of the historical storms.

An evaluation of the Chicago storm was conducted by Watson (6). Fifty-four rainfall events were selected and discretized at 5-minute intervals from an effective record of 16 years. The storms were then ranked according to maximum average intensity for each duration. The top 16 storms for durations of 5, 15, 30, 60 and 120 minutes were selected for the analysis, making 28 storms in all. Peak discharges were computed using ILLUDAS for 2 catchments in South Africa. Soil types were varied to establish whether this would influence results. The Chicago storm predictions agreed closely with those based on historical data, for peak flows.

THOR assumes an infinitely wide rainband in which the rainfall intensity decays exponentially away from the line of peak intensity at different rates ahead and behind it:

$$P = PO \exp(-K1(tp-t)) \qquad t<tp$$
$$P = PO \exp(-K2(t-tp)) \qquad t>tp$$

where PO is the instantaneous point peak intensity and tp is the time of peak at a point. Regression equations were obtained from an analysis of 36 storms in the Hamilton area during the summer of 1980. They are

$$K2 = 0.0818 + 0.299K1$$
$$K1 = 0.101 + 0.0025PO$$
$$PO = 2.6 + 1.81TOTP$$

where TOTP is the total observed precipitation. Correlations were also obtained between the ground level wind velocity VW and the storm velocity VS as

$$VS = 7.39 + 0.933VW$$

Given the total precipitation, the wind velocity and the wind direction, THOR produces time averaged and space averaged hyetographs for any time step and subcatchment area. The model is thus capable of representing a moving storm tracking across the basin in any direction.

CONTINUOUS MODELLING

Perhaps the earliest and most convincing advocates of continuous modelling were Linsley and Crawford (13). These authors point out the futility of using design storm inputs for water quality modelling, where pollutant build-up is a function of antecendent dry-days (and, incidentally, the street cleaning time-series). Unfortunately their models (and most others, if not all models in the public domain) did not allow a variable time step and variable discretization during the model run.

There seems to be a fear that continuous quality modelling will remain expensive. In fact dry-day periods (i.e. no significant wash-off events) do not warrant highly disaggregated models with very fine time scales (14). As soon as a non-variable time step and non-variable discretization model is freely available, this criticism will be negated. Furthermore, if such models are run on inexpensive (ca $30K) 16-bit micro-computers with hard discs and a 0.5 Mbyte central memory (say), computing costs become all but negligible. Distribution of such computing power will be widespread in the near future.

Continuous modelling is preferred to discrete event methods because:

1. Current civil engineering practice using a design storm event (typically a synthetic hyetograph) does not specify a design antecendent dry period which is critical if quality is to be modelled properly (15).

2. No reliable probability can be assigned to the single event (13).

3. The most critical impact on the receiving waters does not necessarily occur under low-flow conditions due to the shock loads from urban runoff sources (14,16).

4. High frequency events which cause repeated combined sewer overflows have been found to produce larger annual pollutant loads than low frequency storms even though the low frequency storms have significantly higher peak flows and volumes. Consequently, storage/treatment facilities sized for the low frequency events will have to be much larger than facilities sized for the higher frequency events. The low frequency facilities may prove to be uneconomical when construction costs are compared with benefits occurring from reduced loadings. the higher frequency facilities may actually provide a more economical reduction in total annual loading.

It is essential therefore to account for hydrologic and meterologic variability in designing a water quality management scheme. In order that the interaction and the probability of occurrence of runoff and pollutant loading events of various magnitudes be taken into account the response of the urban drainage system must be examined on a continuous basis for as long a term as is practical given the available meteorologic record.

PROBLEMS IN CONTINUOUS MODELLING

Most currently available continuous water quality simulation programs operate on a constant, usually one-hour minimum, time step. Precipitation input to such a program is in the form of a long record of hourly totals. Given a set of parameters characterizing the hydrologic processes, the drainage system and the pollutant deposition characteristics, the program simulates mean flows, volumes of surface runoff and loadings of water quality constituents for each hour. While this technique is currently state-of-the-art, it is inadequate.

The effect of averaging an intense rainfall, occurring for, say, five to fifteen minutes, over a period of one hour, is to reduce the computed runoff intensity, to allow more computed infiltration to occur and to allow computed overland flow routing to become more significant. The consequence of this is that computed peak flows will be reduced and to some extent flow volume as well (3).

Pollutant washoff is dependent on runoff intensity, hence a reduction in this input intensity results in a smaller computed volume of pollutant being washed off during the event, and at a slower rate. These reductions in peak flow, flow volume and volume of pollutant washoff have impacts on design sizing of storage/treatment facilities. It has been the authors' experience that differences in peak flow and concentration introduced through data smoothing can range up to 40% between discrete event and continuous modelling results and can range between 10 and 20% where total pollutant loadings are concerned.

Often, the rainfall monitoring stations with sufficiently long-term rainfall records for conducting continuous simulation are located a distance away from the study area. Consequently there is a need to extrapolate data from these locations to the study area. Our experience has shown that summer thunderstorm type events vary considerably over short distances, consequently the use of such records as anything other than a general input time series is questionable, e.g. say for calibration.

Continuous modelling exercises have the potential for generating large volumes of output which must subsequently be analysed. It is therefore necessary to write computer routines which synthesize the output and carry out various tests and analyses on it. In some instances, e.g. SWMM, HSPF, some general statistical post-processing has been included. However, for specific cases, e.g. ARIMA modelling, it is necessary for the user to conduct the data manipulation.

DATANAL PACKAGE

In Canada the Atmospheric Environment Service's (AES) hydrometeorologic data is supplied in a form that is not suitable for direct input to large simulation programs such as SWMM (17,28). Consequently, it is necessary to manipulate the data into a suitable form. A data processing program package DATANAL was written specifically for interpreting, reformatting and carrying out a preliminary statistical analysis of data supplied by the AES on magnetic tape.

The program will interpret a range of hydrometeorologic parameters such as hourly rainfall, wind speed and temperature, supplied on magnetic tape. These data, which are usually in metric units, can be converted into English units if desired. The data is checked for flags indicating missing, estimated, etc. data and the program will fill in or modify according to user-defined criteria. The data can be reformatted into any user-defined form to make it suitable for input to programs for hydrologic or hydraulic analysis.

The program has included some simple statistical routines for calculating means, standard deviations, skewness, lag-1 auto-correlation and coefficient of variation. The modular structure of the program allows easy inclusion of additional processing routines.

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• MAY 26 AND 27, 1981
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CO-ORDINATED MULTIPROCESSING FOR LARGE SCALE DATA ACQUISITION AND SIMULATION OF URBAN DRAINAGE FOR THE HAMILTON-WENTWORTH REGION

by

William James (1) and Mark Robinson (2)

SYNOPSIS

The quantity and quality of stormwater runoff, combined sewer overflows and dry weather flow to the receiving waters of Coote's Paradise and the Hamilton Harbour, from the 120 square kilometer urban drainage system in the City of Hamilton, have been modelled. Various strategies for minimizing pollutant loadings to these receiving waters will also be studied.

The drainage system comprises 3 major drainage systems, with 15 combined sewer outfalls greater than 1.22 m diameter and many known active diversion structures. Five rainfall stations and five streamflow and water quality sampling stations provide data on rainfall intensity, storm direction, storm speed, flowrate, suspended solids, BOD5, total phosphorous and total nitrogen.

A minicomputer executes FORTRAN pre- and post-processor programs for a package of stormwater management models residing and executing on remote mainframes. The paper includes a brief description of the pre- and post-processors, sensitivity analyses and calibration of the RUNOFF quantity and quality block of the Stormwater Management Model (SWMM) for the study area.

A method is proposed by which a data acquisition network, based on microprocessors and transmitting via acoustic line to a central minicomputer can be used to develop and implement a real-time control system for minimizing pollutant loadings to the receiving waters.

KEYWORDS

Stormwater management, urban hydrology, water quality modelling, urban drainage, pollution control, real-time control.

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RESUME

Un modèle d'analyse de la quantité et la qualité d'écoulement des eaux pluviales ainsi que des trop-pleins d'égouts combinés au débit d'étiage provenant du réseau de drainage de la ville de Hamilton, fut établi. Ce réseau de 120 km^2 se déverse dans les eaux réceptrices de Coote's Paradise et du port de Hamilton. Aussi, on examinera diverses méthodes utilisées dans le but de minimiser les matières polluantes jetées dans ces eaux.

Le réseau d'égout se compose de trois principaux réseaux comprenant 15 émissaires dont les diamètres sont plus de 1.22 m, et plusieurs ouvrages de dérivation en activité. Cinq postes d'échantillonnage de débit et de qualité de l'eau procurent des données relatives aux éléments suivants: l'intensité des pluies; la direction et la vitesses des tempêtes; les matières en suspension; la demande biochimique en oxygène dissous 5; la teneur en phosphore et en azote.

On effectue des programmes FORTRAN de prétraitement et post-traitement pour un assortiment de modèles de gestion des eaux fluviales. Ces programmes proviennent, par télécommande, d'un terminal central. Cette communication présente une courte description du programme de prétraitement et post-traitement des analyses de sensibilité ainsi que du mesurage de quantité et qualité du bloc d'écoulement du modèle pour la région à l'étude.

Une méthode est proposée selon laquelle un réseau d'acquisition de données peut servir à établir et mettre en oeuvre un système de contrôle immédiat permettant de minimiser les matières polluantes jetées dans les eaux réceptrices. Ce réseau est constitué de micro-transformateurs communiquant par lignes acoustiques jusqu'à un micro-ordinateur central.

INTRODUCTION

Hamilton is possibly the most heavily industrialized city in Canada. In addition to the two primary steel making companies in Canada, Hamilton supports a population exceeding 300,000. The intense industrial activity and highly urbanized nature of the city strain the receiving waters bordering the city: the Hamilton Harbour and Coote's Paradise. As a result of their recreational and educational benefits to the city, concern has grown regarding the pollutant loading to these receiving waters.

Hamilton is naturally divided into two zones by the Niagara Escarpment. An elevation difference of about 350 feet exists between the old part of the city built around the harbour and the newer part of the city built on the plateau above the escarpment. Below the escarpment the stormwater drainage system is essentially a combined system while above the escarpment separated storm and sanitary sewers predominate. The combined sewer system comprises 290 known overflow or diversion structures and devices (DS) which, under high flow conditions (at which point the treatment plant may be reaching its capacity) divert combined sewage to other sections of the drainage system (relief sewers) or overflow directly to the outfalls at the receiving waters (CSO's). The system dynamics must be carefully studied to properly model the drainage system in terms of both water flow and pollutants. The model of the system must be disaggregated in space and time to account for the effects of all major DS and CSO structures (James, 1972).

Care should be taken to distinguish between a program (such as SWMM, frequently called a model) and the executable program (together with part of its data file) herein called the model. The relevant part of the datafile comprises only those parameters, input to the program, that characterize the physical drainage system to be modelled. Another part of the data file, dealing with the hydrometeorological inputs (rain, temperatures, snow, etc.) and not normally requiring discretization and parameter estimation, does not form part of the model considered here. However we do also suggest separate storm models for the proper descriptions of these hydrometeorological inputs (James and Drake, 1980).

Uncertainty is associated with the estimation of model parameters. Quantity and quality parameters have various effects upon the computed hydrographs and pollutographs. For example, a small variation in the value of one parameter may cause a large difference in one or more characteristics of the hydrograph or pollutograph. In this case, the model is said to be "sensitive" to this parameter. On the other hand, a large variation in the value of a parameter may produce only a negligible effect on the relevant characteristic of the output response. The model is then said to be "insensitive" to this parameter. Of course not all characteristics of the response may be relevant to the objectives of the modelling study.

It is important, prior to any modelling exercise, to evaluate model sensitivity. This allows the modeller to concentrate data abstraction effort on those parameters that most significantly affect the simulation objectives. Precise estimates of parameters which have only a minor effect on the results are not required. Sensitivity analyses were carried out on parameters for both the quantity and quality sections in the RUNOFF block. The results are summarized below.

In computer modelling, a large portion of the study effort must be devoted to the preparation of input data for the computer model and interpretation of the output results. Any stormwater modeling study must have a complete and up-to-date data base upon which to draw. The model results are only as reliable as the data upon which they are based, a point often overlooked by modellers. The proper use of a computer model for hydrologic analyses requires calibration and validation of simulation results against observed data for the study area in order to establish appropriate values of model parameters, prior to production runs.

Several previous engineering studies (Gore and Storrie Ltd., 1980; James F. MacLaren Ltd., 1978; Proctor and Redfern Ltd., 1978) conducted in the City of Hamilton, monitored rainfall, streamflow and water quality for short periods of time. All of these monitoring programs have been discontinued. There exists a well-defined need for implementation of a data acquisition programme operating on a continuous basis.

High-intensity, short duration thunderstorm activity has been observed to have considerable spatial variation over a drainage area (Huff, 1975; Smith, 1974). This variability has been found to have a significant influence on the hydrographs produced in an urban area (March et al, 1979; James and Drake, 1980; Wilson, et al, 1979) in terms of volume, peak flow and time to peak. We present below a preliminary investigation into the potential effect of insufficient temporal resolution on total pollutant loadings.

As part of this study, a data gathering network and a comprehensive reliable data base for stormwater studies in the City of Hamilton was established and documented. This has helped to identify data deficiences and can be used in the analysis of Hamilton's urban drainage system.

Most urban drainage and water quality modelling packages (eg. SWMM) typically execute on mainframes with large memory capacity and high computing speeds. As well, data bases of rainfall, streamflow and water quality observations have been created by various government departments. A typical problem faced by consultants is how to economically implement these packages efficiently and to integrate them with the available data bases.

Many users of these packages could be classed as "casual" users (Cuff, 1980) i.e. they have a good grasp of the concepts being modelled but since they use the model infrequently, they do not easily recall how to execute the model on the system. Hence, casual users tend to shy away from fully exploiting the modelling resources and the system capabilities. For example, a user may avoid interfacing SWMM with data bases or other models describing in detail an individual process (requiring a working knowledge of system JCL) in favour of the single, general model which deals with the processes in a more superficial way but which requires little involvement with the computer system. Kennedy (1975) identified this problem as a major roadblock to the acceptance and full implementation of a computer system.

This paper discusses an interactive package of pre- and post-processing programs developed to facilitate input data preparation, job excution, output interpretation and job resubmission in a multi-program remote batch environment.

Real-time control of an urban drainage system requires interfacing of the data collection network with remotely controlled devices. The operating policy for these devices is dependent on some decision criteria, typically the depth of flow at the control location. A real-time control system must integrate the available data with a predictor model for the decision variable.

MONITORING HAMILTON'S URBAN ENVIRONMENT

Three principal drainage basins in the city have been identified: Chedoke Creek, draining a total area of approximately 27.5 sq. km. (10.6 sq. mi.) to Coote's Paradise in the west, Redhill Creek draining an area of approximately 64.0 sq. km. (24.7 sq. mi.) to Windermere Basin in the east and the Central Business District draining approximately 28.0 sq. km. (10.8 sq.mi.) directly to various CSO's in the Hamilton Harbour. Coote's Paradise and Windermere Basin drain ultimately to the harbour.

Essential to any stormwater modelling study is acquisition of accurate measurements of rainfall intensity throughout the study area. It has been found (James and Drake, 1980) that storm kinematics can have significant effects upon the response of a watershed to a storm. Pronounced physiographic variations, as is the case in Hamilton with the presence of the escarpment, can effect the spatial distribution of rainfall as well.

During the 1980 field programme three rainfall monitoring stations were established in the Chedoke Creek Basin. These stations comprised a tipping bucket raingauge, strip chart recorder, power source and a totalizing raingauge. Rainfall was monitored at a 1-minute time interval. The rainfall records for each gauge have been translated into 1-minute and 5-minute intensities for subsequent use in the computer modelling. Additional data were obtained from the Atmospheric Environment Service's (AES) meteorological stations at the Hamilton Airport and the Royal Botanical Gardens (RBG). These stations are located several miles outside the Chedoke Creek Basin. The continuous data measured at these stations are the only long-term records available for Hamilton. These stations provided rainfall data on a 5-minute time interval, as well as measurements of wind velocity for use in development of the kinematic storm model. Based on the data collected during the field programme, several observations can be made regarding the characteristics of the rainfall.

Rainfall was observed to occur on 57 different occasions during the summer 1980 field programme. An analysis was made of this data to determine a level of rainfall below which direct runoff was not observed to occur. Since the main objective of this study is the prediction of pollutant loadings to the receiving waters, rain events should be defined as those producing pollutant washoff. Washoff is unlikely to occur in the absence of direct surface runoff. It was found that rainfall of at least 1.2 mm was required to generate a vertical or steep rising limb in the hydrograph. The number of dry days preceding an event is determined by totalling the number of days between successive occurrences of rainfalls equal to or exceeding 1.2 mm. This has significant implications with regards to pollutant modelling since total loadings have been found to be sensitive to the number of days during which pollutants have been accumulating.

Precipitation data from the five gauges displayed great variability. The total precipitation and peak intensity differ markedly between the stations situated in the urban area and the AES stations. Differences were also noted within the urban area itself as well.

The general storm motion appears to be from the southwest to the northeast with several convective storm cells of different size and intensity growing and dying over the study area.

Rainfall intensity data for each rainfall monitoring station was cross-correlated with the intensity data for each of the remaining stations as well as the streamflow monitoring stations. In general the correlation between the rainfall intensity measured at the two AES gauges situated outside and our gauges in the study area was sufficiently high to indicate that when rain occurred at these stations rain was also occurring within the study area. However, in most cases the intensities were not comparable. Moreover significant lower correlation with the flow data was observed for the two AES gauges. It appears that without taking into account storm kinematics, the two AES gauges will not adequately represent the proper spatial and temporal distribution of thunderstorm rainfall within the city. The implications for drainage design in Hamilton, based as it is on frequency analysis and continuous simulation using the AES data, are not yet clear.

Recording streamflow gauges were established at five locations throughout the city. Three gauges were installed in the Chedoke Creek basin. Two gauges were installed in the Redhill Creek Basin. A total of 30 runoff events were recorded at one or more gauges.

Water quality samples were collected at the five streamflow monitoring stations, as well as at several points in the outlet channel of Chedoke Creek in Coote's Paradise, for six separate storm events. All samples were analyzed by the Ontario Ministry of the Environment (MOE) for BOD5, total-N, total-P and suspended solids. Of these, four events were thunderstorm type events.

In order to examine the effect of inadequate synchronization and temporal resolution on the total pollutant loadings generated for thunderstorm events, the observed hydrographs and pollutographs of suspended solids were lagged against one another in time and the resulting effect on total pollutant loading was observed. Forward and backward lags of 5, 10, 15 and 30 minutes were applied to the observed hydrographs and the resulting total suspended solids loadings were calculated over the observation period. The difference in total loading due to the lagging was computed as a percentage of the observed loading.

The results show that as the lag increases or decreases due to either poor synchronization or poor temporal resolution of the rainfall input, the maximum error in pollutant loading can increase dramatically. For a 5-minute lag between observed hydrograph and suspended solids pollutograph the maximum percent difference in loadings ranged from 4 to 10 percent for the storms considered. An increase in the lag to 15-minutes resulted in maximum differences ranging from 12 to 35 percent. At a lag of 30 minutes, maximum differences ranged from 26 to 61 percent. The maximum difference for any given lag time was found to decrease as the total loading increased.

In order to minimize the error in computation of pollutant loading the hydrographs and pollutographs should be synchronized to within at least 5-minutes. This implies a maximum temporal resolution of 5-minutes for runoff and pollutant simulation.

The peaky nature of thunderstorms and the resulting urban runoff hydrographs make it essential that the time base of all recording instruments be synchronized or simulated discharge, time to peak and pollutant concentrations cannot be calibrated. It is also important for calibration that both the spatial and temporal variability of the rainfall be accounted for.

HYDROLOGIC SIMULATION OF THE CHEDOKE CREEK BASIN

The data acquired during the 1980 field programme was used to calibrate the model. Any calibration exercise requires prior sensitivity analyses. Clearly insensitive parameters can usually be assigned default values. Emphasis must be placed on estimating sensitive parameters.

Sensitivity analysis and calibration can often be a time-consuming, soul-destroying effort if the program package operates in a cumbersome manner using fixed formats for input data and requiring system JCL to execute the program, and/or integrate established data bases. A solution to these problems is provided by the FASTSWM package.

FASTSWM - AN INTERACTIVE CONVERSATIONAL PROCESSOR FOR SWMM

The FASTSWM package comprises three parts: pre-processor program, the large externally-supported simulation package (SWMM) and post-processor programs. The pre-processor functions are:

1. Solicit and accept user-directed input required by the external package, in conversational free-format mode, usually from a remote terminal.

2. Solicit and accept design-oriented input data (if any), such as a rainfall or temperature data base, from input devices or units as directed by the user for the evolving design.

3. Check the validity of the input and return values of certain key dependent parameters to the user's terminal.

4. Convert the units of the input data (usually metric) to units required by the main program package.

5. Convert the free-format user input to the formats required by the external program, i.e., from line-oriented free-format to rigid arbitrarily fixed formats.

6. Construct a System Job Control Language file to submit the main program to the batch CPU together with this fixed format data file.

7. Save uniquely identified input files in the user's catalogue to be subsequently modified, archived or destroyed.

The function of the external package is to simulate the requisite hydrologic processes. The post-processor's function is to:

1. Return selected output to the user's terminal upon program completion and after reconverting (if necessary) to various units.

2. Calculate various correlation functions between simulation results and observed or expected data (if any), to evaluate the current solution.

3. Printer-plot the results (if requested) where applicable.

4. Reinitiate pre-processing for further exploration of the solution space once the data are modified.

These processors are arranged outside the main external package; any updated version of the package coding is simply substituted for the old version. The processors are easily modified to incorporate any I/O changes.

Our dialogue procedures are primarily concerned with alphanumeric data entry, but the user may take control of the procedure by entering any of the following commands in place of data:

BACKSTEP	(start this step over again)
DEADSTART	(start the entire engineering design package over again)
PROMPT	(alternately switch prompting off/on)
ECHO	(alternately switch echo checking off/on
END	(terminate procedure)

This list does not include procedure instructions such as "Submit" or "Plot".

To accord with larger program packages supported by several U.S. Government Agencies, commands are expressed in at least the first three columns of the data line, while columns 4 through 80 are used for numeric data, keywords and data file commands. Commands, when required, are started in the first space and may be curtailed after the third letter; only three letters are necessary to identify the command.

When entering data the user should first leave three spaces blank as these spaces are reserved for commands. Spaces or commas are used to separate data. When all data corresponding to one prompt are entered, the line ends with a carriage return. When prompts for entering data are printed at the terminal they are followed by a reference to "group N". The group descriptions correspond to program documentation.

The data can be written in any format, but at least one blank space or comma must be left between data items. If two commas occur together, the procedure will insert a zero for the expected data item, a convenient type of shorthand for the experienced user.

A decimal is required for numbers containing fractions, but not for whole numbers. Keywords and comments can be interspersed with the data to describe individual data items. The input translation routine will disregard alphabetic input where it expects only numeric data. This is convenient for annotation for archiving.

Pre- and post-processor dialogue requires short bursts of CPU activity and demands fast response. On the other hand the external simulation package usually requires considerable CPU resources, and so does not qualify for high priority. It is clearly better to separate these activities placing each on a CPU dedicated to that environment. Unless an active CPU is dedicated to one user, the interactive procedures function most efficiently in a multi-processor environment.

Minicomputers have been found to have a number of advantages over large mainframes (Brebner 1973, Gaines 1975):

1. The machine can be dedicated for an appreciable time-slice entirely to the user.

2. The cost of computing on a microcomputer system is negligible.

3. File storage in the user's catalogue is essentially free and not restricted as to number and size of files, especially when the user mounts his own discs.

4. Each user requires only a basic system consisting of a terminal and microprocessor CPU and disc drive.

5. Peripheral devices can be located in close proximity to the user and to one another even if the number crunching is done at a remote site. Therefore the user can have instant access, assistance from similar users and up-to-date documentation.

6. Microcomputer systems are now readily available with 32K to 256K decimal words of memory at reasonable cost.

An arrangement whereby all pre- and post-processing is carried out on a local minicomputer and all intensive number crunching is carried out on a large remote mainframe optimizes the total design turnaround time.

At McMaster University the pre- and post-processors are being implemented on our PDP 11/23. This microcomputer uses

16-bit words and has a user area of 32K decimal words of memory. The user interfaces with the machine using a VC1452 CRT terminal communicating at 9600 baud and a LA120 or letter quality line printer communicating at 1200 baud. The external package executes on a remote CYBER 170/730 mainframe with 140K octal 64-bit words of memory. All softare is written in fully transportable ANSI FORTRAN.

The pre- and post-processing system has been used extensively for both sensitivity analysis and calibration. The processors facilitate modification of data and interpretation of output results.

SENSITIVITY OF THE SWMM-RUNOFF BLOCKS

The FASTSWM package was used extensively in executing multiple runs during the sensitivity analyses performed on the quantity and quality algorithms of the SWMM-RUNOFF block. In general the quantity algorithms in the RUNOFF block were found to be most sensitive to the percentage imperviousness of the subcatchment. An almost linear relatioship exists between percentage imperviousness and both peak flow and volume. Percentage imperviousness for an area can be estimated quite accurately given up-to-date mapping (land use plans or aerial photography).

The model displays moderate sensitivity to the subcatchment width. A number of techniques have been proposed for estimating this width but none have proven applicable in all situations. We feel it is preferable to use this parameter to calibrate the model since it incorporates inherent uncertainty.

The quantity model parameters may be listed in order of decreasing significance in terms of sensitivity:

- percentage imperviousness
- subcatchment width
- infiltration rates
- detention storage
- ground slope
- percentage of impervious area with zero detention storage

The quality section of the RUNOFF block was found to be sensitive to changes in parameters affecting suspended or settleable solids. Examination of the equations used in the model algorithms indicates that a variation in the amount of suspended or settleable solids washed off will have an effect, potentially overwhelming, on the amount of other constituent pollutants washed off.

The parameters affecting RUNOFF quality simulation and the degree to which the model is sensitive to them are summarized below:

High Sensitivity:
- number of dry days

- street sweeping interval
- street sweeping efficiency
- exponential coefficient in washoff equation at low runoff rates
- dust and dirt loadings
- insoluble fraction due to suspended solids
- availability factors for suspended and settleable solids
- total gutter length

Medium Sensitivity:

- street sweeping availability factor
- insoluble fractions due to settleable solids
- pollutant fractions

Low Sensitivity:

- catchbasin storage volume
- initial concentration of BOD5 in each catchbasin
- number of catchbasins

Based on the sensitivity analyses, calibration of the model of the Chedoke Creek drainage system was undertaken. The results showed excellent agreement with the observed data. Validation using the remainder of the data obtained during the summer field program is currently being carried out.

PROPOSED REAL-TIME CONTROL SYSTEM

Real-time control of an urban drainage system usually involves diverting flow from one location to another where sufficient capacity is available to prevent flooding at the first location. These diversions are most often accomplished using some form of gate which controls an overflow or diversion structure. At some pre-determined level of flow the gate mechanism is activated and part or all of the remaining storm flow is diverted, typically directly to the receiving waters.

Our proposal is that these diversion structures could be coordinated using microprocessors and acoustic lines to operate such that the hydraulic capacity of the drainage system is utilized to its fullest. In this way overflows to receiving waters are minimized. A calibrated model of the drainage system producing reliable results, on a mainframe computer, can first be used to develop simple relations for real-time prediction of flow (and hence water level) at a given control location. Statistical analyses (of the output from the continuous deterministic model) such as multiple linear regression, can be used to derive simple prediction equations.

These equations can be programmed into the field microprocessors in machine language together with code representing the control device's operating policy. The real-time control system could be designed to operate in the following manner. The rainfall and streamflow monitoring network transmits data to the PDP 11/23. The minicomputer processes the data into units compatible with the microprocessor predictor equations (eg. number of raingauge tips to intensity in mm/hr.). This data would then be transmitted by the minicomputer to the microprocessor memory, where together with water level data, appropriate action would be taken.

CONCLUSIONS

The 1980 field program was operated successfully for a 7 month period. Three rainfall monitoring stations and five streamflow gauging stations were established in the city. Fifty- seven rainfall events were recorded during the program. Thirty runoff events were recorded of which six were sampled for water quality parameters: suspended solids, BOD5, total-N, total-P.

The rainfall intensity and volume differed significantly between gauges established in the urban study area and those in the surrounding area. Storm characteristics also exhibited considerable variation within the urban area.

Streamflow and pollutant loadings were found to be sensitive to synchronization errors in recorder timing. A maximum temporal resolution for the data of 5-minutes was required to limit maximum potential errors in total loading to about 10%.

The physical characteristics of the city's drainage system have been abstractd from topographic maps and sewer construction drawings in a form suitable for input to the model of the urban drainage system. These data files have been archived on our computer for continuing reference. The model has been calibrated using the FASTSWM package against data observed during the 1980 field season. FASTSWM constitutes a package of pre- and post-processing programs for the SWMM program. The pre- and post-processors create input data files for job execution; the post-processors operate on the output from the simulation package, synthesizing, analyzing and plotting to speed user interpretation at his terminal. The original coding i.e. the batch oriented package is not altered. The FASTSWM package has been found to greatly improve design turnaround time when carryng out sensitivity analyses or calibration. The calibrated model will be used to simulate and evaluate various stormwater management strategies.

A system is proposed whereby the calibrated model of the drainage system is used to develop easily evaluated relations for real-time prediction of flow at a specified location based on observations of rainfall intensity and discharge at other monitoring stations. These relations will be programmed into microprocessor memory. Each microprocessor in turn would control a combined sewer overflow device. These diversion devices will be linked via acoustic line to a central minicomputer which would process incoming signals from the rain and flow gauges. This information will be transmitted to the control devices which would divert flow in a prescribed direction.

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PART 2: TRANSIENTS
(10 papers)

I wrote and produced all the papers in this part. In TR3 the co-author was an M.Eng. student working under my direct supervision. In TR10, the co-author was the contract liaison officer and he played very little part in the research. In all cases, the original idea, the lines of research, and the theoretical and experimental procedures to be followed, were laid down by me, and the basic analysis carried out by myself. In several cases, research assistants were used who had no previous experience in these fields. They were generally responsible for carrying out the experimental procedures and undertaking the computation. In all cases, the design of the equipment, the design of the experimental procedures and the interpretation of the results was my own. Much of this work was based on further exploration of the ideas developed during my Ph.D. studies at the University of Aberdeen, and was carried out at the University of Natal in Durban, Queen's University and McMaster University in Canada.

EXCEPTION:

Paper TR7: The idea arose out of a brief discussion in Hawaii with Dr. R.P. Shaw of the State University of New York in Buffalo, but all of the experiments were carried out by me.

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Resonator studies for Kincardine Harbour, Lake Huron

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Resonator studies for Kincardine Harbour, Lake Huron

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A 1:600 scale acoustic model of Kincardine Harbour originally built by the National Research Council of Canada (NRCC) was delivered to the Applied Dynamics Laboratory at McMaster University late in 1976. After preliminary testing, a second acoustic model was built to a scale of 1:200 in order to narrow the range of acoustic wave frequencies required to simulate the observed lake wave climate. Scale selection and the necessary acoustic frequency band are discussed. The response at eight locations inside the model harbour was measured in this frequency band and the harbour wave amplification determined. A comparison between the acoustic model results and the hydraulic model results (previously carried out by NRCC) is presented.

A public opinion survey of recent users of Kincardine Harbour was carried out during the winter of 1976–1977. The purpose of this survey was to identify potential problems in the harbour. The survey was focused on (a) harbour entrance and resonator, (b) offshore breakwater, (c) rubble-mound breakwater, and (d) inner harbour. The results indicated that the users were generally of the opinion that the resonator had decreased inner harbour oscillations, but that it presented a navigational hazard, particularly at night.

On the basis of the survey, a utility function was proposed; it indicates an average condition of the harbour in relation to outside wave conditions. Tests on the acoustic model were then carried out. Results of those tests showed that model beach reflectivity was comparable to that of the prototype, but reflectivity of the model breakwater was relatively low. When resonators were installed wave amplification in the harbour was reduced.

C'est le Conseil national de recherches du Canada (CNRC) qui a été le premier à construire le modèle acoustique de Kincardine Harbour à l'échelle de 1:600 et qui a été délivré au Applied Dynamics Laboratory de McMaster University vers la fin de 1976. Suite à des essais initials, un deuxième modèle acoustique a été construit à l'échelle de 1:200 afin de diminuer les fréquences d'ondes acoustiques requises pour simuler le climat des vagues observé sur le lac. On discute le choix d'échelle et de la bande de fréquences acoustiques nécessaire. La réaction a été mesurée avec cette bande de fréquences dans huit endroits à l'intérieur du modèle du port, et l'amplification des vagues dans le port a été déterminée. La comparaison entre les résultats du modèle acoustique et ceux du modèle hydraulique (déjà effectuée par le CNRC) est présentée.

Une enquête sur l'opinion publique des personnes ayant récemment utilisé le Kincardine Harbour a été exécutée au cours de l'hiver 1976–1977. Cette enquête avait pour but d'établir les problèmes possibles dans le port. L'enquête visait (a) l'entrée du port et le résonateur, (b) le brise-lames au large, (c) le brise-lames en remblai et (d) l'arrière-port. Les résultats ont indiqué que la plupart des usagers étaient d'avis que le résonateur avait diminué les oscillations de l'arrière-port mais qu'il présentait un danger de navigation, surtout la nuit.

A la lumière de cette enquête, on a proposé une fonction d'utilité qui indique l'état moyen du port relativement aux conditions des vagues au large. Des essais ont ensuite été effectués sur le modèle acoustique. Les résultats de ces derniers ont démontré que la réflectivité du modèle de la plage était comparable à celle du prototype, mais la réflectivité du modèle du brise-lames était assez basse. Lors de l'installation des résonateurs, l'amplification des vagues dans le port a été atténuée.

Can. J. Civ. Eng., 7, 133–143 (1980)

[Traduit par la revue]

Introduction

Wave action in Kincardine Harbour on Lake Huron became excessive for the mooring of pleasure craft after the original concrete quays were shored up by steel sheet piling. Figure 1 shows the location of Kincardine Harbour on Lake Huron, and Figs. 2 and 3 show the general arrangement of the harbour area. Part of the resonator is missing in Fig. 3. A breakwater is located offshore to provide protection from the northwesterly wave action. A hydraulic model study of this harbour was undertaken by the National Research Council of Canada (NRCC) in 1971 (Prandle 1972). In 1972 a preliminary acoustic model study of Kincardine Harbour was also carried

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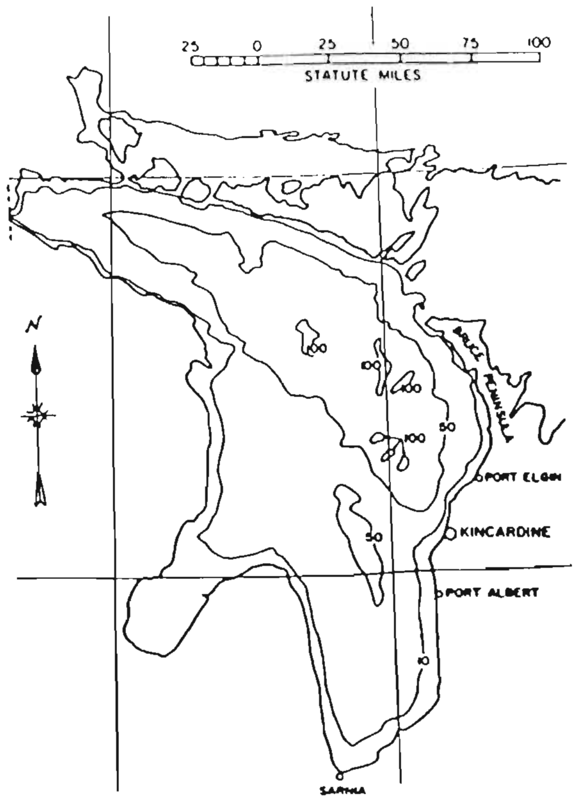


SOUNDINGS IN FATHOMS

FIG. 1. Location of Kincardine on Lake Huron. Notes: 1 mi = 1.6 km; 1 fathom = 1.8 m.

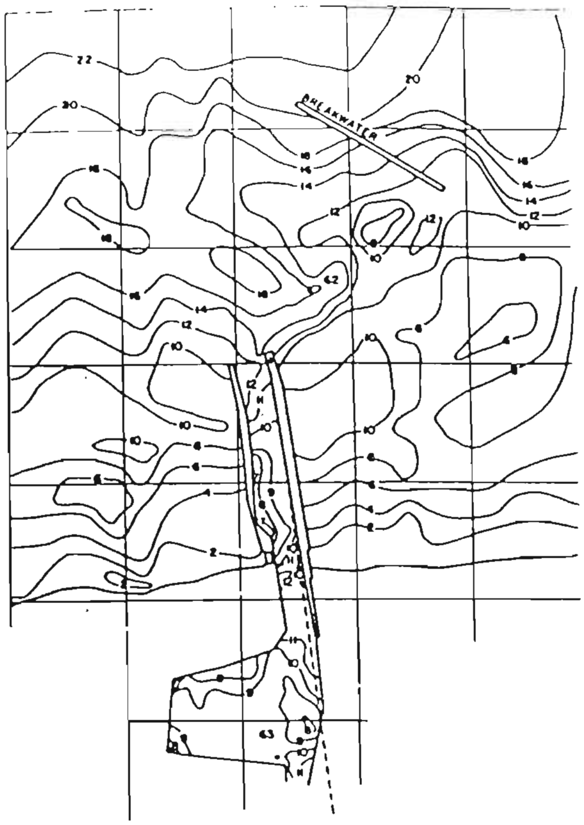


FIG. 2. General arrangement of Kincardine Harbour showing locations of pressure cell wave recorders.

out by NRCC following advice by the primary author. Results of the acoustic model were checked against the hydraulic model study. Subsequently, a single resonator was built in the harbour entrance channel as a means of excluding wave energy from the harbour. The resonator is depicted in Fig. 4 and also from ground level in Fig. 5. A rubble-mound breakwater was also built inside the harbour for more protection as shown in Fig. 4.

More recently, Public Works Canada considered it worthwhile to evaluate both the wave conditions in the improved harbour and the reaction of the users to the resonator. Accordingly, both a laboratory study (James and Kassem 1977*a*; James and Cheung 1977) and a public opinion survey (James and Kassem 1977*b*) were carried out. The main purpose of the public opinion survey was to define potential problems in the harbour and to establish a suitable utility function for oscillations within the harbour and in the harbour entrance.

A few comments on acoustic modelling may be of interest to readers. The propagation of sound waves in air is analogous to that of water waves. The speed of sound in air is constant for constant temperature and humidity (1125 ft/s (342.9 m/s) at 20°C) (Raleigh 1877). The phase velocity or celerity of water waves is similarly constant in water of constant and shallow depth. Reflection and diffraction of sound waves are also analogous to those of water waves (see, for example, any of a number of introductory texts on wave mechanics).

Acoustic model tests in harbour resonance were first compared with hydraulic model tests in 1970 (James 1970*a*, 1971). An acoustic model evidently has certain advantages over a hydraulic model: wave generation and absorption are easier; the fluid does not have to be isolated; wavelengths are generally shorter; and measuring equipment, speed, and accuracy are generally better.

In the 1970 experiment, earlier experiments on resonators in a water wave channel were reproduced. The effect of imperfect sound absorption was checked and found to be unimportant. Tests on scale effects were also negative, although these were not exhaustive. In addition the effects of sound waves entering the duct along its length were also found to be insignificant. The signal-to-noise ratio was easily improved by means of the amplifier on the oscilloscope.

The results showed slight differences from those

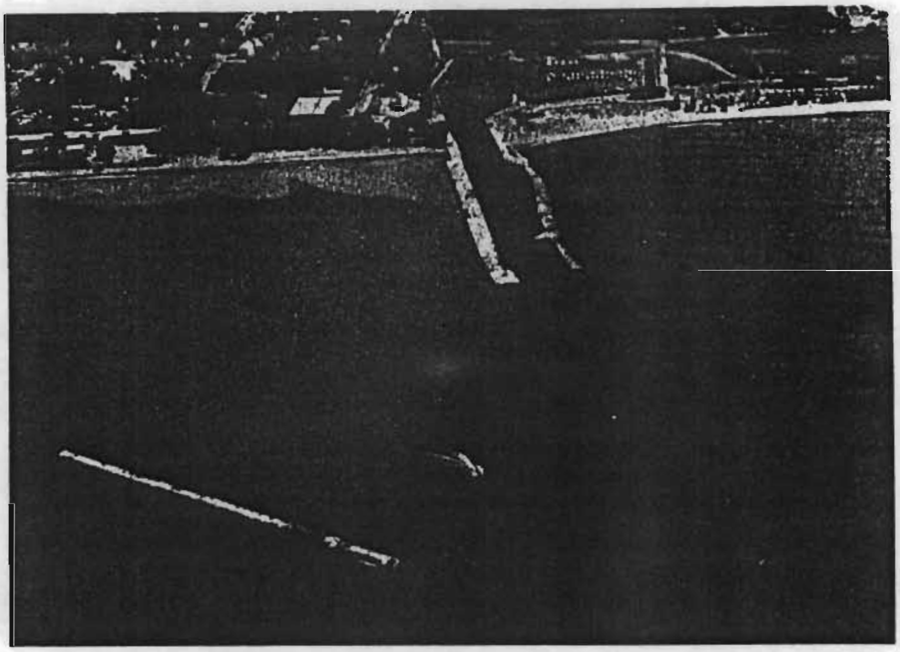
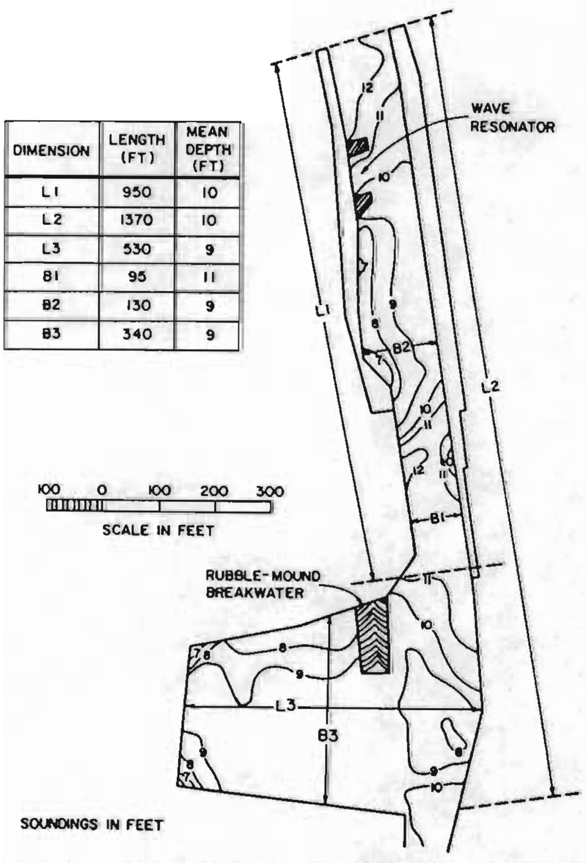


FIG. 3. Aerial view of Kincardine Harbour (elev. 300 m, August 1979).



DIMENSION	LENGTH (FT)	MEAN DEPTH (FT)
L1	950	10
L2	1370	10
L3	530	9
B1	95	11
B2	130	9
B3	340	9

FIG. 4. Detailed dimensions of inner harbour and inlet channel for Kincardine Harbour. Note: 1 ft = 0.3048 m.

of the water wave experiments. Reasonably accurate predictions of harbour resonance modes for various frequencies were obtained. It may be noted that the results were obtained in a few hours (several months in the hydraulic model) and at negligible expense.

Using this analogy between sound waves and water waves, an acoustic model study of Kincardine Harbour was carried out by the authors. Results of the acoustic model could be checked against the previous studies as well as against the wave spectra recorded by the Marine Environmental Data Service (MEDS) inside and outside the harbour in 1971 and 1973.

Water Wave Spectra at Kincardine Harbour

Wave spectra were determined by MEDS using pressure cells inside and outside the harbour in 1971 and again in 1973 after the resonator was installed. The locations of these wave recorders are shown in Fig. 2 (stations 62, 63). Readings were taken at both locations simultaneously. A total of 453 readings were obtained at station 62 between October 20, 1973 and December 18, 1973. A total of 496 readings were obtained at station 63 between October 10, 1973 and December 18, 1973. Wave data obtained in 1973 were analyzed by MEDS.

The original record of water surface fluctuations were not available at the time of this study, but MEDS have now reconstituted the original water level record from the available spectra, making due allowance for the frequency-dependent effect of

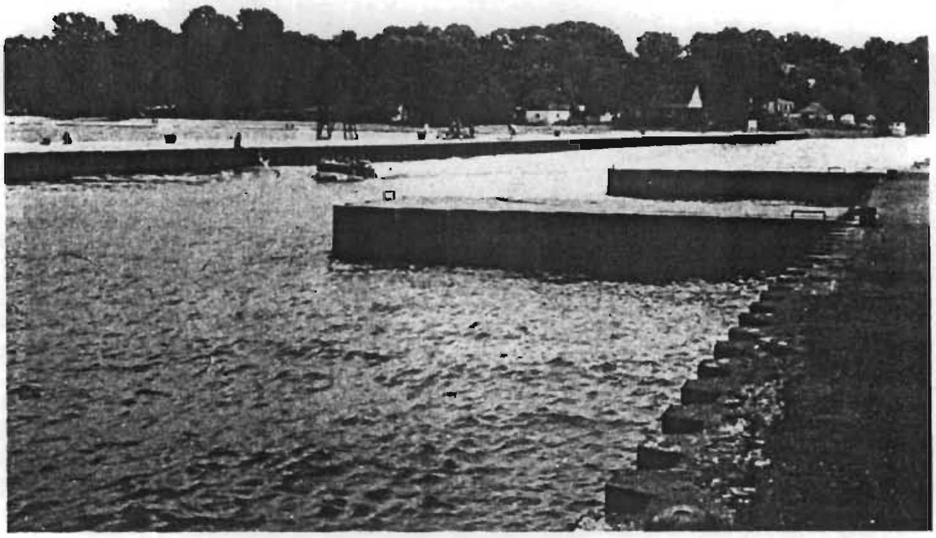


FIG. 5. Ground-level view of the single resonator (1976).

depths on the pressure cells. In any case, it is not possible to utilize such data in an acoustic model without developing expensive instrumentation. Such data would be invaluable in a numerical study, of course, but a numerical model is not covered in the present paper.

The average water depth inside the harbour is approximately 10 ft (3.0 m) and that outside is approximately 15 ft (4.6 m). The depths and other dimensions are listed in Fig. 4. Wave periods observed by MEDS covered the range 3–10 s. Wavelengths in the harbour can be calculated by the well known first-order expression:

[1] $L = (gT^2/2\pi) \tanh (2\pi d/L)$

where g = acceleration due to gravity; T = wave period; and d = still water depth. Assuming a constant depth of 10 ft (3.0 m), the wavelengths vary between 41.8 and 176.9 ft (12.7 and 53.9 m) as follows:

Time (s)	Wavelength (ft (m))
3	41.8 (12.7)
4	62.6 (19.1)
6	101.7 (31.0)
8	138.4 (42.2)
10	176.9 (53.9)

Selection of Scale for Acoustic Model

An acoustic model must be designed to simulate the response of the harbour to the incident wave spectra. The resonance problem is caused by the harbour geometry and the reflectivity of the walls. These must be accurately modelled. Other requirements and limitations are: (1) laboratory space available; (2) the model ocean domain must be long enough for the radiating incident waves to achieve an approximately parallel and rectilinear wave front at the harbour entrance (as they do in the lake); (3) acoustic wave-generating and measurement equipment available must match the frequencies and dimensions required; and (4) total costs must be reasonable.

The scale of the model (S) is determined by the ratio of the wavelength of the acoustic model (L_m) to that of the prototype (L_p). Hence

[2] $S = L_m/L_p$

where

[3] $L_m = C_m T_m$

(from elementary wave-mechanics), and

[4] $L_p = C_p T_p$

where T_p is the period of water wave in the prototype; T_m is the period of sound wave in the model; C_p is the celerity of water wave; and C_m is the speed of sound in air.

The scale of the acoustic model should be chosen such that the sound equipment is able to respond properly within the range of wavelengths and frequencies required. As mentioned before, the prototype has: range of periods, T = 3–10 s; range of frequencies, f = 0.1–0.33 Hz; and range of wavelengths, L = 41.8–176.9 ft (12.7–53.9 m).

The corresponding ranges of wavelengths and frequencies that the model and the sound equipment should respond to at different scales are given in Table 1.

Sample Calculation

Scale chosen is 1/500; range of wavelengths of the real harbour is 41.8–176.9 ft (12.7–53.9 m). Thus the range of wavelengths of the model calculated by equation [2] is: 41.8/500 to 176.9/500, i.e., 0.08–0.35 ft (0.02–0.11 m). The relation between the speed of sound in air and the air temperature, from Raleigh (1877) is:

$$[5] \quad C_1/C_2 = (T_1/T_2)^{1/2}$$

where C is the celerity of sound wave and T is the temperature. The speed of sound in air at 20°C = 1125 ft/s (343 m/s) and the speed of sound in air at 25°C = 1135 ft/s (345.9 m/s). The range of frequencies for the model is: 1135/0.08 to 1135/0.35, i.e., 14.2–3.2 kHz. (Note: $f = C/L$.)

It can be seen from Table 1 that for a model built to a scale of 1:600 (the scale of the original acoustic model built by NRCC), the range of frequencies required is 3.8–16.3 kHz. Inexpensive audio microphones cannot respond reliably to frequencies greater than about 13 kHz. Also, inexpensive speakers do not have a reasonably flat response over this wide range of frequencies.

For the space and equipment available, it was desirable to rebuild the model to a scale of 1:200. In this case, the range of frequencies required for the acoustic model was 1.3–5.4 kHz, and inexpensive audio equipment would provide a flat response.

Acoustic Model of Kincardine Harbour

The acoustic model built in the Applied Dynamics Laboratory at McMaster University in February 1977 is shown in detail in Fig. 6. The model harbour was housed inside a wooden chamber 16 ft × 8 ft

TABLE 1. Model wavelengths and frequency ranges

Model scale	Range of wavelengths, L (ft)	Range of frequencies, f (kHz)
1/100	0.42–1.77	2.7–0.6
1/200	0.21–0.88	5.4–1.3
1/300	0.14–0.59	8.1–1.9
1/400	0.10–1.44	10.8–2.6
1/500	0.08–0.35	13.6–3.2
1/600	0.07–0.29	16.3–3.8
1/700	0.06–0.25	19.0–4.5
1/800	0.05–0.22	21.7–5.1
1/900	0.05–0.20	24.4–5.8

NOTE: 1 ft = 0.3048 m.

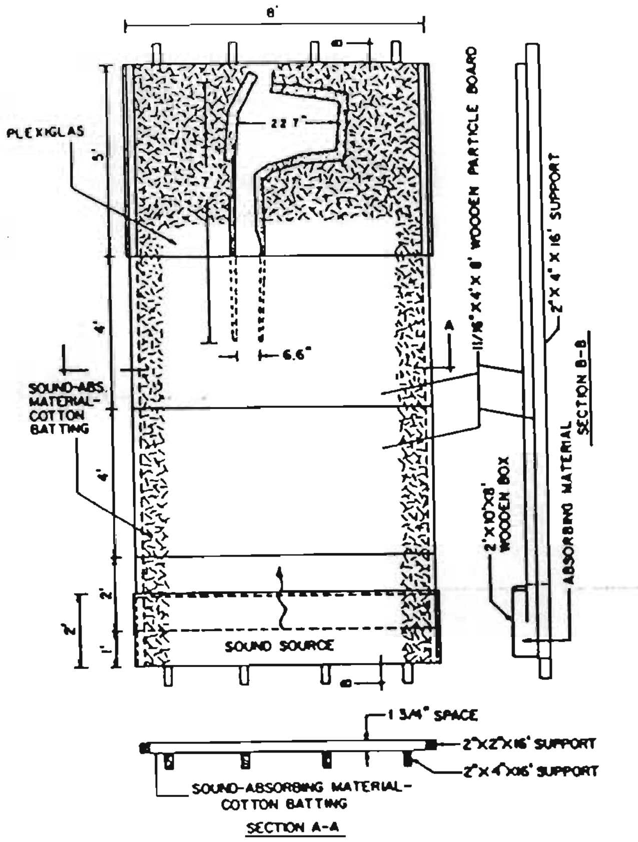


FIG. 6. Detailed dimensions of acoustic model for Kincardine Harbour.

(4.9 m × 2.4 m) in plan and approximately 1.75 in. (44.5 mm) high. The top part of the chamber was made from 0.5 in. (12.7 mm) Plexiglas. The harbour configuration was made of wood to a scale of about 1:200. Holes were drilled in the Plexiglas top to fit a microphone so that sound could be recorded at different locations outside and inside the model harbour. Cotton batting was used as a sound-absorbing material along the sides of the model and the shoreline. The cotton batting was made more dense near the external edges, with decreasing density towards the interior of the model. The sound was introduced into the chamber from the lake or ocean edge, using speakers.

Preliminary Tests of the Acoustic Model

Preliminary tests were carried out to examine the performance of the acoustic system. In these tests the speaker was first connected to the signal generator through an amplifier. The signal was detected through the microphone and the oscilloscope.

All instrumentation used was regular, inexpensive equipment available on the audio market. In the initial tests carried out in the open air a dome tweeter, microphone, and amplifier were selected such that the response to the signal generator on the oscilloscope was essentially linear.

Tests were then carried out to determine the attenuation and reflectivity of the walls and absorbers in the acoustic model. This was done by measuring the variation of response when a sound source was directed into a long narrow channel. These tests were carried out under two conditions: (a) closed end, and (b) open end. The one-dimensional channel was placed in the model semi-infinite ocean domain (i.e., where sound is totally absorbed at all edges) and tested inside the laboratory facility used for the final tests.

The channel was constructed so as to run the full length of the model, approximately 16 ft (4.9 m), and measured 1.75 in. × 1.75 in. (44.5 mm × 44.5 mm) in cross section. The sound source (dome-tweeter) was introduced at one end of the channel and measurements of response were made at three separate locations. The first location was 2 ft (0.6 m) from the sound source to avoid high intensities, with the second and third measurement stations 5 and 8 ft (1.5 and 2.4 m) respectively from the sound source. Cotton batting was placed in front of the sound source to absorb reflections of the reflected wave.

Closed-end and open-end one-dimensional channel conditions were tested. For the closed-end condition, a wooden block of the same material used to construct the walls of the acoustic model was placed to completely close the remote end of the channel. For the open-end condition, cotton batting was used to absorb the sound waves. The frequency range of 1–6 kHz was chosen to avoid transverse resonance in the channel.

By assuming that only two waves (the incident and the reflected waves at the same frequency) exist in the narrow channel, reflectivity and attenuation can be calculated (James 1970*b*). The reflectivity of the absorber was comparable to that of the prototype and the hydraulic model (less than 10%) but the reflectivity of the wooden walls (about 35%) was probably low. The wooden walls of the channel evidently absorbed some acoustic energy or allowed some to escape.

Test of Acoustic Semi-infinite Domain

A series of tests was then carried out inside the chamber without the model harbour or channel structure. Cotton batting was placed across the remote end of the model and sound was emitted into the chamber. These tests were carried out to: (a) test the efficiency of the cotton batting in absorbing sound and to insure no (or little) reflection from the sides of the model; (b) determine the acoustic intensity at different locations inside the chamber; (c) determine the best incident sound wave frequency band to use in testing amplification in the harbour. Acoustic response was measured at seven locations inside the chamber. The results showed that the response decreased with distance from the speaker, as expected. In addition, the sound intensity dropped rapidly and remained almost constant at frequencies greater than 3.5 kHz and a bump in the frequency band 2.0–3.5 kHz was observed. The explanation for this was a resonant vibration between the floor of the model and floor of the laboratory. Elevating the model and acoustically isolating the laboratory floor effectively detuned this vibration.

In the final acoustic tests the harbour model was placed inside the chamber, and the responses at eight locations inside the model harbour were tested under various conditions. The results of these tests are discussed later in this paper.

It should be noted that comparisons between the acoustic model results and the NRCC hydraulic model results using the harbour model without any resonators (Fig. 7) showed general agreement. However, at frequencies lower than 2 kHz (6.6 s period prototype), the amplification was lower than that achieved in the hydraulic model results. Of course the hydraulic model results should not be considered "accurate" in view of scale effects in boundary reflectivity.

Public Opinion Survey

A survey of local opinion regarding wave action and the resonator unit in Kincardine Harbour, to gain a better understanding of the harbour problem, was considered worthwhile. A questionnaire was prepared with a map of the harbour (Fig. 8) and the inlet channel and circulated to area residents who use the harbour. This survey was prepared to establish: (a) the nature of any internal oscillations that might constitute a nuisance; (b) whether wave conditions in the harbour were considered to have improved as a result of the existing resonator; (c) the attitude of boaters and sailors to resonators; (d) the extent to which the existing resonator is a navigational hazard; (e) whether additional resonators are considered acceptable; and (f) whether users were of the opinion that the rubble-mound breakwater inside the harbour provided protection from wave action and whether it is a navigational hazard.

It was expected that the answers to these questions would help establish a suitable utility function for oscillations within the harbour and in the harbour entrance channel.

The survey confirmed that Kincardine Harbour is used almost exclusively for pleasure purposes. It is frequently used for the full duration of the boating

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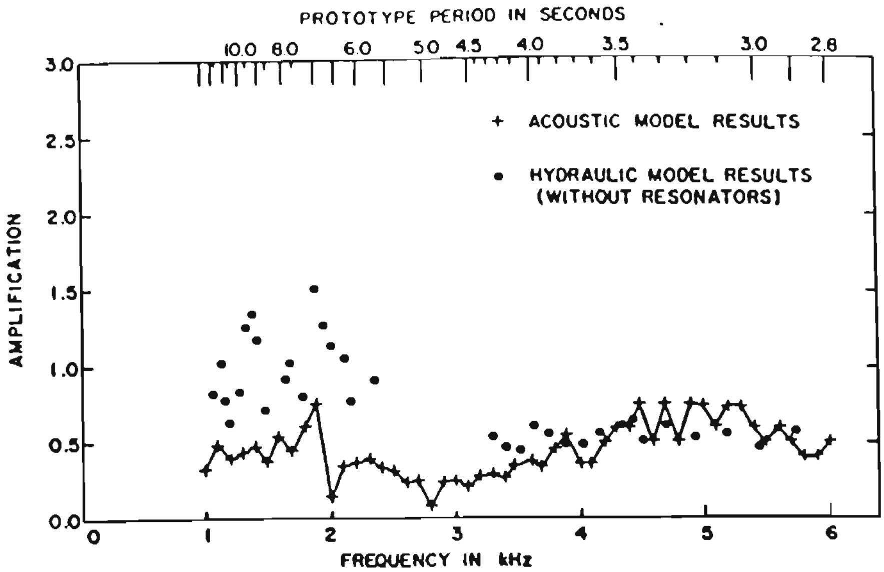

FIG. 7. Acoustic model and hydraulic model results without resonators. Note: 1 ft = 0.3048 m.

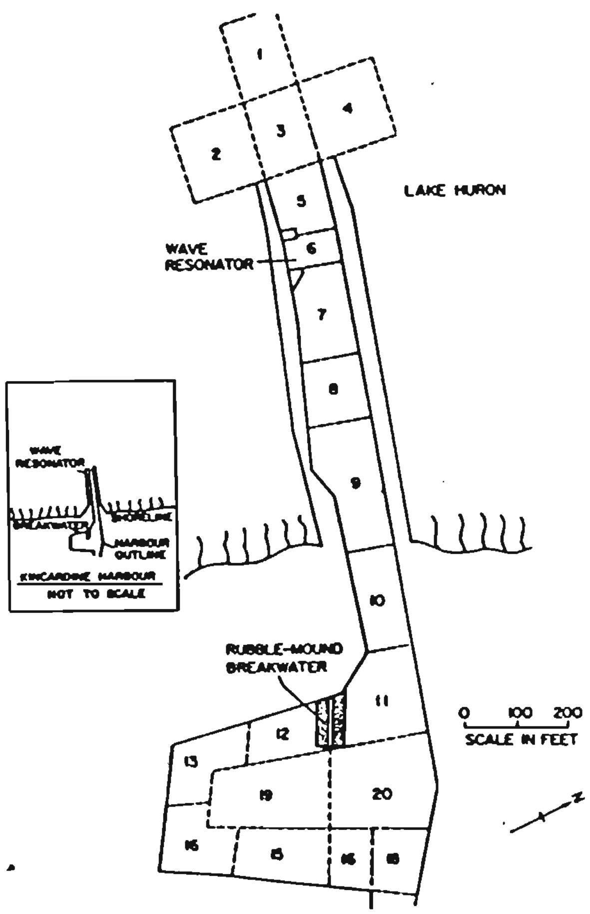

FIG. 8. Wave agitation zones within Kincardine Harbour. Note: 1 ft = 0.3048 m.

season with concentrated use during the months of June to September. The survey did not include boaters who visit the harbour from other areas during the season, but who may be numerous.

The types of craft commonly used are sailboats and motorboats of small power. Areas commonly used for mooring are locations 12–15 and 19 (Fig. 8). The survey revealed that there has been less wave motion in the harbour since the reasonator was installed in 1972 (approximately 60% of the respondees), and the same portion of the respondees indicated that the rubble-mound breakwater also offered protection. Wave action has been observed to be excessive at the channel entrance and near the resonator, i.e., zones 3, 5, and 6, with zones 12–14 and 16 ranked next. All of these zones are also notable for excessive vertical and horizontal vessel movement. Fifty percent of the users considered the resonator structure a hazard.

Interestingly, comments and suggestions concerning the entrance and resonators were often in conflict, for example: control of boats in area of resonator is difficult due to excessive wave and surge (15%); resonators are navigational hazards at night, expecially in bad weather and thus they should be marked and lit at night (50%); another resonator should be installed on the north pier; and another one should be installed east of the existing resonator in the entrance channel.

Utility Function

One of the objectives of the study was to establish a suitable utility function for oscillations inside the harbour and in the harbour entrance. The function is to indicate the general inconvenience caused by water oscillations in the harbour. A simple linear function was formulated on the basis of the importance and degree of usage at different locations inside the harbour, as well as the ratios between the wave amplitudes inside the harbour and the incident wave amplitude:

[6] $D = 1 - (1/N)\sum(C_i A_i/A_0)$

where D is the overall utility coefficient for the harbour; C_i are the coefficients depending on the importance of different locations; A_0 is the incident wave amplitude; A_i are the amplitudes measured at different locations inside the harbour; and N is the total number of zones considered.

The survey revealed that zones 12–16 and 19 (Fig. 7) are the most frequently used for mooring. Zones 5–7, 11, and 20 are also important for navigation. For the survey we selected the C_i as follows: $C_i = 1.5$ for zones 12–16 and 19; and $C_i = 0.5$ for zones 5–7, 11, and 20. The other zones were considered of little interest. An average amplification of 0.25 (amplification is defined as the ratio of the wave amplitude inside the harbour to the incident wave amplitude) was considered acceptable for a sheltered harbour. The utility will then be 0.74. A value of 1.0 indicates perfect shelter. On the other hand, if D is zero, the harbour is not providing any protection—conditions are the same as external incident conditions. A negative value indicates that wave motions are worse inside the harbour than outside. The values of C_i selected reflect the importance of mooring over navigation because of the large amount of unattended time spent by the boats at the moorings, and the anxiety of absent boat owners.

Tests on the Acoustic Harbour Model

The acoustical responses at eight locations (Fig. 9) inside the model harbour were tested under the conditions listed below and those represented in Fig. 10:

(a) Without any resonators.

(b) With 1 resonator.

(c) With 2 resonators.

(d) With 3 resonators.

(e) Same as (d) but with the middle resonator blanked out.

(f) Similar to condition (b) but with a thin layer of cotton batting on the lakeside of the first resonator structure to simulate wave absorption by rockfill, which could be placed at this location to protect the structure against undermining and eventual failure; an angled backface of about 30° was incorporated on the second resonator structure.

(g) Extension of the first resonator structure to the lakeside limit of the south jetty; again an angled backface was incorporated on the second resonator structure.

(h) Same as (f) but with the addition of a third resonator structure placed at a distance of 80 ft (24.4 m) from the second resonator structure; an angled backface was incorporated on the third resonator structure.

(i) Same as (g) but with the addition of a third resonator structure as in (h).

All resonator alternatives were located on the south jetty because of the following: (a) the north jetty is relatively new; (b) it would be costly to incorporate resonators in the north jetty wall; (c) north jetty resonators would seriously encroach on the navigation channel; and (d) the deteriorating state of the south jetty promotes incorporation of resonators in its eventual reconstruction.

The model resonator structures for the tests were made of Plexiglas with a width of 1.8 in. (45.7 mm) (i.e., 30 ft (9.1 m) prototype) and a height of 1.75 in. (44.5 mm), the same as the space between the top and the bottom of the acoustic chamber. The location of the first resonator structure was 6.6 in. (167.6 mm) (i.e., 110 ft (33.5 m) prototype) from the harbour entrance. The second resonator structure was located at 3.6 in. (91.4 mm) (i.e., 60 ft (18.3 m) prototype) from the first. The location of the third resonator structure under conditions (h) and (i) was

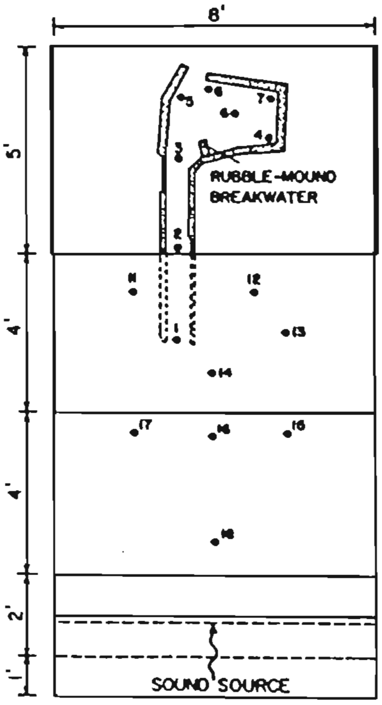

FIG. 9. Locations of wave measurements in Kincardine Harbour model. Note: 1 ft = 0.3048 m.

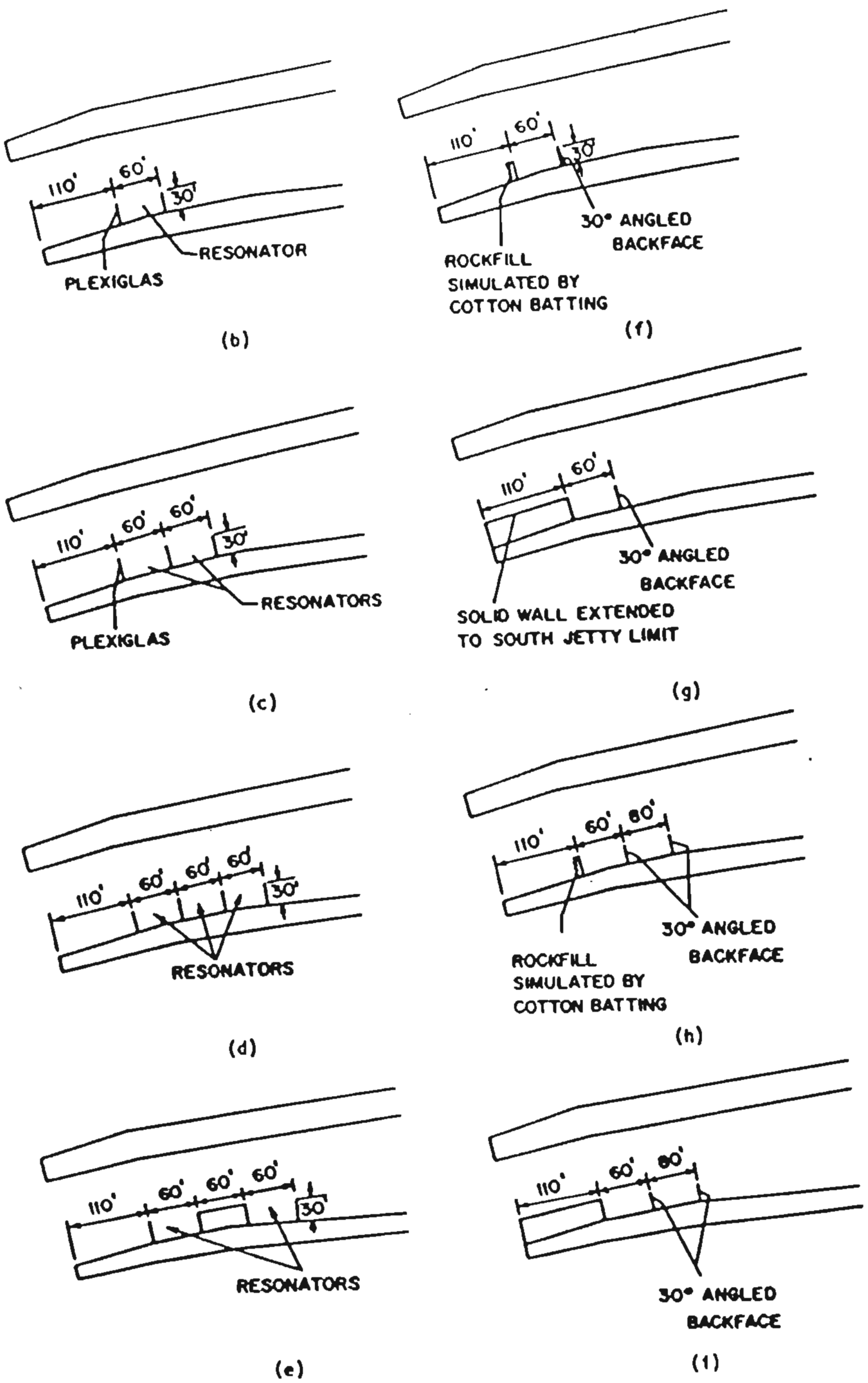

FIG. 10. Wave resonator configurations evaluated in the acoustic model tests. (See text for details of test conditions (b)–(i).) Note: 1 ft = 0.3048 m.

determined by obtaining a minimum response measured at reference point 2 inside the harbour. Tests were carried out by measuring the responses when the distance of the third resonator structure from the second was varied from 30 to 120 ft (9.1 to 36.6 m). Frequencies used in these tests were 2, 3, 4, and 5 kHz. It was determined that for both conditions (h) and (i) that a 4.8 in. (121.9 mm) (80 ft (24.4 m) prototype) separation between the second and third resonator structures provided the lowest response measured at location 2. All tests were done with the model rubble-mound breakwater in place.

The effect of the resonators was determined by comparing frequency plots of amplification for con-

TABLE 2. Percentage reduction in observed wave amplitudes in the acoustic model for various resonator configurations as compared with the condition with no resonators

	Test conditions and resonator configuration							
	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)
Number of resonators	1	2	3	2	1	1	2	2
Resonator spacing	—	Uniform	Uniform	Separated	—	—	Unequal	Unequal
Front wall condition	Unprotected	Unprotected	Unprotected	Unprotected	Simulated stone	Extended	Simulated stone	Extended
Backface condition	Perpendicular	Perpendicular	Perpendicular	Perpendicular	Angled	Angled	Angled	Angled
	Reductions in wave amplifications at various locations (%)							
Harbour location of wave amplitude observations								
1. Channel mouth	20	20	20	20	20	10	20	20
2. Channel	40	40	40	40	40	40	50	47
3. Channel	30	30	30	30	30	30	40	37
4. Mooring area	20	30	30	30	20	20	40	40
5. Harbour basin	30	35	35	30	30	30	40	35
6. Harbour basin	15	15	15	15	15	15	40	35
7. Mooring area	30	30	40	30	30	30	60	50
8. Mooring area	10	10	40	10	10	10	30	25
Average reduction—all locations (%)	24	26	31	26	24	23	40	36
Average reduction in harbour basin (locations 4–8) (%)	21	24	32	23	21	21	42	37

ditions (b)–(e) with that of condition (a) over the band 3–6 kHz for locations 1–8. The difference in amplification was expressed as a percentage of (a) and averaged as follows:

$$\text{response} = \frac{A_a - A_x}{A_a N} \times 100\,\%$$

where N is the total number of samples taken for each location; A_a is the plot of amplification for condition (a) between 3 and 6 kHz; and A_x is the plot of amplification for conditions (b), (c), (d), etc. between 3 and 6 kHz. Table 2 summarizes the results of the acoustic model tests with the eight different resonator configurations.

Conclusions

The study was innovative in many ways, since resonators are not commonly used in harbours, only a few acoustic model studies have been carried out on harbour problems, and the utility function formulated is novel. Hence, a number of conclusions that are of general relevance can be drawn and these are listed below for the benefit of readers.

(1) Using the analogy between sound waves and water waves, an acoustic model can be effectively used to simulate harbour resonance. The reflectivity of the cotton batting absorbers is comparable to that of prototype beaches and equal to that of the hydraulic model (less than 10%), but the reflectivity of wooden walls (about 35%) is probably low and the model results should thus be interpreted qualitatively. It should be noted that hydraulic modellers do not generally define model boundary reflectivities as a function of frequency and so a complete comparison of amplification is not possible.

(2) A comparison between the acoustic model and the hydraulic model results (carried out by the National Research Council of Canada) shows general agreement (commensurate with the accuracy of the hydraulic model) at periods greater than 4.1 s prototype. However, at periods less than 4.1 s prototype the amplification obtained from the acoustic model (amplification defined as the response inside the harbour divided by the response outside the harbour) is about half that obtained from the hydraulic model (see Fig. 7). Full correspondence between both models appears to be difficult across the whole frequency band.

(3) The public opinion survey indicated that vertical and horizontal surging in the inner harbour

after the installation of the resonator was still excessive, causing vessels to strike the dock walls. Further protection against wave action is therefore desirable to improve the harbour as a shelter for small craft.

(4) The then-existing resonator and the in-harbour rubble-mound breakwater did provide protection. However, they present a hazard to navigation at night, especially in bad weather, and as a result such structures should be suitably marked.

(5) Evidently the offshore breakwater could be extended to provide protection from southwesterly winds. According to the opinion survey, this breakwater is a potential hazard in bad weather conditions or at night and should similarly be marked.

(6) A utility function indicating the inconvenience of water motion, which appears to be meaningful, has been suggested for Kincardine Harbour.

(7) Additional resonators improve utility but there will be limited incremental improvement for each resonator provided. In considering the cost of construction and the anticipated navigational difficulties, probably not more than two resonators should be provided. The areas where amplification is reduced following the installation of resonators are concentrated near locations 2–5 and 7 (see Fig. 8).

(8) Reductions in wave amplification for the eight locations inside the harbour under conditions (f) and (g) were similar to those obtained under condition (b) (see Fig. 10).

(9) Where two resonators were used, the best length of the second resonator was approximately 80 ft (23.4 m) with the first one measuring 60 ft (18.3 m) in length, 110 ft (33.5 m) from the harbour entrance. These dimensions are closely related to the wave climate and the harbour entrance geometry.

(10) By using two resonators of unequal lengths (60 ft and 80 ft (18.3 and 23.4 m) in this case), the reductions in wave amplification were higher than that of equal lengths (60 ft × 60 ft (18.3 m × 18.3 m)). For condition (h), the average increase in reduction was approximately 14% over that of condition (c), and for condition (i) this increase was approximately 10%.

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The preliminary acoustic model was built and tested by the National Research Council of Canada, who transported it to McMaster University.

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Power from Waves Using Harbor Resonators

By William James,[1] M. ASCE

Introduction

The provision of an easy shipping approach to a harbor is difficult due to the limited methods available for preventing wave penetration into the harbor. These methods include conventional breakwaters, to dissipate and reflect the waves, and staggered or offset entrance breakwaters, to isolate the harbor from reflected and diffracted waves. Breakwaters are also used to prevent littoral sediments from encroaching upon and entering into the harbor entrance channel (1). This disruption of sediment transport along a coastal zone often results in extensive erosion along the down drift coastline and substantial accretion in the area immediately updrift of the breakwater. Methods for bypassing this material include sediment bypassing plants, pumping dredges, and dragline dredges (3).

Harbor resonators have recently been used in a number of Canadian harbors, both tidal and nontidal (5). It has been demonstrated experimentally (17) that these structures effectively reduce wave energy transmitted into the harbor, yet provide a clear navigation channel.

The performance of rectangular resonators for various configurations, wave periods, entrance widths, and water depths has been previously studied and documented (2,8,12,13,24,25,26). The theory describing their behavior is based upon first-order linear wave theory and the assumption of irrotational motion (15). A summary of the important findings has been published (20).

Battery configurations can be used to cover a broader band of incident wave frequencies. The response then has been shown to be approximately linear in that the effect of the battery is the sum of the effects of the independent

Note.—Discussion open until July 1, 1980. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. This paper is part of the copyrighted Journal of the Waterway, Port, Coastal and Ocean Division, Proceedings of the American Society of Civil Engineers, Vol. 106, No. WW1, February, 1980. Manuscript was submitted for review for possible publication on March 29, 1979.

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resonators (13). Resonator batteries operate most efficiently when they are separate and not contiguous (13,16). Design curves that can be used to calculate the anticipated spectral response for the battery, but only for the first resonant mode, are presented elsewhere (20). The spectral response can also be determined very effectively by using an acoustic model (13). Acoustic models used in this type of work showed no scale effects and provide quick and accurate response curves at little cost (18).

Rectangular resonators, used either individually or in battery form should be placed as near as possible to the ocean end of the entrance channel. This reduces the extent of the partial clapotis, caused by the superposition of the reflected waves on the incident waves, to a relatively short section of the entrance channel. The partial clapotis at the mouth helps to prevent influx of littoral

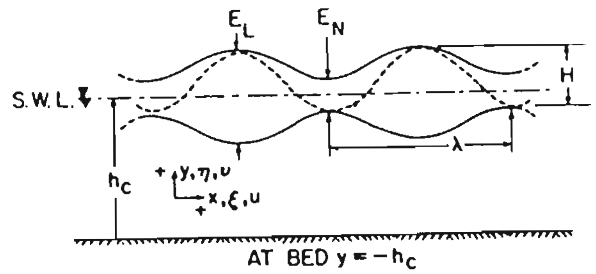

FIG. 1.—Wave Notation

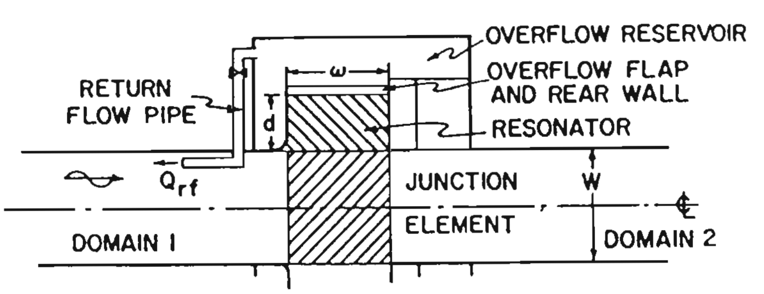

FIG. 2.—Plan of Resonator Layout (Showing One Resonator)

sediments by reducing mass transport (19). The amplified orbital bed motion causes bed sediments to saltate; this suspension is transported by any superimposed second-order mass transport or drift currents present. In a series of tests, Lates (21) observed that by constructing a minimum of three resonators in a battery configuration, where main channel widths were limited to approx 0.7λ, influx of littoral sediments was greatly reduced by the action of the reflected waves or partial clapotis in the channel mouth. Agitation in the harbor entrance discourages deposition of material and is evidently an effective means of sediment bypassing.

The configuration tested in this study was a single pair of geometrically opposed rectangular resonators. The partial clapotis to provide the agitation required to keep the sediment in suspension is maximized by tuning the resonators to reflect as much incident wave energy as possible, while a return flow derived from overflow in the resonator basins prohibits the movement of this sediment into the harbor entrance. Cornick (4) gives typical critical velocities to move various classifications of sediment but the return flow velocities in this study cannot be compared directly with these critical velocities because of the effect of the amplified orbital velocities obtained near the bed under the partial clapotis.

This paper is intended to provide data for the design of on-channel resonators to: (1) Eliminate or reduce the transmission of a selected band of harmful wave frequencies; or (2) maximize orbital velocities near the bed so as to prevent the influx and deposition of littoral sediments in harbor entrances; or (3) maximize return from power extracted from the wave action in the resonators, or all three.

Theoretical Description

A definition sketch, together with the coordinate system used, is presented in Figs. 1, 2, and 3. The notation is listed and defined at the end of the paper.

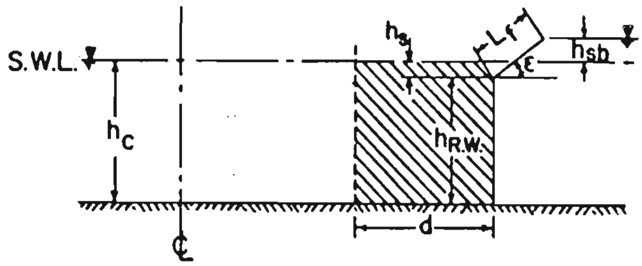

FIG. 3.—Resonator Notation

Relevant Results from Potential Theory (9).—We use the following results of first-order theory. The average potential energy per unit surface area is

$$\bar{E}_p = \frac{\gamma H^2}{16} \qquad (1)$$

The average kinetic energy per unit of surface area is given by:

$$\bar{E}_k = \frac{\gamma H^2}{16} \qquad (2)$$

Wave power is the time rate at which work is done by a wave. Wave energy propagates at the group velocity

$$C_G = \frac{C}{2}\left(1 + \frac{2kh}{\sinh 2kh}\right) \qquad (3)$$

in which C = phase velocity of the individual waves. The energy propagated is the total wave energy, which is the sum of both kinetic and potential wave energies. Thus total average wave power over one wave period per foot of width may be written as

$$\bar{P}_{to} = \bar{E}_t C_G = \frac{\lambda\gamma H^2}{8} C_G \qquad (4)$$

Return Flow Mechanism.—The return flow mechanism depicted in Fig. 4 is a dynamic system; for simplification, some averaging must be incorporated in the equations describing the system. It is necessary to use average values for both stilling basin height and discharge, due to the fluctuations in water surface elevation above the discharge end of the return duct in the main channel. This head fluctuation across the return flow pipe greatly affects discharge uniformity due to the small relative magnitude of the stilling basin head developed. The orbital motions at the discharge end also affects the uniformity, since the discharge end will not be deeper than twice the incident wavelength. The effect of both inertial and frictional drag on the fluid in the pipe tends to help dampen out

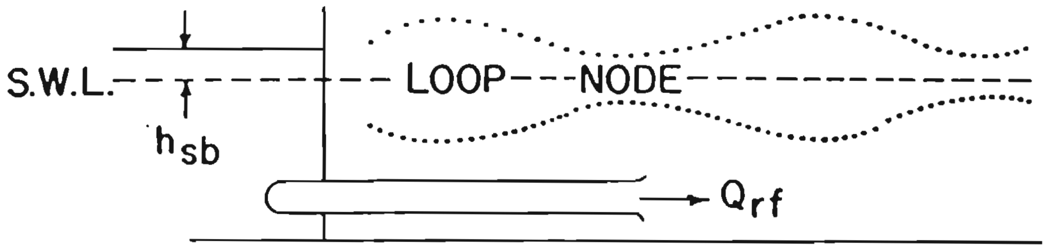

FIG. 4.—Return Flow Notation

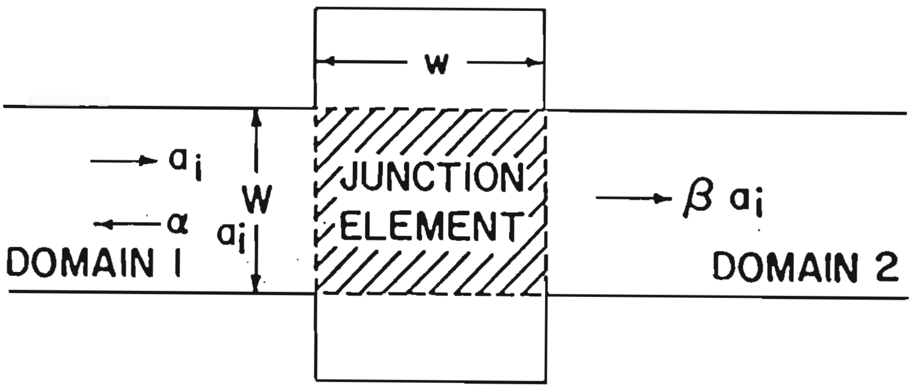

FIG. 5.—Wave Notation, Plan of Junction Element

the fluctuations somewhat. The fluctuation of the stilling basin depth can be minimized by providing a large holding basin or reservoir from which the return flow discharge may be drawn. An expression is developed in terms of the average head and discharge.

Continuity Equation.—A continuity equation for the junction element depicted in Fig. 5 may be formulated as

$$I_e - O_e - 2Q_{rf} = A_e\left(\frac{\partial\bar{\eta}}{\partial t}\right) \quad \text{. . . . . (5)}$$

in which Q_{rf} = return flow from each resonator; A_e = surface area of junction element ($W \times w$); I_e = inflow into junction element from main channel; and O_e = flow out of junction element. Motion in the junction element was observed to be very complicated. Thus the partial derivative of the average vertical surface displacement with respect to time cannot be evaluated at this time, preventing further development along these lines.

Energy Equation.—Power in flowing water, as in pipes, is the product of the number of pounds of fluid flowing per second and the head lost or gained across the control section:

$$P = \gamma Q h \quad (6)$$

Eqs. 4 and 6 provide expressions for all the energy involved in the junction element except that of losses due to eddy formation and turbulence. Thus

$$\bar{P}_{to} = \frac{W\lambda\gamma H^2}{8} C_G \quad (7)$$

represents the total wave power in a channel of width W feet. This applies to all waves including incident, reflected, and transmitted; we distinguish between them by subscripting the wave height term:

$$\bar{P}_{rf} = 2\gamma \bar{Q}_{rf} \bar{h}_{sb} \quad (8)$$

represents the total average return flow power from both resonators, neglecting the frictional losses and inertial effects in the pipe.

By performing a power balance on the junction element, it is possible to develop an expression for the average return flow that might be expected from the operation of a return flow resonator:

$$\bar{P}_i = \bar{P}_r + \bar{P}_t + \bar{P}_{rf} + \text{power losses} \quad (9)$$

Thus, neglecting losses

$$\frac{W\gamma H_i^2}{8} C_G = \frac{W\gamma H_r^2}{8} C_G + \frac{W\gamma H_t^2}{8} C_G + 2\gamma \bar{Q}_{rf} \bar{h}_{sb} \quad (10)$$

Substituting:

$$H_r^2 = \alpha^2 H_i^2 \quad \text{and} \quad H_t^2 = \beta^2 H_i^2 \quad (11)$$

we get

$$\bar{Q}_{rf} = [1 - (\alpha^2 + \beta^2)] \frac{W H_i^2}{16 h_{sb}} C_G \quad (12)$$

For perfect reflection or transmission, the return flow will be zero. The return flow would only be dependent on W for $W \leq \lambda$. For W larger than λ, energy will be transmitted into the harbor and not greatly affected by the resonators. This equation of course does not relate Q_{rf} to w or d directly, but α and β values are highly dependent on both w and d.

Experimental Apparatus

It was necessary to install certain essential wave devices in the experimental wave flume. For each of the two absorbers, two filter banks and resonator test section, distances at least equal to the maximum test wavelength were allowed. For each of the two measuring domains, distances equal to three maximum wavelengths were allowed.

The wave flume was 44 ft (13.4 m) long, 2 ft (0.6 m) wide and 1 ft (0.3 m) deep, with a central resonator basin 4 ft (1.2 m) long and 6 ft (1.8 m) wide.

At the extreme ends of the tank, wave absorbers 4 ft (1.2 m) in length, as described by Lean (22), fabricated from 1/4 in. (6 mm) steel plate, with 1/2-in. (13-mm) diam holes drilled evenly on a 2-in. (51-mm) square grid, were rigidly fixed to the channel end walls. The downstream absorber was also fitted with sheets of 1/2-in. (13-mm) thick fiberglass matting to further reduce wave reflection as described by Garrett (6). This will produce a better decoupling of the system. Permeable absorbers such as these are particularly suited to low waves that are harder to absorb than steep waves (23).

At the extreme upstream end immediately adjacent to the wave absorber, a paddle-type wave generator was installed consisting of a plate linked by a drive arm to an electrically driven eccentric. Both eccentricity and linkage length were variable so that paddle stroke and position could be adjusted. The waves generated were approx 0.5 in. (13 mm) in height. The linkage connections remained constant throughout the test period, with the result that wave amplitudes varied and were frequency dependent (7). However the maximum wave steepness of 0.005 was such that linear theory was still appropriate.

Immediately adjacent to the wave generator and wave absorbers, a system of low pass filters was installed. These filters improved the uniformity of the monoperiodic wave train by eliminating harmonics of higher frequency. The filters were constructed of aluminum tubing to form a framework, over which continuous fiberglass screening was stretched so as to form a series of vertical panels, alined in the orbital planes, spaced at 1/2-in. (13-mm) intervals across the width and extending the full depth of the tank. Each of these filters was approx 2 ft (0.6 m) long by 2 ft (0.6 m) wide to enable rigid installation in the flume. At each end, two of these filters were used in series to ensure that the length exceeded the maximum wavelength to be tested. The performance of such filters is amplitude independent but frequency dependent (17).

In the upstream domain between the resonators and the filters, a wave guide was installed to prevent transverse oscillations. Such transverse resonance occurs at frequencies whose wavelengths are near or at multiples of the flume width. The wave guides consisted of aluminum screening stretched parallel to the tank walls positioned at the nodal points for the fundamental and first harmonic transverse resonant frequencies.

On the central 25 ft (7.63 m) of the tank a system of horizontal elevated aluminum rails was constructed to accommodate the instrumentation trolly. The trolly carried a micrometer with 2 in. (51 mm) travel, set in a steel support frame, for accurate measurement of wave envelopes. The probe was insulated from the carriage frame and connected directly to a cathode ray oscilloscope. Determination of probe entry and exit points was thus more precise than visual observations would be (11).

At the channel midpoint, a resonator and junction element with a total width of 6 ft (1.8 m), was provided. This section covered the central 4 ft (1.2 m) of the tank. Within this area on each side of the tank, two false walls were constructed to provide a lateral base for the resonators. The downstream wall was a U section installed so that one leg of the section was in line with the main channel wall while the open end faced the external "ocean" wall of the expansion chamber. The upstream wall was a curvilinear L section with a circular re-entrant curve instead of a sharp 90° corner. This curve effectively reduced turbulent eddies that resulted from the high velocities produced when the incident wave fronts expand at this point. These two sections formed the side walls of the resonators. The rear walls were constructed from 1/16 in. (1.6 mm) steel plate. The rear wall had to be hinged to provide an adjustable overflow flap. The vertical section of the rear wall was 6-1/2 in. (165 mm) high and the movable flap section was 2-1/2 in. (65 mm) wide so that when the flap section was vertical the total height of the rear wall was approx 9 in. (229 mm). The water depth during the tests was held constant at 7-1/2 in. (191 mm). The hinge comprised an overlapping rubber strip bolted to each section of the rear wall. This provided a water-tight hinge that was considered essential. This rear wall was simply clamped in place for testing.

A number of U shaped walls were used to systematically reduce the resonator width simply by adding them one at a time to the existing wall already secured in the tank. Each time an additional wall was added the rear wall was accordingly cut down in length. These false walls in the expansion section provided a storage area for the overflow from the resonators. Large quantities of fiberglass screening were packed in the stilling chamber to prevent oscillations. A Plexiglass stilling chamber was secured to the rear wall of the overflow basin. A 2-in. travel (51-mm) micrometer with a needle proble was used to monitor water depth fluctuations in the overflow basin.

A 3/4 in. (19-mm) diam pipe was attached to the rear corner of each upstream wall and passed externally back into the main channel entering in an upstream direction near the bed. A control valve was placed in each of these return flow pipes to interrupt the flow returning to the main channel. A vertical overflow was provided free to rotate in a vertical circle. At the start of each test the overflow invert was set to the same level as the main channel water level. Once the test had started, resonator overflow discharge from this pipe, with the return flow valve shut, was measured using a stopwatch and a 500-mL graduated cylinder. The overflow was caught in a large drum and pumped back into the main tank behind the wave generator by means of a small submersible bilge pump. This maintained a constant water depth throughout the test period.

Experimental Procedure

Before experiments on the resonator arrangement could begin, it was necessary to evaluate the whole rig. The first objective was to determine the best operating depth. The criteria for "best depth" was the correlation between first-order theoretical wavelengths and generated measured or observed wavelengths. A depth of 7-1/2 in. (191 mm) was found to provide the best relationship within the limits of the measuring instruments, and over the range of frequencies and amplitudes of interest.

Reflectivity and Transmissivity Calibration.—The second consideration was an evaluation of the reflectivity and transmissivity transfer functions, with the resonators isolated from the main channel. That is, the behavior of the flume decoupling system alone was evaluated as a function of incident wave frequency. Envelope loop and node heights were measured in both the upstream (Domain 1) and downstream (Domain 2) regions and resulting values of transmissivity and reflectivity were plotted against wave frequency (14). Results indicated that both transfer functions are approximately linear over the frequency band

$1.05 < 1/T < 1.82$, which was satisfactory for testing the resonator configuration.

Resonator Geometry Limits.—This frequency band also dictated the range of resonator dimensions to be tested (15), since it was felt that maximum overflow conditions would be at or near resonant geometries. For wave periods ranging from 0.60 sec to approx 1.5 sec, it was evident that resonator tests widths w ranging from 21 in. (533 mm)–12 in. (305 mm) and resonator test depths d ranging from 9 in. (229 mm)–4 in. (102 mm) would cover the required range of resonant geometries. In fact such resonator shapes would vary between the expansion chamber resonant mode and the branch canal resonant mode (10) and should cover the best mode of operation for maximizing return flow energies and minimizing transmitted wave energies.

Test Procedure for Variation of Geometry.—In order to expedite the large number of tests it was decided that the geometry would be held constant while the wave periods were varied over the full range. The resonator depth d was varied in 1 in. (25 mm) steps from 9 in. (229 mm)–4 in. (102 mm). Once all resonator depths d had been tested, each for the full frequency range, the tank was drained and resonator width w was then decreased by welding into place the next u-shaped wall section. The rear wall was then simply cut to the new width and refitted for the next test series of depths and periods. For each geometry and test period the bollowing were measured: (1) Upstream maximum and minimum envelope heights; (2) downstream wave height; (3) stilling basin depth increase; and (4) overflow discharge.

The loop and node method used is based on the assumption that the incident wave form is sinusoidal. Real laboratory waves are usually nonlinear. This difference introduces some error in the analysis of the partial clapotis envelope (14) and gives a larger value than is indicated by the first-order theory. The downstream wave height was measured at two stations separated by one-quarter of the generated wavelength (11). The transmitted wave amplitude as given by the average of these two readings would not be in error, because of the small reflections from the downstream absorber and filter.

Return Flow Measurement.—The discharge was naturally related to the stilling basin depth increase and thus a steady-state condition had to be achieved before measurements could be taken. This measurement was the last to be taken in all test series. The position of the overflow flap was critical in the discharges obtained. It was observed that if the overflow flap was low, excessive backwash over the flap would take place. If the flap was too steep the stilling basin water level would be lower than the crest of the overflow flap. The most effective position was set manually by observing the system in operation. The "best" position was found such that a small quantity of backwash was allowed. This produced maximum discharges and stilling basin depths. The overflow discharge was then measured at least three times to ensure accuracy of measurement and uniformity of flow.

Control of Experiments.—The incident wave period was measured over a 2-min interval, to achieve the required accuracy, both prior to and immediately following the test procedure. This was particularly necessary when obtaining the peak values of θ, the ratio of the return flow power to the incident wave power. All data measured were reduced and plotted immediately so that the resulting curves were well-defined at all points. Any anomalies encountered were immediately reexamined and either corrected or noted.

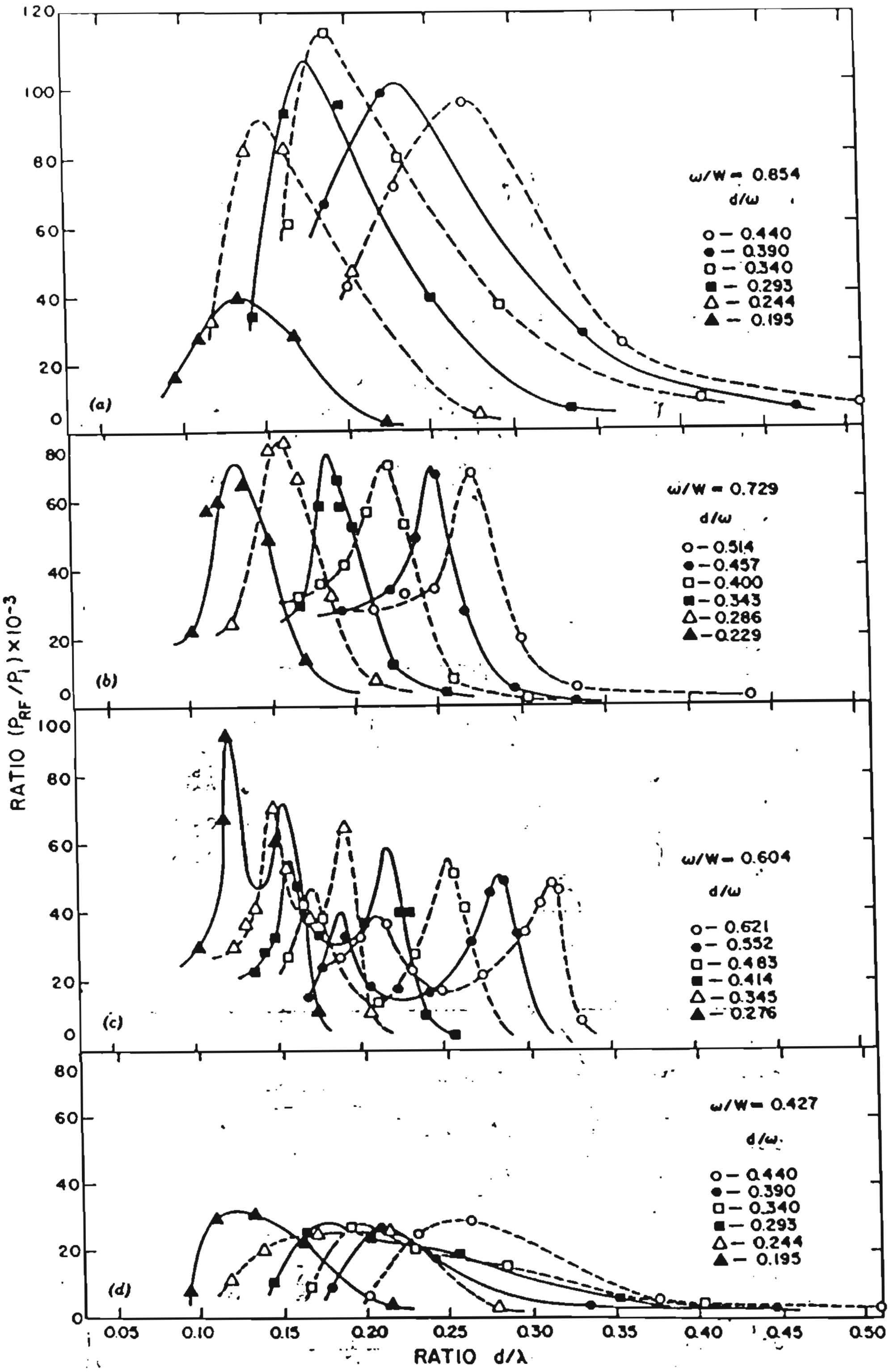

FIG. 6.—Power Extraction Coefficient: (a) w/W = 0.854; (b) w/W = 0.729; (c) w/W = 0.604; (d) w/W = 0.427

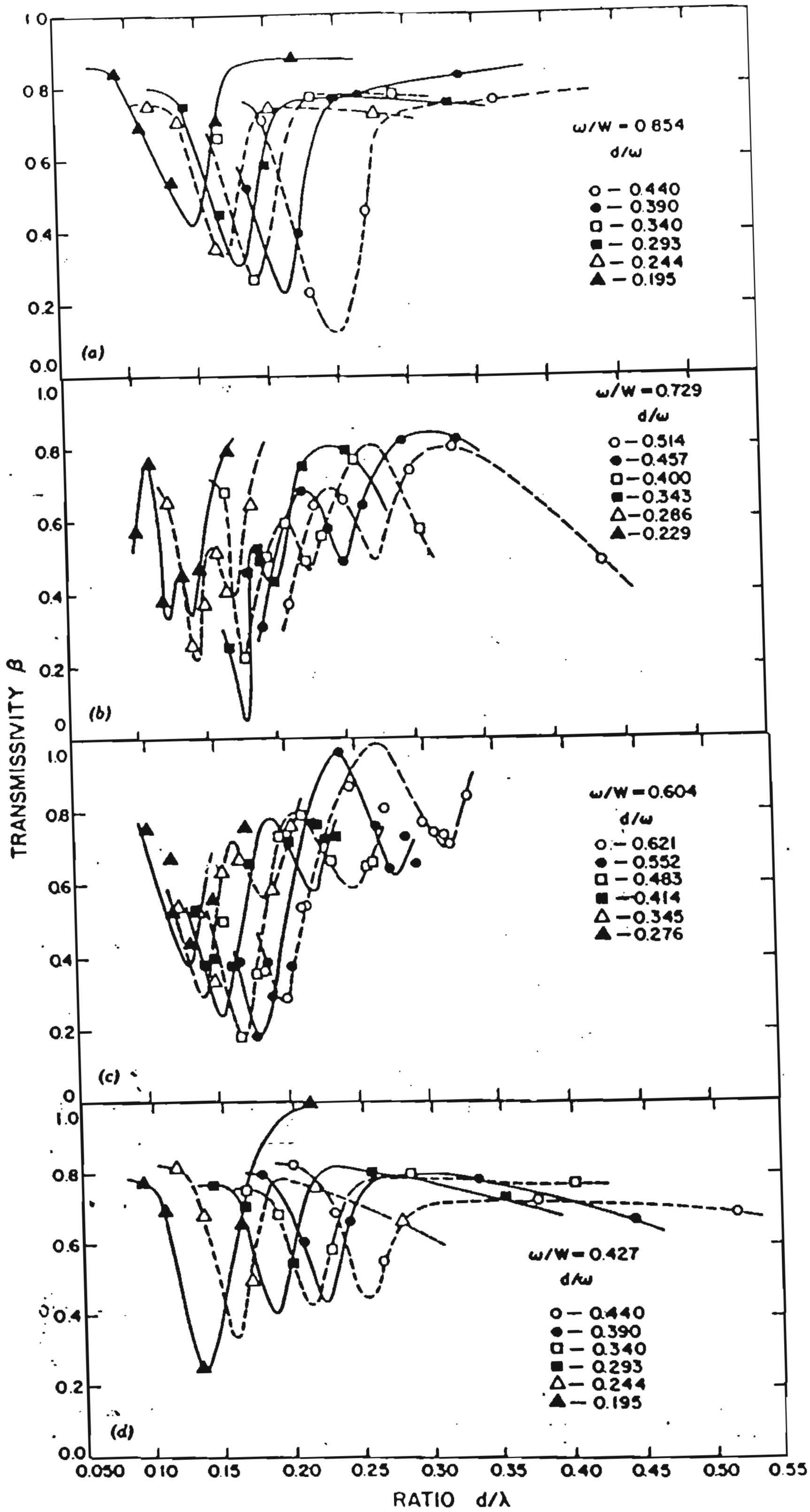

FIG. 7.—Observed Transmissivity: (*a*) $w/W = 0.854$; (*b*) $w/W = 0.729$; (*c*) $w/W = 0.604$; (*d*) $w/W = 0.427$

Experimental Results

The relationship used to express the effectiveness of the return flow resonator is termed the power extraction coefficient θ. It is the ratio of return flow power

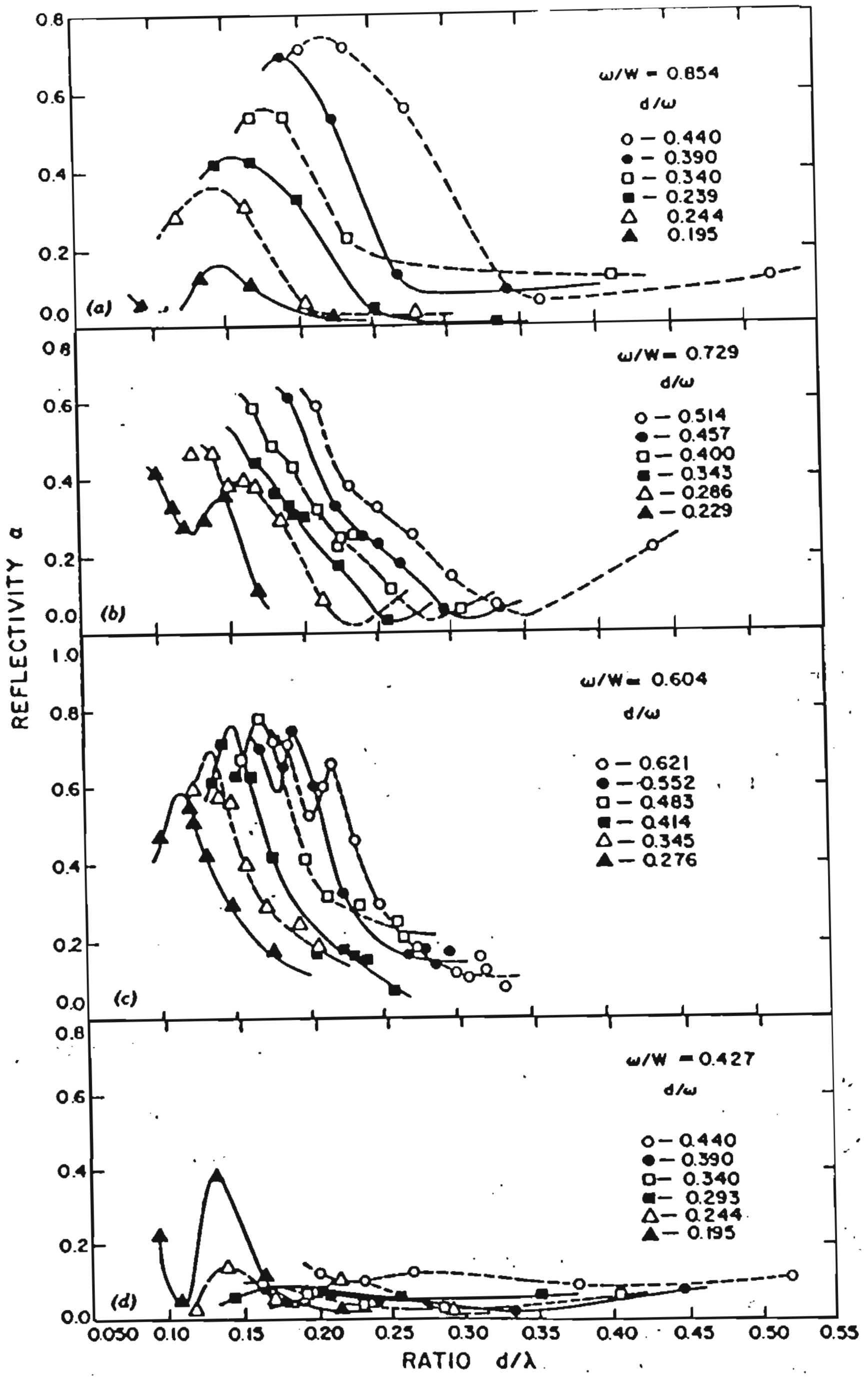

FIG. 8.—Observed Effectivity: (*a*) $w/W = 0.854$; (*b*) $w/W = 0.729$; (*c*) $w/W = 0.604$; (*d*) $w/W = 0.427$

to available incident wave power, and is plotted against the tuning parameter d/λ for different values of d/w, producing families of response curves for each w/W ratio tested (see Fig. 6).

Transmissivity.—Transmissivity is the ratio of the amplitude of the wave transmitted into the harbor past the resonators to the amplitude of the incident wave and is similarly plotted in Fig. 7. The coefficient of reflection is the ratio of reflected wave amplitude to incident wave amplitude, and is plotted in Fig. 8.

Because of resonator draw-off it may be expected that small vessels may be drawn into the resonator. Surface agitation was examined by observing the movements of a float approx 1 in. (25 mm) by 1/2 in. (13 mm) by 1/8 in. (3 mm). The results were qualitative and are summarized in the conclusions.

Conclusions

The following conclusions can be made:

1. General observations may be made regarding the power extraction coefficient curves presented in Fig. 6. The most significant result, easily observed from the curves, is that both efficiency and versatility of return flow resonators is very much dependent on the resonator width w. Efficiency decreases with width, and becomes very poor (approx 1%) for w/W ratios below 0.5.
2. The bandwidth for power extraction at a given efficiency is also dependent on resonator width. The narrower the resonator, the finer the tuning for given power extraction, as shown by the narrow response curves in Fig. 6(c).
3. Peak power extraction tends to increase for decreasing d/w. In other words, the best resonator configuration for power extraction occurs for small values of d. Thus it is evident that the most effective resonator geometry for return flow is the channel expansion configuration. Fortunately this geometry is also the most economical, due to the reduced lengths of breakwater required. Channel expansion resonators are probably more easily adapted to existing harbor entrances.
4. Between 2% and 5% of incident wave power might be used for return flow. This probably represents a significant quantity of virtually inexhaustible energy.
5. When Eq. 12 was checked against experimental discharges it yielded flows that were generally an order of magnitude too large. This might be explained by the absence of any energy loss and inertial terms in the derivation. This suggests significant energy dissipation in the operation of the resonator or overflow ramp. No attempt was made to account for this energy loss. More work is required to identify the complete energy loss terms.
6. Parameters for peak return flow in some cases did not coincide with other design curves (20), the values for d/λ obtained in this study being consistently larger. The disagreement increases for increasing resonator depth d and also increased for increasing main channel width W. Insufficient data were available for preparation of a similar design curve for geometry for peak performance of return flow resonators.
7. Transmissivity and reflectivity peaks were found to coincide with the return flow peaks. Thus transmissivity minima, reflectivity maxima and return flow power maxima all occur at the same geometries. That is to say, the resonator is working most effectively with regard to reduction of transmitted waves at the same time as maximum efficiency for power abstraction is achieved.

8. A general trend in the transmissivity and reflectivity curves suggests that minimum wave transmittance, accompanied by maximum wave reflection, occurs for smaller values of resonant d/λ. This again verifies that the channel expansion configuration is more efficient.

9. During the experiments, the overflow flaps in the resonators were set manually to obtain the maximum discharge and maximum overflow reservoir height. The method was slow and subject to error. Thus the discharges recorded may not have been the absolute maximum possible. The most efficient flap angle was generally observed to occur when a small quantity of water spilled back over the flap into the resonator.

10. Amplitudes in the resonator were, of course, affected by the flap angle. With the flap in a vertical position, maximum amplitudes were obtained; as the flap was lowered increasing the overflow, amplitudes decreased producing a smaller overflow than expected. The flap had to be lowered further until a steady-state condition was achieved and at which point some backwash generally occurred. This coincided with maximum levels in the stilling chanber behind the reservoir.

11. Stilling basin depth increases above still water level varied in magnitude between the incident wave amplitude and the incident wave height for the geometries tested.

12. Navigability, especially for small craft, in the vicinity of the resonator will be a concern to designers. Small craft will experience amplified movement in the junction element, and could be drawn into the resonator chamber if navigating close to the resonator month.

13. The highest orbital velocities, accelerations, and displacements occur immediately adjacent to the resonator mouth and the curved corner. The bigger the radius of curvature the slighter will be the vector accelerations. The velocities and displacements appear to be inversely related to the width of the resonator.

Acknowledgments

The writer would like to acknowledge the Department of Public Works of Canada for assistance and encouragement, and the National Research Council, Canada, for graduate student support.

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Appendix II.—Notation

The following symbols are used in this paper:

$A_e = W \times w$ = area of junction element;
a = wave amplitude;

C = velocity of wave propagation (phase velocity);
C_G = wave group velocity;
d = resonator length, normal to x;
E_k = average kinetic energy;
E_p = average potential energy;
E_t = transmitted wave energy;
E_{to} = total wave energy;
g = gravitational constant;
H = wave height;
H_i = incident wave height;
H_r = reflected wave height;
H_t = transmitted wave height;
h = water depth or head;
h_c = main channel water depth;
h_{ob} = mean depth increase in overflow reservoir;
h_{Rw} = depth at resonator rear wall;
I_e = flow into junction element;
$k = 2\pi/\lambda$ = wave number;
L_f = length of overflow flap;
O_e = flow out of junction element;
p = wave power;
P_i = average incident wave power;
P_r = average reflected wave power;
P_{rf} = average return flow power;
P_t = average transmitted wave power;
P_{to} = total wave power;
Q = discharge;
Q_{rf} = single resonator return flow discharge;
SWL = still water level;
T = wave period;
t = time;
W = main channel width;
w = resonator width, parallel to x;
x = horizontal displacement, in direction of wave propagation;
α = coefficient of reflectivity;
β = coefficient of transmissivity;
$\gamma = \rho g$ = specific weight of water;
ϵ = overflow ramp angle to horizontal;
η = horizontal displacement from mean position;
θ = power extraction coefficient; and
λ = local wave length.

Subscripts

e = junction element;
i = incident;
r = reflected;
rf = return flow;
t = transmitted; and
to = total.

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Proceedings of the American Society of Civil Engineers

TWO INNOVATIONS FOR IMPROVING HARBOR RESONATORS

By William James,[1] A. M. ASCE

INTRODUCTION

The general results of a study on resonators were published in 1968 (2). A method based on these results to predict the response of a single resonator to ocean wave spectra is also being published (5). The experiments reported herein involve: (1) Variable distances between resonators in a battery configuration; and (2) nonuniform depths in the resonator-harbor entrance area.

Very little description of the experimental procedure is presented herein. The wave channel, filters and wave absorber have been described previously (2). The wave measurement technique (3) and the general considerations relating the wave lengths tested to wave channel dimensions and wave measurement accuracy (4) have also been briefly set out. Readers are referred to the earlier publications for further details of the experimental procedure.

Corrections for nonlinear partial clapotis (4) were made, and, for a complete description of the errors that arise using the loop-and-node method to calculate reflectivity, readers are referred to the work of Goda and Abe (1).

The experiments and results for each of the two aspects investigated herein for improving resonator behavior are briefly described, and then general conclusions are drawn at the end of the paper.

BATTERY OF RESONATORS

Experimental Arrangement.—The geometries tested are detailed in Fig. 1. Rayleigh worked on acoustic resonators (6), and he recommended that the ratio of the fundamental periods of the individual resonators in a battery configuration should be $1{:}2^{0.5}$. The resonators were constructed from 3/4-in. thick

Note.—Discussion open until July 1, 1971. To extend the closing date one month, a written request must be filed with the Executive Director, ASCE. This paper is part of the copyrighted Journal of the Waterways, Harbors and Coastal Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 97, No. WW1, February, 1971. Manuscript was submitted for review for possible publication on May 14, 1970.

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Perspex, firmly attached to a Perspex outer resonator housing, as shown i Fig. 1. The second geometry depicted in that figure coincides with that ana lyzed by various other investigators. Reflectivity (ratio of amplitudes of re flected wave train to incident wave train) was measured at a point approximatel one wave length upstream from the battery configuration in the ocean domain and transmissivity (ratio of transmitted to incident wave amplitudes) wa

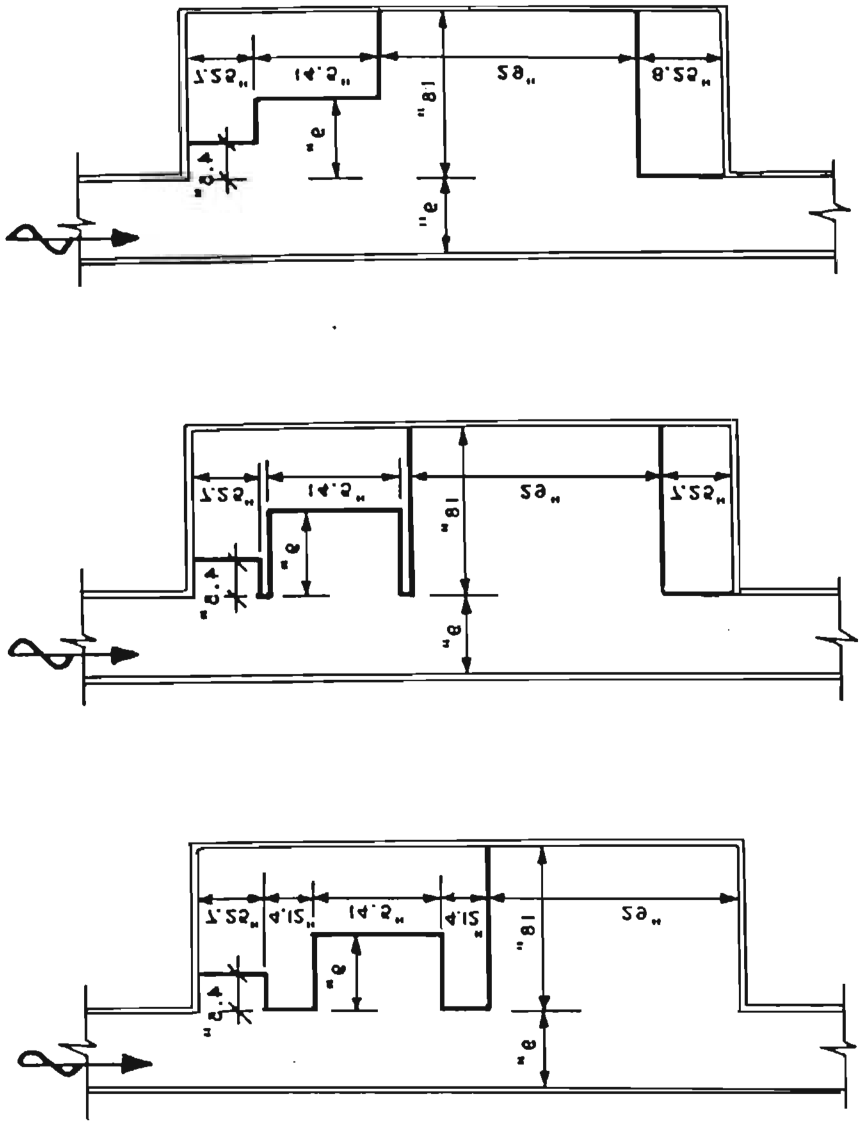

FIG. 1.—BATTERY GEOMETRIES TESTED

measured approximately one wave length downstream from the configuratio in the harbor domain. In these tests the still water depth was held constant a 8.16 in., while the wavelength was changed by altering the wave generato frequency. The amplitudes of the incident wave train varied between 0.21 ir and 0.56 in. while the wavelength varied between 12 in. and 63 in.

Similar measurements were carried out with the resonators blocked off i order to assess channel attenuation and channel-system reflectivity (see Fig

2). In the results plotted herein, the transmissivity is shown by a dashed line and reflectivity by a dotted line.

Results.—Results of the tests on the battery configurations shown in Fig. 1 are presented in Figs. 3, 4 and 5, respectively. For abscissa, the parameter $9/\lambda$ where λ = wavelength, has been chosen because the length of the middle resonator was 9 in. The response is complicated and indicates that careful experiments will be required to define adequately the response of a battery of resonators in any given harbor arrangement. Note that no corrections for attenuation have been applied to these results. Measured attenuation for the wave lengths and steepnesses tested is plotted in Fig. 2. System reflectivity is also plotted in that figure.

Analysis.—Considering the range $0.2\lambda < d < 0.5\lambda$, where d is the length of the resonator, beyond which limits high reflectivity and high attenuation respectively reduce accuracy, each configuration evidently acts as a 4-degree-of-freedom oscillator. The characteristic frequencies are defined by d = 0.46λ, 0.35λ, 0.29λ, and 0.24λ, except in the case of Fig. 5 where the last characteristic value (0.24λ) is replaced by $d = 0.18\lambda$.

The efficacy of each configuration compared as follows: Defining, the Q-value as the reciprocal of the range of abscissa for which the transmissivity is less than 0.5:

$$\left.\begin{array}{l}\text{For Fig. 3 } \sum \dfrac{1}{Q} = 0.27 \\[2ex] \text{For Fig. 4 } \sum \dfrac{1}{Q} = 0.23 \\[2ex] \text{For Fig. 5 } \sum \dfrac{1}{Q} = 0.19\end{array}\right\} \qquad (1)$$

Defining the Q-value as the reciprocal of the band-width for which the transmissivity is less than 0.2:

$$\left.\begin{array}{l}\text{For Fig. 3 } \sum \dfrac{1}{Q} = 0.22 \\[2ex] \text{For Fig. 4 } \sum \dfrac{1}{Q} = 0.15 \\[2ex] \text{For Fig. 5 } \sum \dfrac{1}{Q} = 0.08\end{array}\right\} \qquad (2)$$

From the point of view of reduced wave transmission, the geometry of Fig. 3 is clearly superior to that of Fig. 4, which in turn is better than that of Fig. 5.

No attempt was made to determine the resonance modes; substantial interaction was evident, and, under limited conditions, a transverse oscillation propagated into the harbor domain.

Prediction of Battery Performance.—The results evidently accord with the resonant geometry for a single resonator reproduced from the earlier publication (2) in Fig. 6. Generally, in a design problem the harbor entrance geometry will be fixed within certain limits. For given geometry the frequencies of the resonance peaks in the response of the battery can be estimated as follows:

Plot the equation for each resonator shape (in this case $w = 1.61d$ where

w is the width of the resonator) in Fig. 6. The ratio of the entrance chann width to the resonator lengths are respectively $W = 4.0d, 2.0d, 1.0d$, where is the entrance channel width. Fig. 6 is then scanned for locations such th intercepts of ordinates and interpolated plotted curves satisfy these condition For the experiments this occurs at $d = 0.18\lambda$, 0.24λ and approximately 0.4 denoted A, B and C in Fig. 6. This corresponds to abscissae of 0.35, 0.24, ar one other value, approximately 0.20, in Figs. 3, 4 and 5.

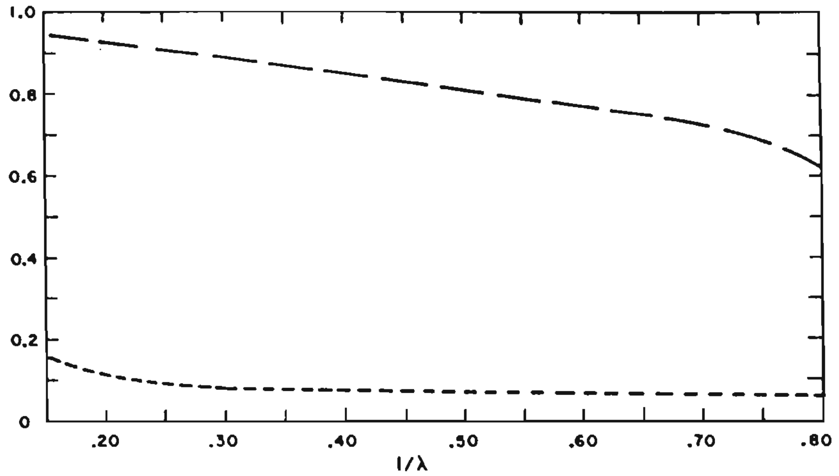

FIG. 2.—SYSTEM ATTENUATION AND REFLECTIVITY

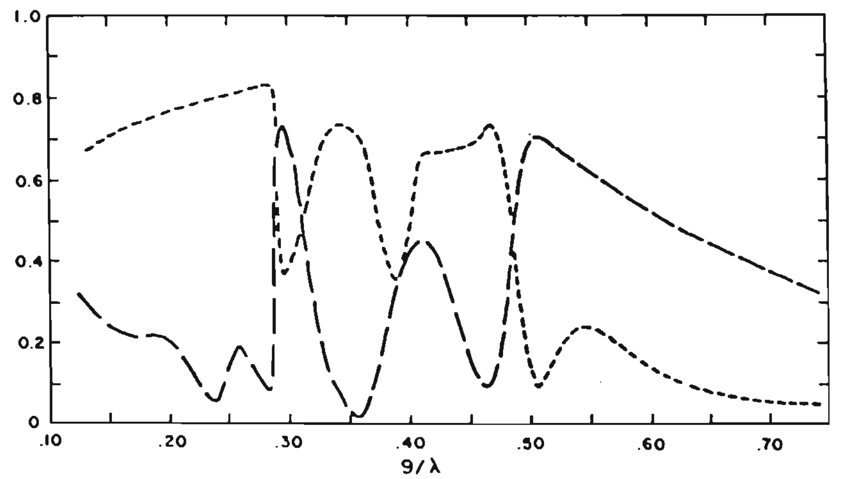

FIG. 3.—REFLECTIVITY AND TRANSMISSIVITY FOR FIRST BATTERY

Figs. 3 and 4 both include transmissivity minima at abscissae of 0.36 ar 0.24. However, another minimum occurs at 0.465, for which $W = 0.93\lambda$. Res onance cannot occur in the mode defined by Fig. 6 for the values of w(0.362 0.722λ, and 1.444λ, respectively) under these conditions, even assuming th Fig. 6 repeats at higher values for abscissae, e.g. for the $d = 3\lambda/4$ binod mode of resonance. Accordingly, it may be concluded that the minimum $d = 0.465\lambda$ is caused by either a second-degree-of-freedom mode in one of th resonators or a new characteristic mode of resonance.

The minimum predicted by Fig. 6 for $d = 0.20\lambda$ approximately, is probably beyond the limits of the experiments; the experimental results were unreliable

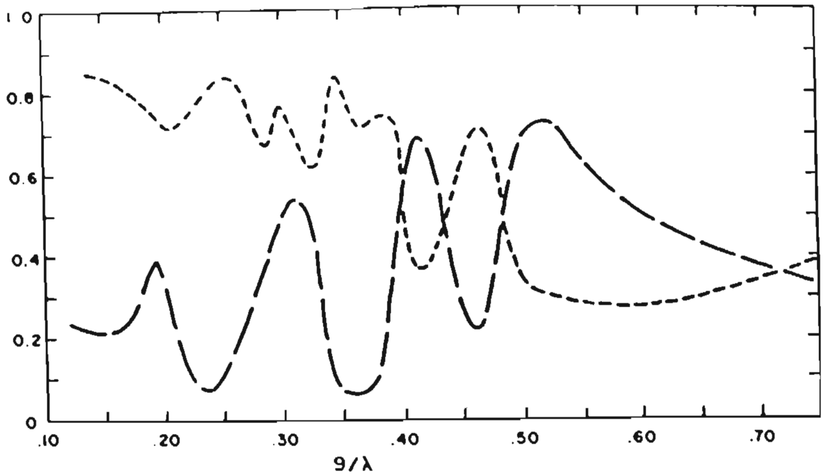

FIG. 4.—REFLECTIVITY AND TRANSMISSIVITY FOR SECOND BATTERY

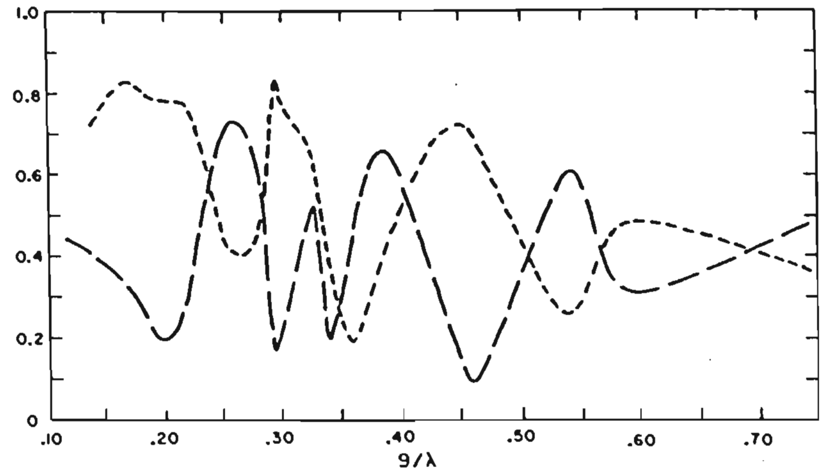

FIG. 5.—REFLECTIVITY AND TRANSMISSIVITY FOR THIRD BATTERY

at such large periods. However, this may explain the minima: $d = 0.27\lambda$ in Fig. 3; $d = 0.15\lambda$ in Fig. 4; and $d = 0.20\lambda$ in Fig. 5.

NONUNIFORM DEPTHS

Experimental Arrangement.—The apparatus was similar to that of the earlier experiments (2), except that an artificial rectangular invert was inserted in the resonator. The geometry is depicted in Fig. 7, and detailed in the following:

h = 8.16 in.	w = 10.0 in.	λ = 23.41 in.
h_R = 1.15 in.	W = 18.0 in.	

Here d varied between 0.5 in. and 14.5 in. and the incident wave height wa constant at approximately 0.5 in. The still water depth, period (and wavelength were held constant, and only one entrance channel width and resonator width wa examined. The length of the resonator was changed by retracting the rear wal outwards. Reflectivity and transmissivity were measured as before, and th results are plotted in Fig. 8 after having applied corrections for attenuation Note that the wave length used in these results is that computed for the harbour entrance channel.

Results.—The results indicate a reduced efficiency for resonators of comparatively small resonator still water depth. Also, lower values of reflectivit (and therefore upstream agitation), are associated with given transmissivit than is the case for uniform depths. The mechanism is clearly affected by th impedance offered by the abrupt change of depth at the mouth to the wave entering the resonator. The ratio of still water depth in resonator to still wate

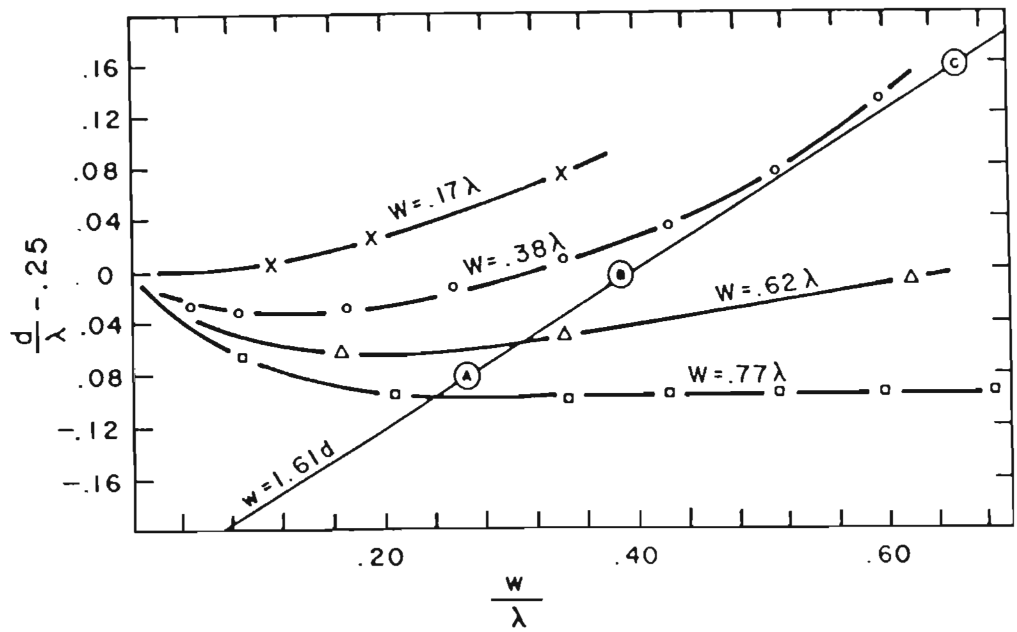

FIG. 6.—GEOMETRY FOR RESONANCE

depth in channel in the test was 0.14, which may be regarded as an extrem case. Higher relative depths would no doubt improve the overall performance

Analysis.—The abscissa in Fig. 8 refers to the incident wavelength, fo purposes of comparison with the results from uniform depth geometry. However, the tuning of the resonator is related to the wavelength in the resonator A singular phenomenon was detected in the range $0.52\lambda < d < 0.61\lambda$, when th surface of the junction element in the main channel contiguous with the resonator mouth assumed the irregular choppy appearance typical of multi directional waves of short period. Probably resulting from noncomitant energ loss, the values of transmissivity clearly decline in this range, and the value of reflectivity show instability.

Surprisingly, higher and irregular values of transmissivity, associate with high values of reflectivity were apparent at the second harmonic. Not that the length of the resonator for resonance has been reduced to 0.45 of tha

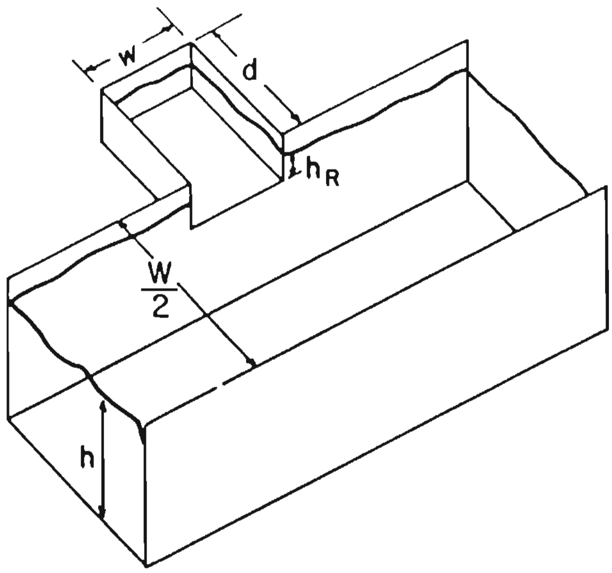

FIG. 7.—VARIABLE DEPTH GEOMETRY

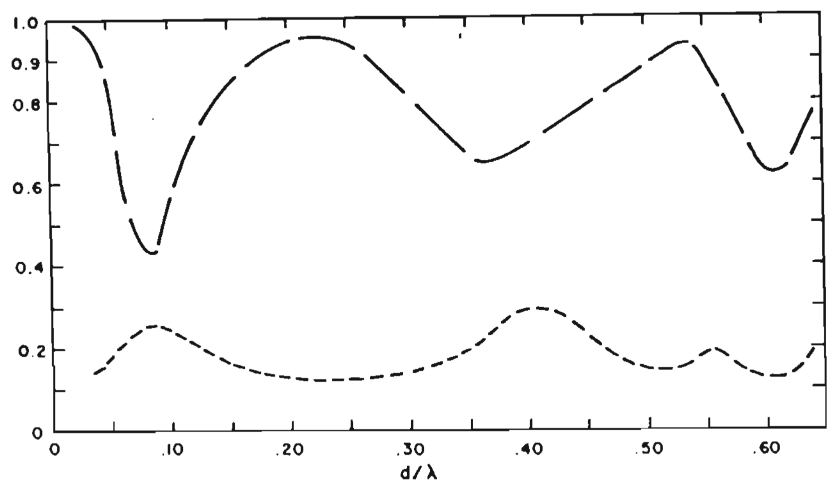

FIG. 8.—REFLECTIVITY AND TRANSMISSIVITY FOR VARIABLE DEPTH GEOMETRY

required for uniform depths. The depth was approximately 1/7 of the entrance channel depth.

CONCLUSIONS

Results of experiments on batteries of three resonators and nonuniform water depths in the resonator-harbor entrance channel area have been presented. The results indicate that a wider frequency band of the incident ocean wave spectrum will be reflected (and not transmitted into the harbor) if the distance between the resonators in a battery is real. The resonators should not be contiguous.

A method for estimating many of the frequencies of the resonant peaks in the battery response is also given. The method is based on previously published experimental results, and does not yield frequencies for all resonant modes.

Results for the tests on nonuniform depths indicate that the cost of constructing resonators can be considerably reduced by reducing both the lengths and depths of each resonator. The efficiency would, however, also be reduced.

ACKNOWLEDGMENTS

Guidance and facilities were provided by Jack Allen and G. D. Matthew of Aberdeen Univ., Scotland, and A. Brebner of Queen's Univ., Canada; financial assistance by the Univ. of Natal, S. Africa, and the CSIR.

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APPENDIX II.—NOTATION

The following symbols are used in this paper:

d = length of resonator;
h = still water depth;
h_R = still water depth in resonator;
W = entrance channel width;
w = resonator width;
$1/Q$ = tuning parameter d/λ band width; and
λ = wavelength in entrance channel.

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Desk-Top Model of Harbor Resonators

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A simple small-scale model of a generalized harbor entrance with rectangular resonators was set up. Using simple electronic equipment and sound waves of the order of 1000 cycles, the geometry for resonance was determined. The results are compared with those obtained from a hydraulic model, using gravity waves of similar wavelength. It was found that the results were not influenced by imperfect boundaries or reasonable changes in frequency. It is concluded that the results are sufficiently accurate to warrant further development of the method as an aid for harbor design, particularly in view of the simplicity and size of the equipment.

Rectangular resonators have been recommended from time to time as a means of eliminating harmful wave frequencies from the spectrum incident upon a harbor. Relevant geometry is shown in Fig. 1. The recommended[1,2] length of the resonator is one-quarter of the wavelength to be reflected back into the ocean.

More recently, it was found[3] that the end effect for such resonators is considerably influenced by the width of the velocity field at the mouth of the resonator. In fact, this effect was sufficient for quarter-wavelength resonators to be rendered completely ineffective for certain harbor-entrance geometries.

It was decided to extend the scope of the hydraulic experiments by using an acoustic model, since the latter evidently has certain advantages: wave generation and absorbtion is easier; the fluid does not have to be isolated; wavelengths are generally shorter; and measuring equipment, speed, and accuracy are better.

It is not intended at this stage to develop the technique in detail, but the results obtained indicate that such development may prove useful to harbor-design engineers.

The Experiments: The variable geometry shown in Fig. 1 was built up of 3-in. movable timber walls, placed on a glass plate on top of a desk. A second glass plate was placed on top of the walls. A loudspeaker was connected to an oscillator and placed against sponge rubber at the entry to the main duct. Note that the continuous longitudinal wall of the main duct was taken to be perfectly reflective, and hence equivalent to the centerline for symmetrical geometry (i.e., with resonators on both sides of the duct).

A microphone was placed just inside the sponge-rubber absorber at the "harbor" end of the main duct and was connected through a small preamplifier to an oscilloscope. The whole arrangement is shown on Fig. 2.

By setting the widths of the entrance duct and the resonator and holding the oscillator frequency constant, the length of the resonator was adjusted until resonance occurred. This was monitored by a minimum signal on the oscilloscope. The test was repeated for the geometry listed in Table I.

Results: The wavelength was calculated (room temperature was 23°C), and the results were plotted nondimensionally for various

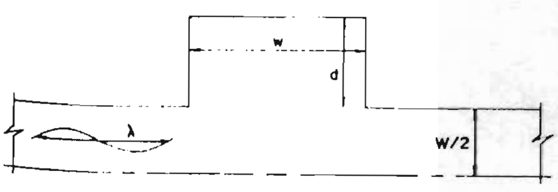

FIG. 1. Resonator geometry.

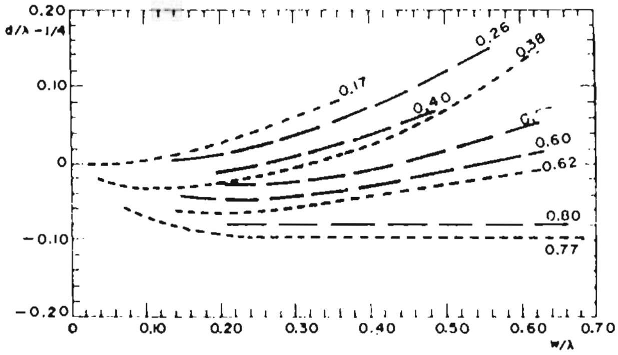

FIG. 2. Experimental results; geometry for resonance. Dashed lines refer to acoustic model, dotted lines to hydraulic model, for various values of W/λ.

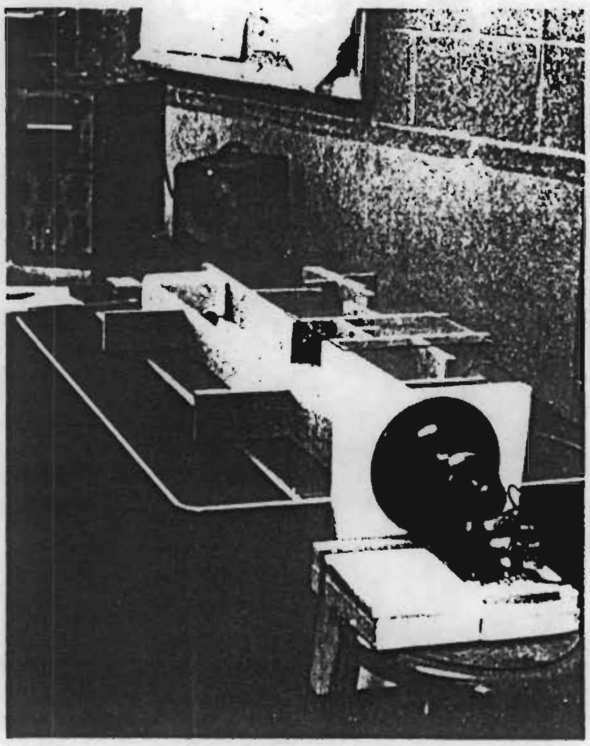

FIG. 3. Experimental apparatus.

TABLE I. Geometry tested.*

$W=12$	$N=900$	$w=3.2, 6, 9, 12$
$W=8$	$N=900$	$w=3.2, 6, 9, 12$
$W=6$	$N=900$	$w=3.2, 6, 9, 12$
$W=6$	$N=600$	$w=3.2, 6, 9, 12$
$W=12$	$N=677$	$w=3.2, 6, 9, 12$
$W=12$	$N=600$	$w=3.2, 6, 9, 12$

* All dimensions in inches or cycles per second.

values of W/λ as shown by the dashed lines in Fig. 3. The dotted lines were obtained from the hydraulic model. The shaded area refers to the zone where distinct resonance was obtained. Outside this zone, the resonance is not clearly defined.

The effect of the reflectivity of the absorber was checked by moving it along the duct. This alters the position of the acoustic pressure envelope in the main duct if there are reflections present. No effect was noticed on either the pressure monitored at the oscilloscope or the geometry for resonance.

In addition, the effect of sound waves entering the duct along its length was insignificant, since the pressure was found to be essentially constant across the cross section at a reasonable distance from the mouth of the resonator. The signal-to-noise ratio was improved by means of the amplifier on the oscillator.

The last test run was designed to test the effect of scale. The results plotted on the same curve obtained for the second test run.

Conclusions: Although there are slight differences in the results obtained from the two models, Fig. 2 indicates that reasonably good results can be obtained in the zone of distinct resonance. Certainly the results are to be preferred to the blanket quarter-wavelength recommendation.

No scale effects were detected in the acoustic model, at least for the frequency range used in the experiments. Moreover, no effect on the geometry for resonance arising from imperfect walls and absorbers was discernible.

The test indicates that reasonably accurate predictions of harbor-resonance modes and frequencies can be obtained using an acoustic model; however, further development of the method may be necessary if absolute values of harbor amplification are required. It may be noted that the results were obtained in a few hours (several months in the hydraulic model) and at negligible expense.

Acknowledgments: The writer wishes to record his gratitude to Professor H. Stewart of the Department of Electrical Engineering, who kindly loaned the equipment. Dr. Richard Shaw, of the ESSA tsunami group at the University of Hawaii, deserves most of the credit for suggesting the experiments.

* Senior lecturer on leave from Univ. of Natal.

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7336 Response of rectangular resonators to ocean wave spectra

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7336 Response of rectangular resonators to ocean wave spectra

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The design problem for harbour entrances includes the elimination of those band-widths in the incident wave spectra that cause range action and high mooring forces, unreasonable wave forces and wave overtopping, and the drift of littoral sediments into the harbour. This Paper describes a method to tune a resonator accurately, in order to eliminate such harmful frequencies. Earlier work on resonators is listed, and experiments performed are briefly described. The results presented clearly demonstrate the effect of the end contraction. Using maximum attainable values of reflectivity, minimum values of transmissivity and approximate sharpness of the resonance, an approximate method is devised for estimating the performance of resonators, assuming a linear system. The response for a typical design incorporating the correction for the end contraction is computed, using a Breitschneider spectrum. This is compared with the response of a standard resonator, and it is concluded that the method allows the design of more effective and cheaper resonators.

Notation

d	length of resonator
e	exponential
E_t	total energy in the incident wave spectrum
h	still water depth
H	wave height
i	subscript denoting incident wave spectrum
Q	tuning parameter bandwidth at half resonant value
R	subscript denoting resonant value
$S_H2(\sigma)$	spectral energy density distribution
T	wave period
w	width of resonator
W	width of harbour entrance channel
α	reflectivity
β	transmissivity, or, as a subscript, transmitted value
σ	circular frequency
λ	wavelength

Subscripts $_0$, $_1$ and $_2$ refer to peak energy value, and first and second computed examples respectively.

Introduction

Range action causes large mooring forces in harbours,[1] and in important parts of a harbour (e.g. a tanker terminal), which may resonate at ocean wave frequencies that occur relatively often in the locality of the harbour. Moreover, harbour installations such as sand by-passing units are susceptible to wave force damage. In these problems the forces are approximately proportional to the square of the local wave height, and, hence, any reduction in the amplitude of the waves entering the harbour will be of considerable value: e.g., if the

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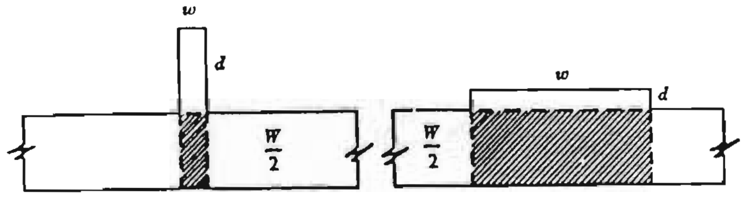

Fig. 1. Resonator geometry

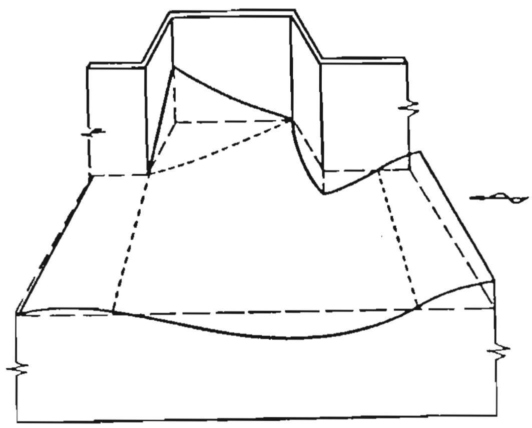

Fig. 2. Surface shape for mode 1

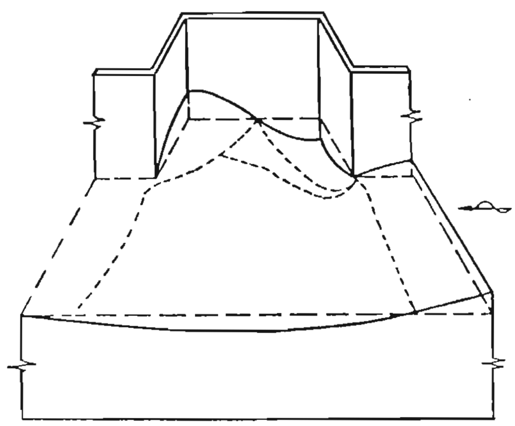

Fig. 3. Surface shape for mode 2

causative wave heights are reduced to one third of their incident values, the forces will be reduced to one ninth of their original values (the local amplification factor does not affect this proportional reduction of mooring forces). Similar arguments apply to local wave overtopping, especially where this hampers cargo handling. A further problem is the dredging of the harbour entrance channel. Where the wave conditions cause an ingress of littoral sediments, this may be reduced by prohibiting those frequencies that cause a high drift,[2] or at any rate by reducing the amplitudes of such frequencies.

2. It is necessary to process a large amount of wave recordings in order to identify the harmful wave spectra. It would then be feasible to design resonators at the harbour entrance, tuned to radiate back into the ocean those wave frequency bandwidths considered to be harmful. This Paper presents an approximate method for predicting the transmitted and reflected spectra from given incident spectra, using a simple configuration of rectangular resonators located at the harbour entrance. However, the method is restricted to linear waves.

Geometry

3. The geometrical variables are shown in Fig. 1. It will be seen that resonator geometry may vary between extremes in which the resonant motion is predominantly either in a direction parallel to that of the original incident wave (expansion chamber) or at right angles to it (branch canal).[3] The two modes are denoted modes 1 and 2 respectively, and the exaggerated shape of the free surface observed in the experiments is shown in Figs 2 and 3. These design procedures exclude the second resonant mode. Local velocities at the mouth of the resonator will be generally lower for mode 2 and this is clearly more desirable for navigation.

4. The linearity of the system examined allowed the continuous reflecting boundary opposite the resonator to be taken as equivalent to a totally reflective centreline. The configuration of Fig. 1 is, therefore, considered equivalent to one involving resonators on both sides of the harbour entrance channel, providing that the main channel width shown is doubled.

5. Uniform depths in the entrance channel and the resonators are investigated. Where the depth in the resonator is less than that in the entrance channel, the efficiency is reduced but, the optimum length of the resonator would be considerably reduced. Excluded from this work are all reflexions from the harbour domain towards the harbour entrance, and batteries of resonators tuned to cover a wide frequency band will not be discussed.

Earlier related work

6. Lamb[4] calculated the reflectivity (ratio of amplitudes of reflected and incident waves) and transmissivity (ratio of amplitudes of transmitted and incident waves) of shallow water waves past constrictions in channels. Rayleigh[5] examined acoustic resonators and derived a method for estimating the end effect; this did not include the case where the entrance channel was of finite width. Honda[6] *et al.* simplified Rayleigh's method to estimate resonant periods for tidal oscillations in bays, an analogous situation. Valembois[7] first suggested the application of rectangular resonators in harbour engineering. He recommended that the length and width of the resonator should be respectively a quarter and a half a wavelength. Miles and Munk[8] presented curves for evaluating the end correction for harbours. Nakazawa and Takano[9] investigated the expansion chamber mode by expanding the potential function for the wave train in each domain in series form. Horikawa and Nishimura[10] used a finite difference scheme to solve the fundamental equations for similar geometry, including complete reflexion from the harbour domain. To date (1970), no unified theory for the general behaviour of rectangular resonators is available.

Experiments

Wave channel

7. The wave channel is described in an earlier publication.[3] The maximum usable wavelength, limited by reflectivity, was determined experimentally to be of the order of 2 m, and the minimum wavelength, limited by attenuation, was found to be of the order of 0·5 m. General considerations relating the choice of wavelength tested to the geometry of the wave channel and measurement accuracy have been given elsewhere.[11] The filters and the absorber[3] effectively decoupled the experimental system.

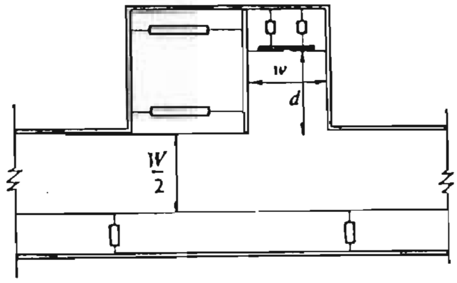

Fig. 4. Variable geometry

Variable geometry

8. In contrast to the usual methodology, in these experiments the wavelength was held constant and the geometry changed as shown in Fig. 4. This resulted in a high order of accuracy. The width of the harbour entrance channel was adjusted by means of a false continuous wall, and the width of the resonator was similarly adjusted. The rear wall of the resonator was systematically retracted outwards. Dimensions of the geometries tested are detailed in Table 1.

Wave measurement

9. The general wave measurement procedure has been described elsewhere.[12] Wave attenuation was directly measured, i.e. transmissivity at zero resonator length. All values of transmissivity for one such test run were then corrected by multiplying by the reciprocal of this transmissivity. It should be noted that significant errors can arise in the measurement of reflectivity of real waves. Errors introduced by the loop and node method have been studied by Goda and Abe.[13] Similar corrections[11] were applied to all the readings in this study. Corrected results were subsequently plotted, and those relevant are reproduced in Figs 5–12.

Table 1

h	λ	W	w	d
22	59·5	45·7	5·08, 12·3, 20·3, 25·4, 30·4, 35·6, 40·6	0→33
22	59·5	22·8	2·54, 5·08, 10·2, 15·2, 20·3, 25·4, 30·4, 35·6	1·2→31·8
22	74·2	45·7	12·4, 25·4, 45·7	1·9→33
22	134	22·8	15·2, 25·4, 45·7	0→45·7

All dimensions are in cm.

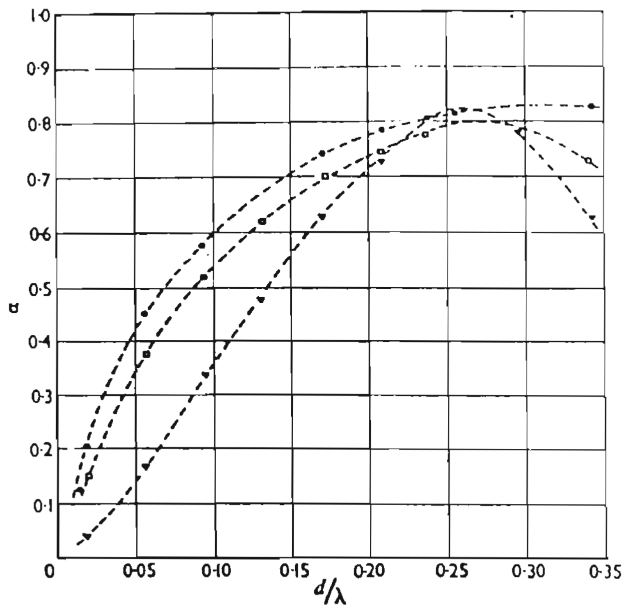

Fig. 5. $W=0{\cdot}17\lambda$

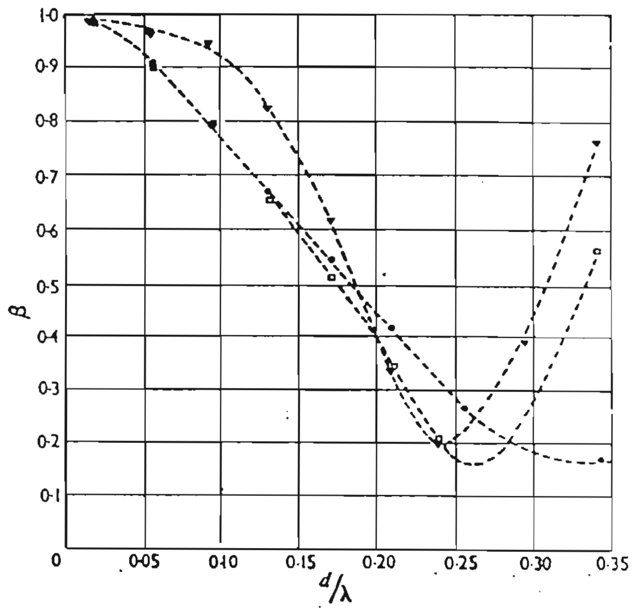

Fig. 6. $W=0{\cdot}17\lambda$

Figs 5–12. Reflectivity and transmissivity results

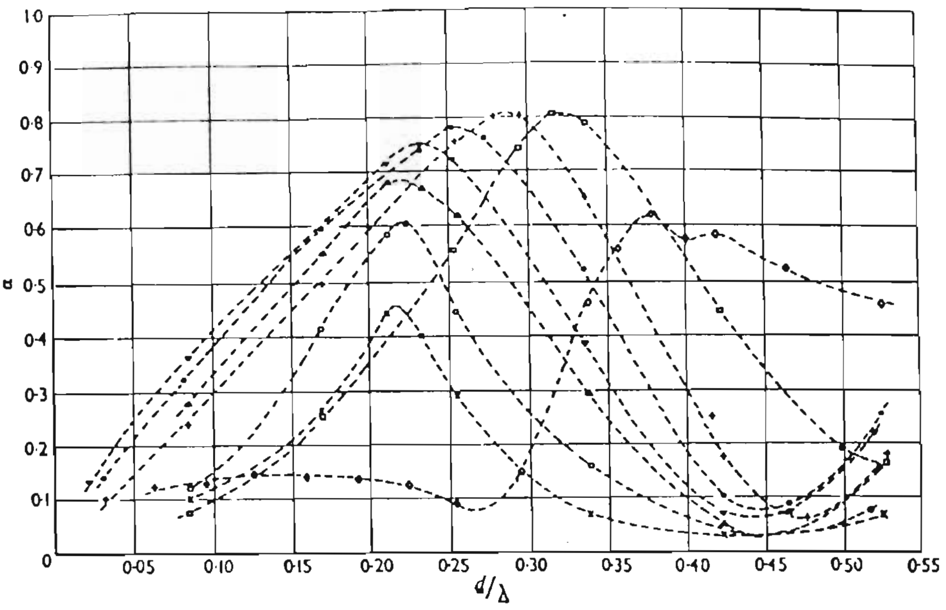

Fig. 7. $W=0{\cdot}38\lambda$

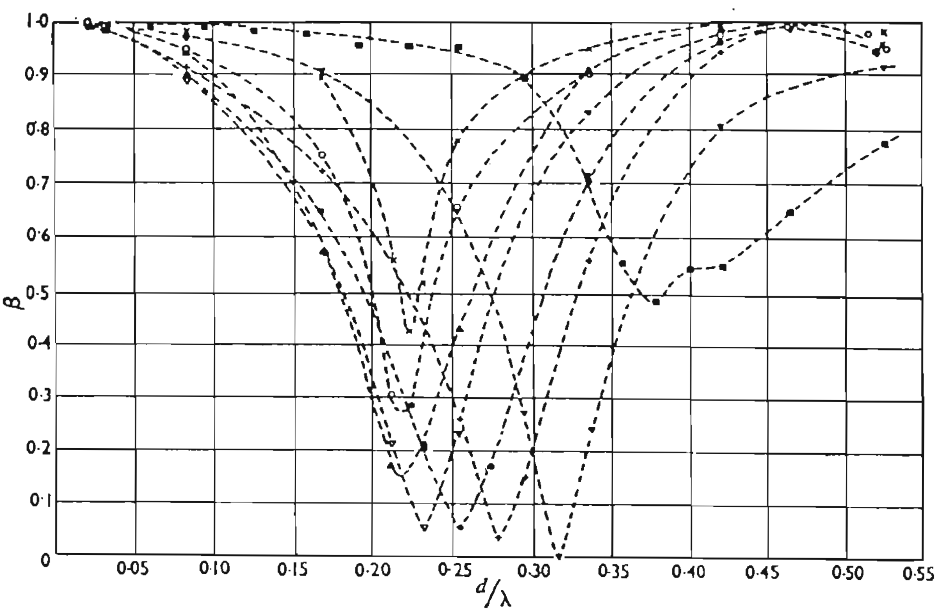

Fig. 8. $W=0{\cdot}38\lambda$

Figs 5–12. Reflectivity and transmissivity results

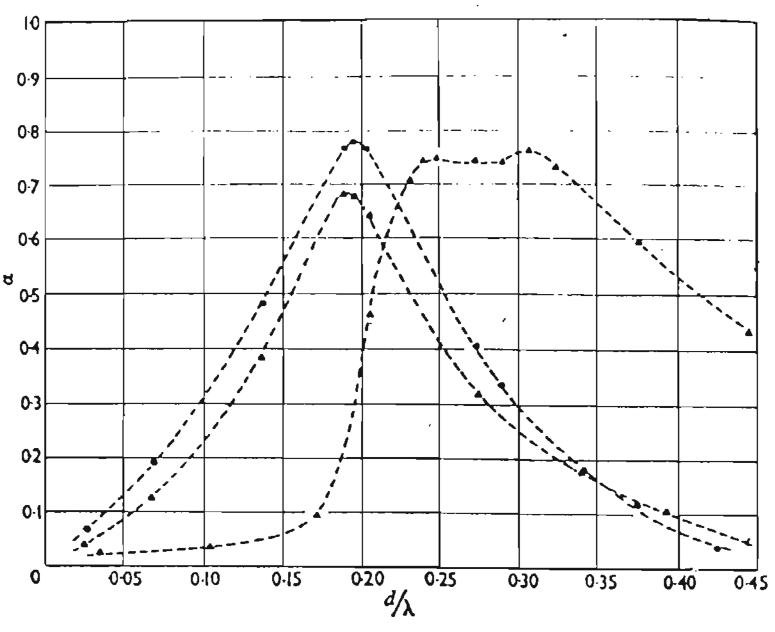

Fig. 9. $W=0{\cdot}62\lambda$

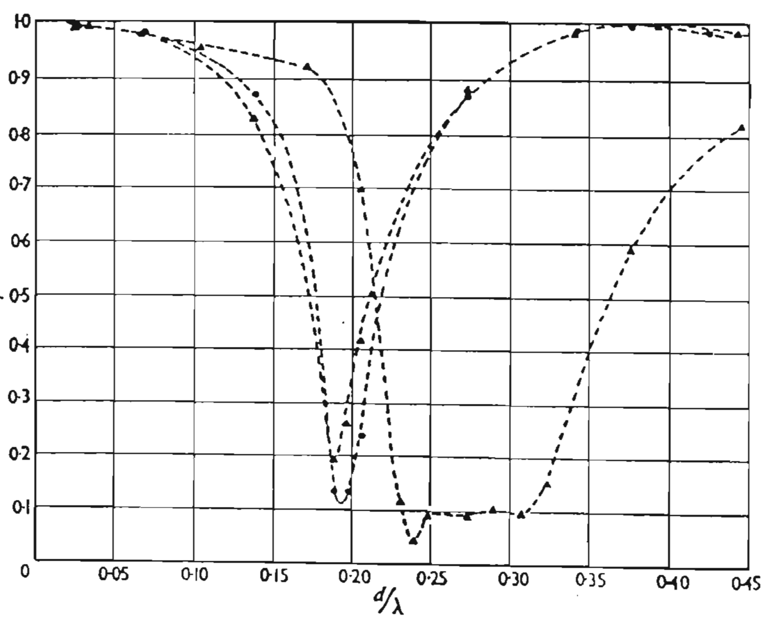

Fig. 10. $W=0{\cdot}62\lambda$

Figs 5–12. Reflectivity and transmissivity results

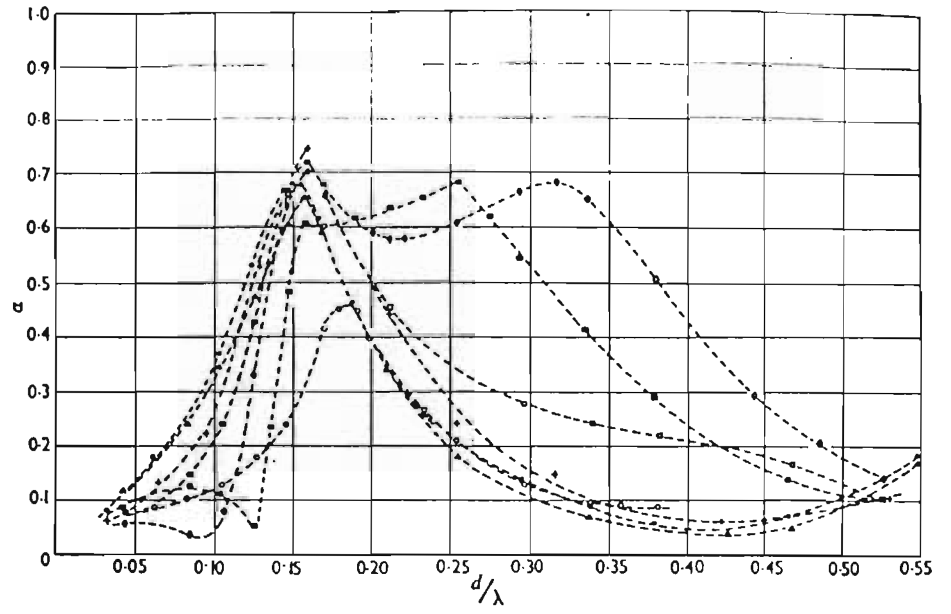

Fig. 11. $W = 0{\cdot}77\lambda$

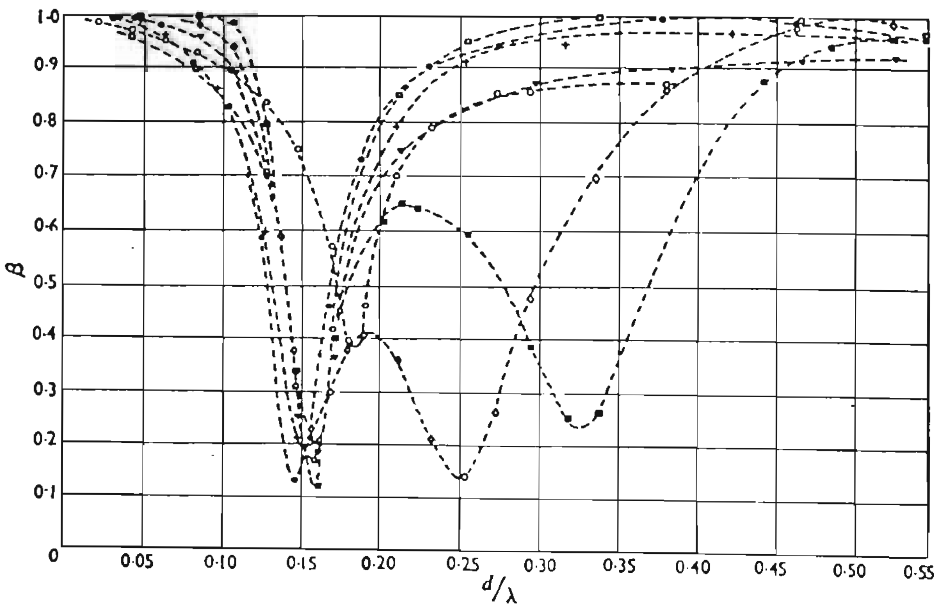

Fig. 12. $W = 0{\cdot}77\lambda$

Figs 5–12. Reflectivity and transmissivity results

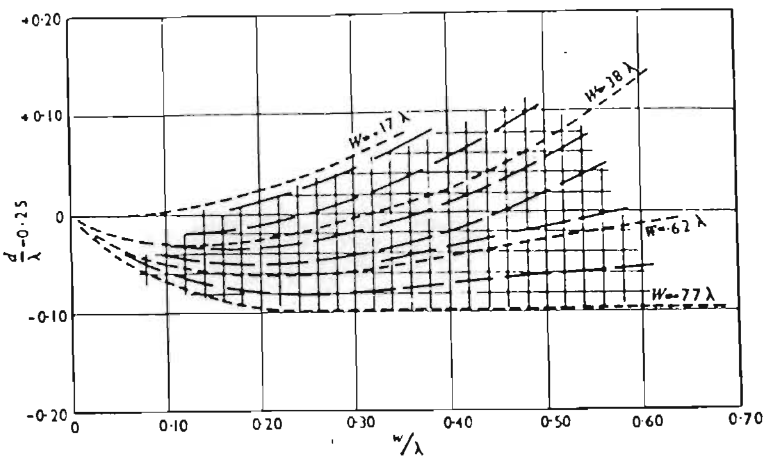

Fig. 13. Geometry for resonance : method applicable in shaded zone

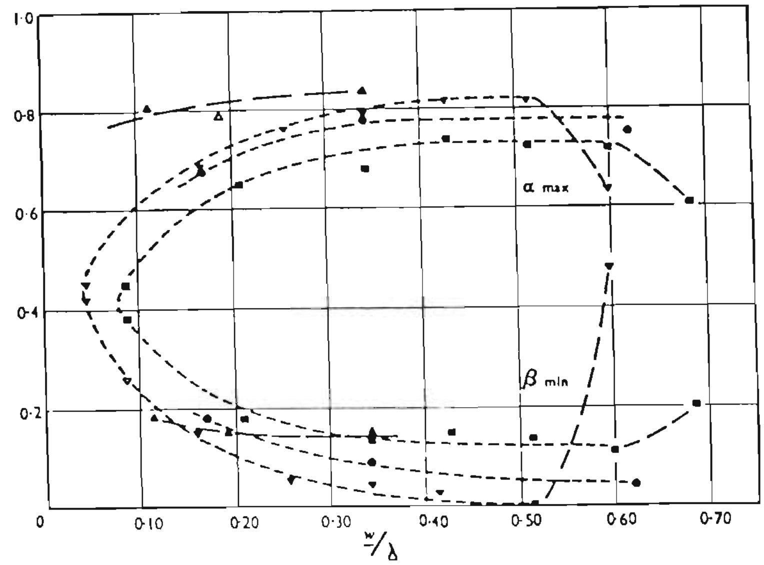

Fig. 14. Reflectivity maxima and transmissivity minima

Results

10. Abstracting from these plots, the geometrical conditions necessary for resonance are shown in Fig. 13. The end contraction[3] can be clearly seen.

11. The maximum values of reflectivity and minimum values of transmissivity achieved in the experiments are plotted in Fig. 14. It should be noted that these values are sensitive to incident wave amplitudes because steepness of the waves for these curves did not exceed 0·05 and it has been assumed that the plotted response was approximately linear, i.e. applied to small incident wave amplitudes. For increased amplitudes the velocities at the upstream re-entrant of the resonator and energy dissipation would be considerably increased. Tests were carried out in which the incident amplitudes were more than doubled, but plotted minima and maxima did not deviate

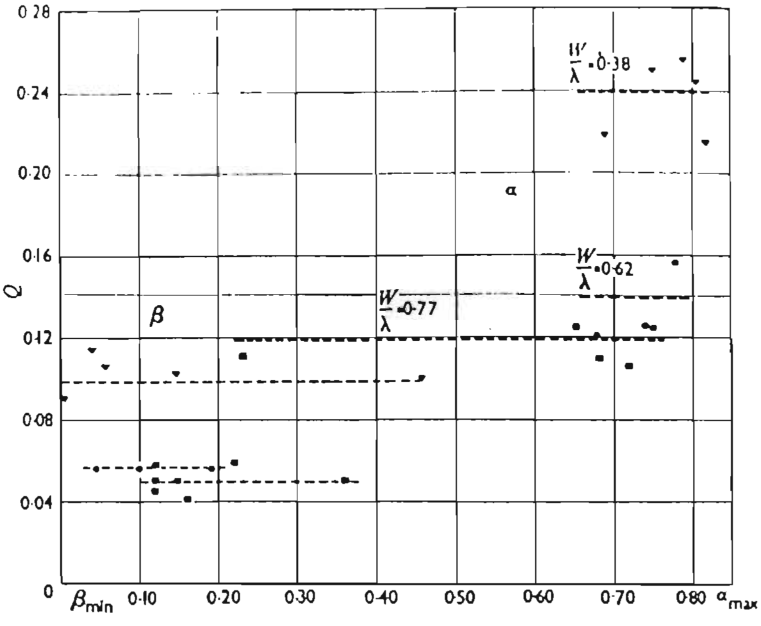

Fig. 15. Bandwidth at half resonant value

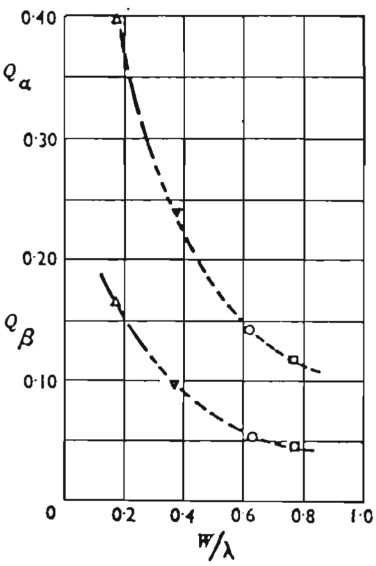

Fig. 16. Average bandwidth at half resonant value

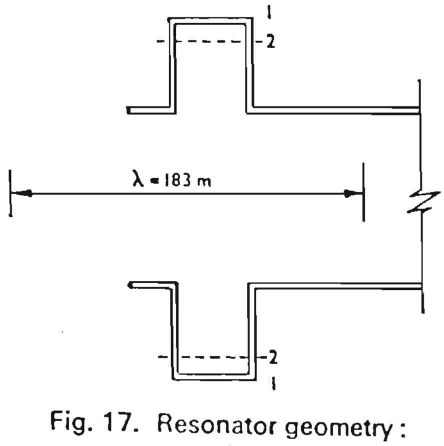

Fig. 17. Resonator geometry: computed example

by more than about 10% from those given in Fig. 14. This supports the contention that the values shown in Fig. 14 are close to the maximum and minimum attainable. Fig. 14 indicates that optimum resonator performance is independent of resonator width for $w/\lambda > \frac{1}{5}$, and that there is a tendency for resonator performance to gradually deteriorate with increase in main channel width. Poor performance can be expected for $W/\lambda < \frac{1}{5}$ and for $W/\lambda > 1$.

12. By fitting an approximate linear apex into the sharp portions of the resonance curves (Figs 5–12), it was discovered that the band widths of the tuning parameter d/λ were reasonably constant at the half peak value, for most of the range tested. These band widths are plotted in Fig. 15, and the average values in Fig. 16. It is possible to predict with some accuracy the response of a resonator of given geometry to any wavelength by means of Figs 13, 14 and 16, provided that the resonator geometry lies within the shaded area of Fig. 13.

13. It should be noted that, because the effect of energy dissipation has been ignored in this work, the response of steep waves will not be accurately predicted using these figures.

Spectral response

14. Following the lines of Miles and Munk,[8] the response of a resonator to a continuous spectrum of incident waves may be obtained by assuming a linear frequency response. The spectral response also provides a useful indication of the efficiency of a resonator responding to a range of individual frequencies.

15. Adopting the Breitschneider energy density spectrum[14] (all symbols are defined in the notation):

$$S_H2(\sigma) = \frac{5E_t}{\sigma_0}\left(\frac{\sigma_0}{\sigma}\right)^5 \cdot e^{-1\cdot 25}\left(\frac{\sigma_0}{\sigma}\right)^4$$

$$\sigma_0 = \frac{2\pi}{1\cdot 17} T_{1/3}$$

$$H_{1/3} = 2\cdot 83\sqrt{E_t}$$

and assuming the following values:

$$T_{1/3} = 12\cdot 7\ \text{s} \quad H_{1/3} = 3\cdot 66\ \text{m}$$

the Breitschneider spectrum is plotted in Fig. 18.

16. By way of example the response of two resonators is calculated:

(1) $W = 85\cdot 4$ m	(2) $W = 85\cdot 4$ m
$w = 35\cdot 5$ m	$w = 35\cdot 5$ m
$d = 45\cdot 8$ m	$d = 37\cdot 4$ m

(The geometry is shown in Fig. 17.)

17. For a harbour depth of 15·2 m the significant wavelength is 183 m. The first resonator is one quarter of a wavelength long, while the length of the second resonator accords with Fig. 13. The computation for the transmitted spectra is given in Table 2, and is explained in the Appendix. The transmitted spectra are obtained from:

$$S_H2(\sigma)_\beta = \beta^2(\sigma) S_H2(\sigma)_t$$

and are plotted in Fig. 18.

Table 2

λ	$\frac{W}{\lambda}$	$\frac{w}{\lambda}$	$\frac{d_1}{\lambda}$	$\frac{d_2}{\lambda}$	$\left(\frac{d}{\lambda}\right)_R$	β_{min}	Q_n	$\left(\frac{d}{\lambda}\right)_{\beta_{1/2}}$		β_1	β_2	$\beta_1{}^2$	$\beta_2{}^2$
800	0·350	0·150	0·187	0·154	0·225	0·14	0·105	0·173	0·278	0·46	0·73	0·21	0·53
750	0·384	0·160	0·200	0·164	0·220	0·13	0·095	0·173	0·268	0·31	0·64	0·10	0·41
700	0·400	0·171	0·214	0·175	0·215	0·13	0·090	0·170	0·260	0·14	0·51	0·02	0·26
650	0·420	0·185	0·231	0·189	0·210	0·13	0·085	0·168	0·253	0·35	0·35	0·12	0·12
600	0·467	0·200	0·250	0·205	0·205	0·12	0·075	0·168	0·243	0·66	0·12	0·44	0·01
550	0·510	0·218	0·273	0·224	0·200	0·12	0·070	0·165	0·235	1·0	0·42	1·0	0·18
500	0·560	0·240	0·300	0·246	0·195	0·11	0·065	0·163	0·228	1·0	0·82	1·0	0·67
450	0·622	0·267	0·333	0·273	0·190	0·11	0·055	0·163	0·218	1·0	1·0	1·0	1·0
1	2	3	4	5	6	7	8	9	10	11	12	13	14

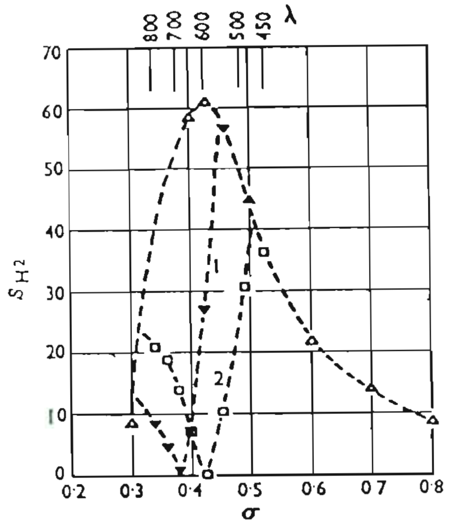

Fig. 18. Computed linear spectral response

Conclusions

18. Figure 18 clearly demonstrates the influence of the end contraction on the tuning of the resonator. The standard resonator reflects 26% of the total incident energy whereas the tuned resonator reflects 36%. For a wavelength of 168 m the standard resonator transmits all energy (87% of peak value) whereas the tuned resonator transmits about 25% of the peak value (wavelengths (in feet) are also plotted in Fig. 18).

19. The enhanced performance results from an 18% reduction in resonator length, a point of some practical significance.

Acknowledgements

20. The experiments were conducted at the University of Aberdeen, Scotland, with guidance and encouragement from Professor J. Allen and Dr G. D. Matthew. The Author received a grant administered by the South African CSIR. The method was developed and the paper written while on

leave from the University of Natal, Durban, and while visiting at Queens University, Canada, where assistance from Dr A. Brebner is acknowledged.

Appendix

21. Calculation of the response of a resonator is performed for discrete wavelengths (in feet) in the spectrum (see first column of Table 2). The next four columns follow from the specified geometry. Using values of W/λ and w/λ the resonant values $(d/\lambda)_R$ are obtained by interpolation from Fig. 13, the transmissivity minima from Fig. 14, and Q_β the average tuning parameter bandwidths at half resonant value from Fig. 16. The half value $\beta_{1/2} = 0{\cdot}5 + 0{\cdot}5\beta_{min}$. The value of the tuning parameter at this ordinate is obtained from $(d/\lambda)_R \pm 0{\cdot}5Q_\beta$ and is set out in columns 9 and 10.

22. Now the resonant curve can be constructed:

apex: $$d/\lambda = (d/\lambda)_R; \quad \beta = \beta_{min}$$

half values: $$d/\lambda = (d/\lambda)_R - 0{\cdot}5Q_\beta; \quad \beta = 0{\cdot}5 + 0{\cdot}5\beta_{min}$$
$$d/\lambda = (d/\lambda)_R + 0{\cdot}5Q_\beta; \quad \beta = 0{\cdot}5 + 0{\cdot}5\beta_{min}$$

The required transmissivity β_1 and β_2 are obtained by linear interpolation for $d/\lambda = d_1/\lambda$, d_2/λ.

23. It should be noted that the method generally results in conservative estimates of transmissivity at higher values, e.g. an estimated value of 0·85 may apply to a real value of 0·75. The reverse argument applies to reflectivity estimates.

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An experimental study of end effects for rectangular resonators on narrow channels

By W. JAMES

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An experimental study of end effects for rectangular resonators on narrow channels

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(Received 3 February 1970)

Quarter-wavelength resonators for harbour entrances and similar applications are usually designed and built without regard to the end effect. This paper describes tests on rectangular resonators using water waves of unique frequency in a long narrow wave channel fitted with a wave generator at one end, a wave absorber at the other end and suitable wave filters. In these tests the wavelength was kept constant while the geometry of the rectangular resonant branch canal was systematically varied. Wave transmission across the resonator, as well as wave reflexion upstream of the resonator was measured. The results clearly indicate a considerable effect of main channel width on optimum resonator width and resonator length, invalidating the usual resonance theory (which predicts complete reflexion of the incident wave train when the length of the branch canal is one-quarter of a wavelength). For example, it is found that, under certain conditions, quarter-wavelength resonators may not have *any* effect on the incident wave. The work described is limited to the first resonant mode and to semi-infinite domains on each side of the resonator.

Introduction

A resonator is taken to be a short rectangular branch canal on a narrow main water wave channel. Resonator geometry may be considered to be intermediate in shape between extremes that are either long and narrow, when the motion is in a direction normal to that of the waves in the main channel, or short and wide, when the motion is parallel to that in the main channel (see figure 1). Each of these cases can be handled by one-dimensional wave-propagation theory (James 1968), but the generalized behaviour for intermediate shapes has not been treated mathematically, so far as the writer is aware. (It is hoped that the publication of these results will stimulate interest in a mathematical analysis of the problem.) Valembois (1953) investigated models of short rectangular resonators built orthogonally onto parallel entrance breakwaters of a harbour, but did not observe any end effects. Rayleigh (1929, p. 487) had previously developed an approximation for the end effect of an acoustic tube in a semi-infinite domain, an analogous situation. In this investigation the configuration tested was similar to that of Valembois; the effect of distributed reflexions from the downstream domain was not considered.

In general the dimensions of a resonator should be carefully chosen so that the mass of liquid contained within the resonator and that part of the main channel contiguous with the resonator mouth, i.e. the *junction element* shown in figure 1, has a fundamental frequency equal to that of the incident wave. The incident wave is to be totally reflected back into the upstream domain. For the purposes of this paper the lowest-frequency eigenvalue is used; where a battery of resonators is tuned to cover a wide frequency band, resonance also occurs at higher harmonics.

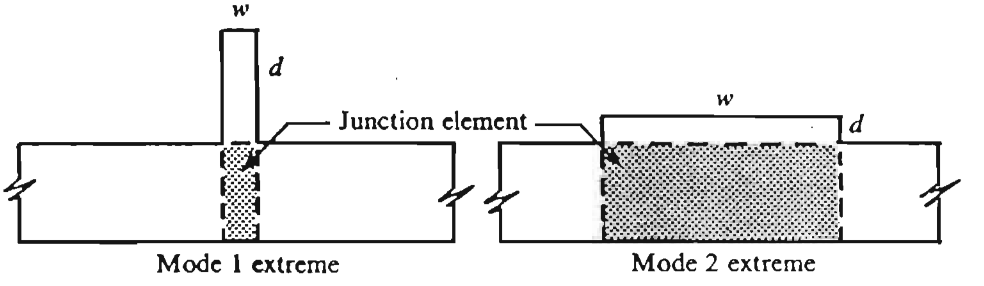

FIGURE 1. Resonator geometry.

Evidently, from the work by Penny & Price (1952), the maximum zone of protection in the downstream domain provided by a resonator at resonance will be limited to a width of one half the incident wavelength. Observations certainly show that wave energy transmitted into the downstream domain increases directly with an increase in major channel width above one wavelength, for the case where resonators are placed on both sides of the main channel. It follows that a resonator investigation of this type can be limited to a major channel width of one wavelength. Le Mehaute (1961) found that wave propagation is one-dimensional at distances of the order of twice the depth from a discontinuity in such narrow channels. This was confirmed in these experiments (except where the width was close to an integral number of half-wavelengths, when a transverse oscillation tended to be set up).

Limiting the treatment to semi-infinite domains results in simpler mathematics (James 1968) but introduces complications into the experiments, since the wave channels should be completely decoupled in both domains. Further mathematical simplification results from the use of first-order linear wave theory, but, for accuracy, non-linear effects in the experimental waves must be accounted for (Goda & Abe 1968). Consequently, in this study wave measurement was effected by micrometers, and the readings were correct to the nearest 0·005 inch.

Typical but exaggerated shapes of the water surface for the first two resonant modes are sketched in figures 2 and 3. Evidently the oscillation differs significantly from that to be expected for quarter-wavelength (i.e. long and narrow) resonators. One consequence of this discrepancy is reported in this note.

Scope of the experiments

The water depth (usually 8·65 in.) and wave period were kept constant throughout each sequence of changes of geometry; reflectivity (ratio of amplitudes of reflected and incident wave trains), transmissivity (ratio of amplitudes of

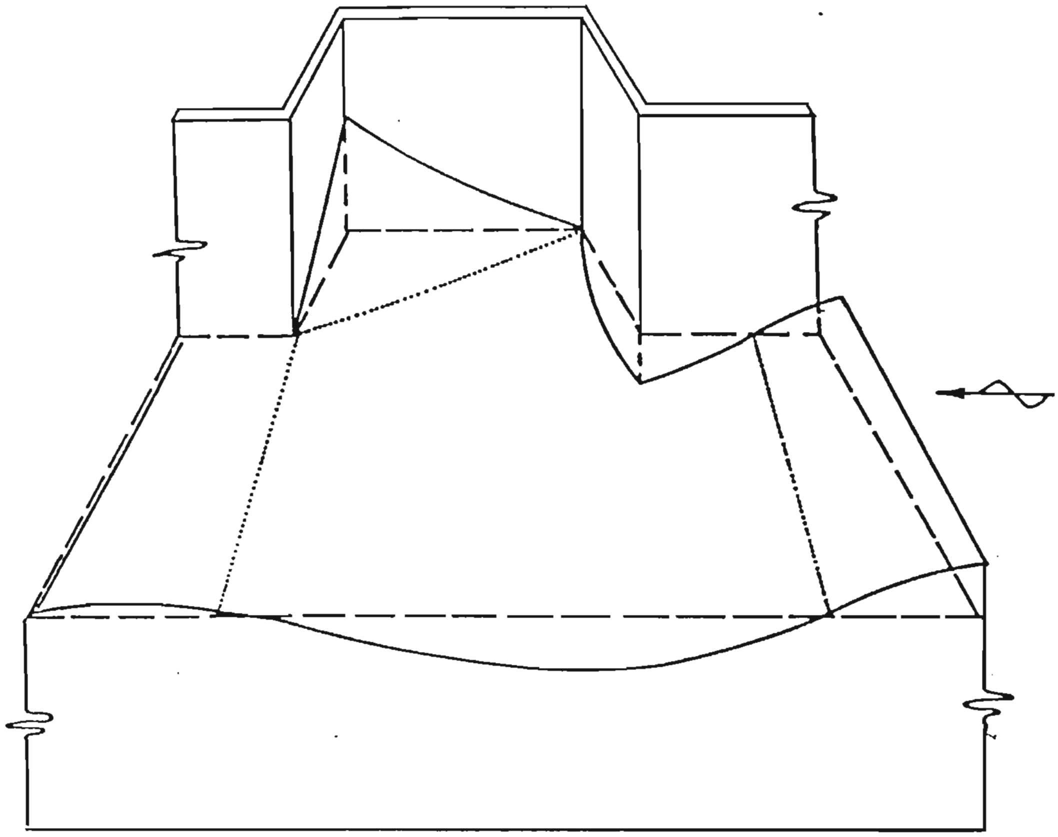

FIGURE 2. First resonant mode.

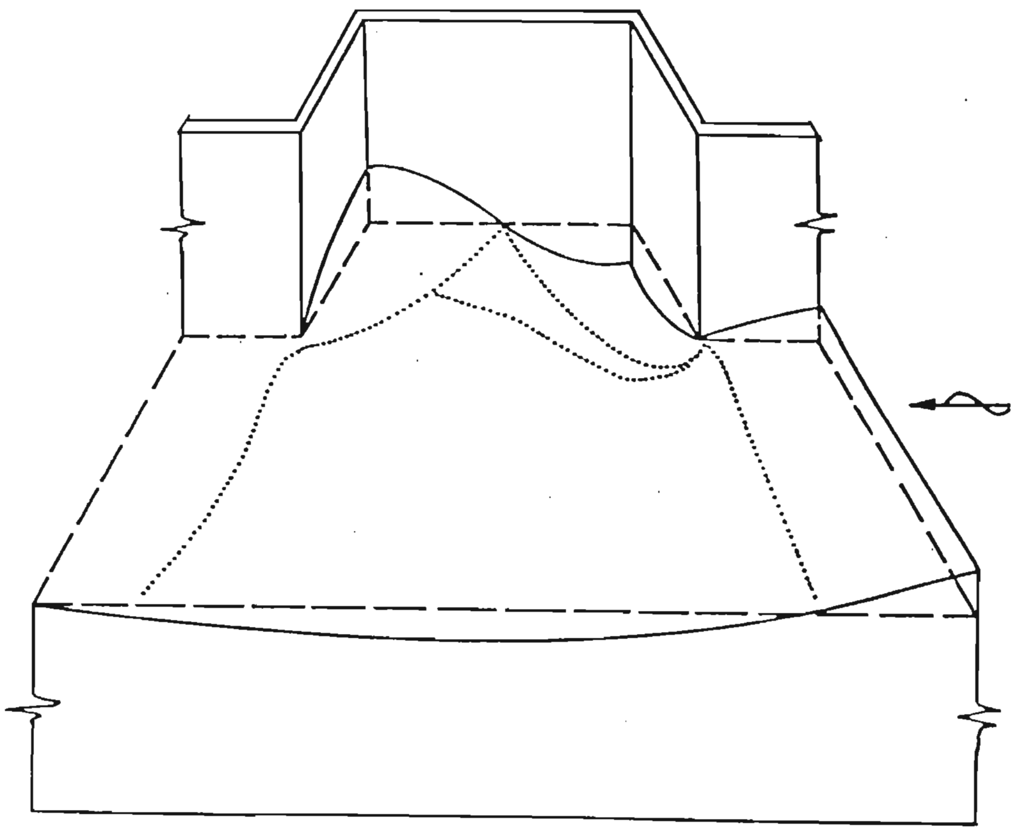

FIGURE 3. Second resonant mode.

transmitted and incident wave trains), and resonator activity were measured for each geometrical configuration tested. The geometries were chosen to give an even spread of results, and to avoid the frequencies resulting in transverse resonance.

All experiments were carried out in a 9 in. wide Perspex wave channel, approximately 50 ft. long, with the resonator housing equidistant from each end. The paddle arrangement was capable of delivering consistent waves of unique period over a sustained time. (A variation in period of 0·001 sec was the maximum tolerated.)

An intermediate Perspex wall was used to vary the width of the main channel, W, between the limits

$$0{\cdot}17 < W/\lambda < 0{\cdot}77,$$

where λ = wavelength.

Adjustable side walls were used to vary the widths of the resonators w for each main channel width tested within the limits

$$0{\cdot}05 < w/\lambda < 0{\cdot}69.$$

The rear walls of the resonators were also adjustable and were clamped sequentially in 8 positions for each resonator width. Resonator geometry tested varied between the limits

$$0{\cdot}23 < w/d < 4{\cdot}7,$$

where d is the length of the resonator. The symbols, w and d are also defined in figure 1. The geometries tested are summarized in table 1.

h	λ	W	w	d
8·65	23·41	18·0	2·0, 4·85, 8·0, 10·0, 12·0, 14·0, 16·0	0 → 13·0
8·65	23·41	9·0	1·0, 2·0, 4·0, 6·0, 8·0, 10·0, 12·0, 14·0	0·5 → 12·5
8·65	29·25	18·0	4·87, 10·0, 18·0	0·75 → 13·0
8·65	52·63	9·0	6·0, 10·0, 18·0	0 → 18·0

TABLE 1. All dimensions are in inches. These tests refer to the data summarized in figure 6

Experimental arrangement

Two 36 in. long Neyrpic wave filters, constructed from 16 gauge perforated zinc plate at 0·5 in. pitch, were used in both domains. Measured transmissivity for each filter varied between 0·4 and 0·6, depending upon wavelength, whence the *decoupling coefficient* for each domain was at worst 0·06, assuming total reflexion off the paddle. The absorber was similar to that constructed by Hamill (1963), and reflectivity was of the order of 0·02. Measurements of reflectivity for the entire domain 2 arrangement demonstrated two points: (*a*) decreasing transmissivity with decreasing wavelength, caused by viscous damping; (*b*) increasing reflectivity with increasing wavelength, caused by the limited dimensions of the filters and absorber. The range of wavelengths tested was carefully limited to minimize both these effects, but, for the viscous damping effect, corrections were applied to all results.

Wave measurement

Where reflectivity is to be measured to 1 %, envelope heights should be measured to 0·5 %. By using fine probes, and limiting instantaneous contact to a depression of the crest surface rather than actually puncturing it, it is usually impossible to see any meniscus effect. Hence sharpened stainless steel probes, nominal diameter about 0·015 in., were connected to an oscilloscope with a stationary time base. Dispersive agents further reduced surface-tension effects and resulted in clear signals being relayed to the cathode ray oscilloscope.

As a check on the consistency of the experimental waves, standard deviations were measured using a dekatron counting unit. The count recorded the 'number of crests greater than' (or 'number of troughs less than') for each setting of the micrometers attached to the probes. Batches of 100 consecutive waves for

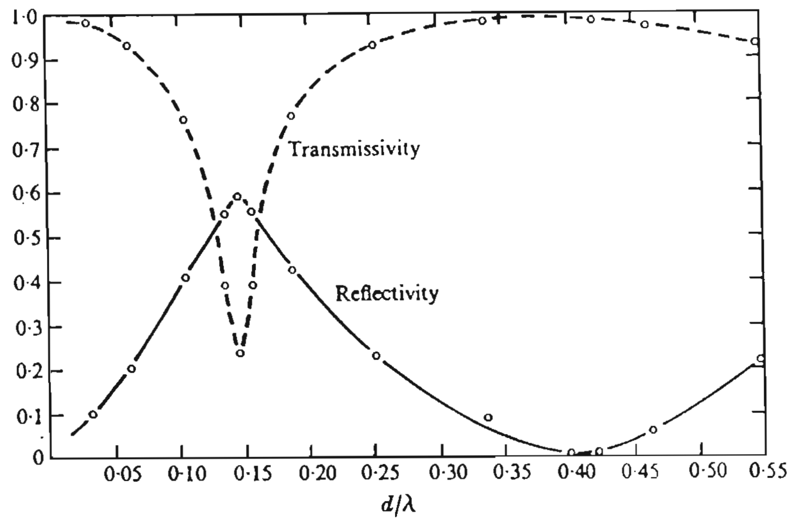

FIGURE 4. Typical results for narrow resonators. $w/\lambda = 0{\cdot}34$, $W/\lambda = 0{\cdot}77$, $2a/\lambda = 0{\cdot}049$.

micrometer increments of 0·0005 in. were counted, and histograms of the crest and trough relative frequency plotted. Most were approximately symmetrical, and computed standard deviations were of the order of 0·0015 in. for incident wave amplitudes of 0·25 in. Hence the mean reading was given by approximately half the number of a fairly large sample of waves, say 20. An analysis of the scatter on the curves with a high density of plotted points eventually indicated a random error of the order of 1 %.

Results

Healy's method (Goda & Abe 1968), also known as the loop-and-mode method (James 1969), was used to compute reflectivity and incident wave amplitudes. Since this method is based upon assumptions of linear theory and of only two wave components being present in the upstream domain, corrections had to be applied (Goda & Abe 1968). Results for each width of resonator were plotted as shown in figures 4 and 5. Corrections for attenuation, downstream reflectivity and non-linearity do not affect the geometry for resonance. The first peak in figure 5 applies to the first mode (figure 2); and the second (flatter) peak, the second mode of resonance (figure 3).

From the first maxima and minima of all these plots the family of curves in figure 6 was derived. The end effect is given by $(d/\lambda - 0{\cdot}25)$.

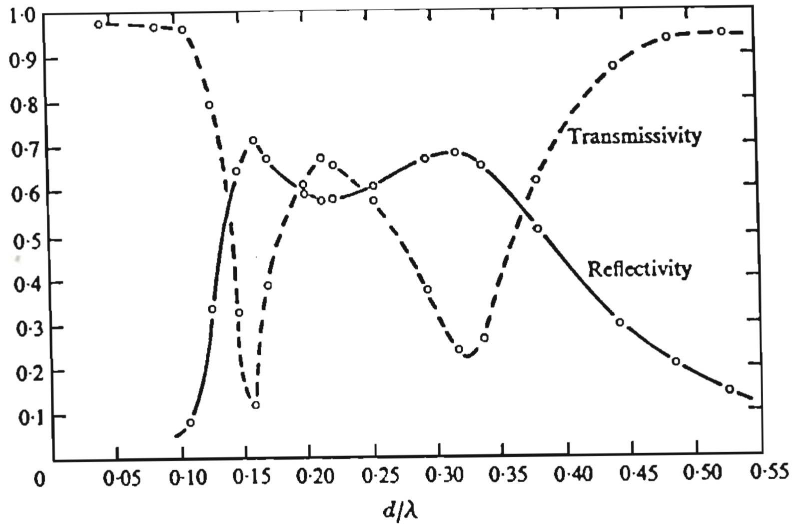

FIGURE 5. Typical results for wide resonators. $w/\lambda = 0{\cdot}60$, $W/\lambda = 0{\cdot}77$, $2a/\lambda = 0{\cdot}017$.

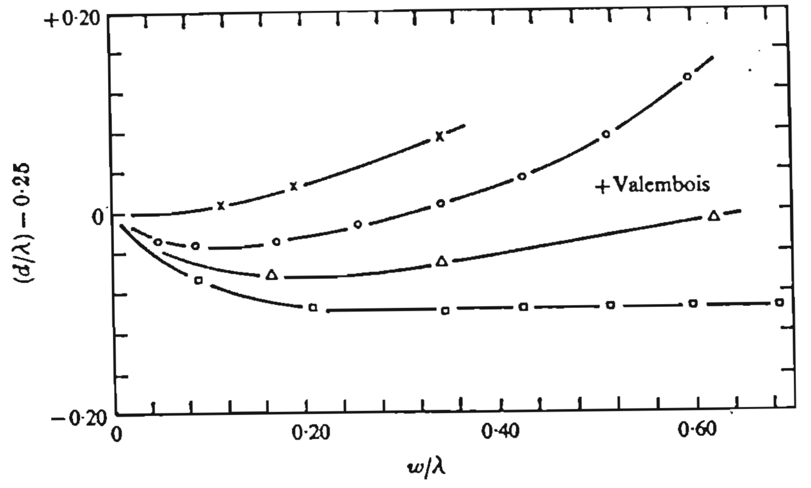

FIGURE 6. Geometry for resonance. Values of W/λ: ×, 0·17; ○, 0·38; △, 0·62; □, 0·77.

A number of tests were also carried out to investigate the effect of water depth, and it was found that geometry for resonance was not affected by such changes. Further tests also demonstrated that geometry for resonance was unaffected by incident wave amplitudes. Transmissivity minima and reflectivity maxima, were, however, sensitive to amplitude changes.

Conclusions

Resonator geometry for resonance is independent of still-water depth, wave amplitude, and energy dissipation in the resonator, and is given by figure 6.

It is clearly seen on figure 5 that the transmissivity for $d/\lambda = 0{\cdot}25$ is sufficiently high ($\beta = 0{\cdot}93$) to render the resonator ineffective. (For this case $W/\lambda = 0{\cdot}77$ and $w/\lambda = 0{\cdot}34$.)

The end effect for narrow resonators (say $w/d < \frac{1}{3}$), and for wide channels is negative, as shown on figure 6. This accords with the work done by Rayleigh.

From the same figure we see that for narrow entrance channels (e.g. $W/\lambda < \frac{1}{3}$) the end effect is generally positive, and that this effect increases with width of resonator. The geometry recommended by Valembois is indicated by a cross, and evidently is suited to a main channel width of approximately $0{\cdot}5\lambda$.

Figure 6 indicates that, for minimum resonator lengths, resonator and main channel widths should not be narrow. Such narrow widths in any case result in higher velocities and concomitant decreased efficiency.

The experiments were financed by the South African C.S.I.R. and the University of Aberdeen, and were conducted at the University of Aberdeen. Thanks are also due to Professor J. Allen and Dr G. D. Matthew for encouragement and help in the experiments. The paper was prepared at Queen's University, Ontario, where facilities were kindly made available by the Head of the Department of Civil Engineering, Dr A. Brebner.

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RESOLUTION OF PARTIAL CLAPOTIS

By William James,[1] A. M. ASCE

INTRODUCTION

Many published investigations on gravity waves of unique period (e.g. experimentation on spending beaches and breakwater units) quote measurements of boundary reflectivity accurate to 1%. Such results would demand wave measurements accurate to, say, 0.5%, and this effectively sets a limit to the minimum wave amplitude, viz. 200 × recording accuracy. In a recent study by the writer amplitudes were recorded to an accuracy of 0.01 mm which, for an accuracy of 0.5%, necessitated a minimum wave height of the order of 2 cm.

There is little point in recording wave heights to an accuracy that is spoiled by surface noise resulting from the limitations of the recording instruments or from boundary irregularities. Consequently the measurement accuracy should also be commensurate with the tolerances to which the experimental wave flume has been constructed.

For efficient decoupling and filtering, waves should not be much longer than about one-tenth of the length of the flume. This could set a limit to the maximum wavelength for consistent results, e.g. 1 m for common short laboratory flumes.

Clearly, then, the minimum steepness of the waves tested could depend upon the characteristics of the flume (length and tolerances) as well as the accuracy desired. In the preceding case the minimum steepness is 0.02. (Published experimental results often quote steepnesses ranging up to 0.08.)

It follows that many experimental waves capable of accurate amplitude measurement may be significantly trochoidal.

A common laboratory problem is to evaluate the amplitudes of both the original incident component wave and the reflected (negative) component wave in the observed partial clapotis. Two procedures often used to resolve the clapotis into these components are: (1) The "loop and node" method; and (2) the "three point" method. Both derive from the assumption of a sinusoidal wave profile. It is shown, herein, that these procedures could result in con-

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siderable error when reflectivity is high, and the real laboratory waves are significantly trochoidal.

REAL PARTIAL CLAPOTIS

In an attempt to estimate the reliability of the two techniques for resolving partial clapotis, the envelope for the real partial clapotis was computed using the fourth order equation given by Lamb (2).[2] Both real components were simply superimposed for this computation.

The actual coefficient of reflection was computed from the ratio

$$\alpha = \frac{\text{total real wave height of reflected component}}{\text{total real wave height of original component}} \quad \ldots\ldots\ldots\ldots (1)$$

LOOP AND NODE METHOD

This method involves laborious scanning for antinodes and nodes in the envelope of the partial clapotis. If E_L and E_N = the two envelope heights,

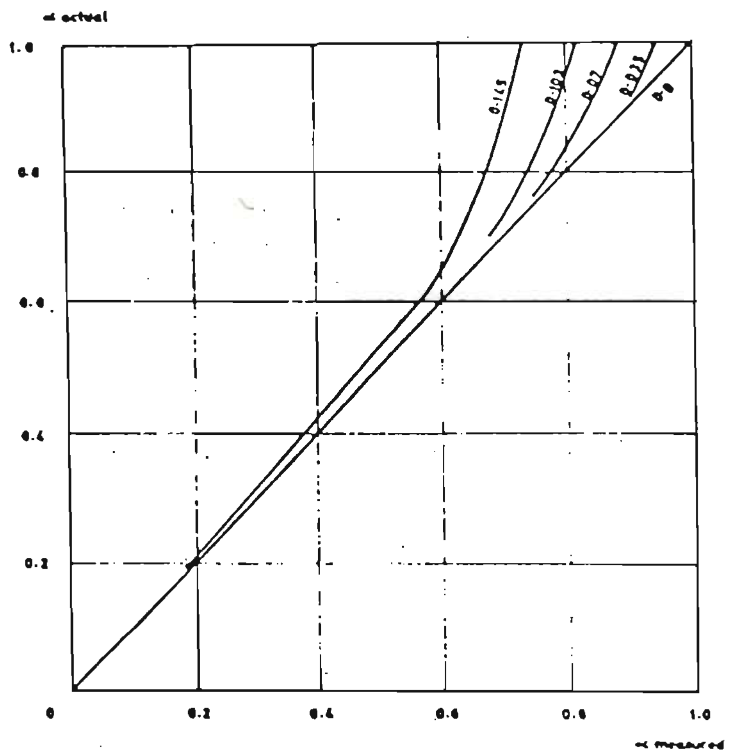

FIG. 1.—ACTUAL VERSUS MEASURED REFLECTIVITY FOR VARIOUS STEEPNESSES

and a = the incident amplitude, the well-known simple formulas

$$a = \frac{E_L + E_N}{4}; \quad \alpha = \frac{E_L - E_N}{E_L + E_N} \quad \ldots\ldots\ldots\ldots (2)$$

[2] Numerals in parentheses refer to corresponding items in the Appendix I.—References.

allow tractable control of those experiments requiring frequency variations.

It is easily seen that the shape of the trochoid has a serious effect when applying this method to such real clapotis: (1) The condition for both the upper and the lower bound of the antinode does not differ very much from first-order theory; (2) the condition for the upper bound of the node similarly differs little from first-order theory; it is, of course somewhat higher; (3) the lower bound of the node now occurs at the instant (and at a slightly different position) that the trochoidal surface of the original incident component coincides with the undisturbed surface, when the negative component trochoidal surface is slightly negative. Obviously, from (2) and (3), the nodal envelope is greater in trochoidal waves that is indicated by first-order theory.

A computer program was accordingly written to compute the measured coefficient of reflection from Eq. 2. The results are presented in Fig. 1 for various steepnesses.

THREE POINT METHOD

In this method (1), three envelope heights E_1, E_2 and E_3 are measured one-eighth wavelength apart and combined to yield

$$a^2 = \frac{1}{4}\left(E_1^2 + E_3^2 + \sqrt{4\,E_2^2(E_1^2 - E_2^2 + E_3^2) - (E_1^2 - E_3^2)^2}\right) \quad \text{(3a)}$$

$$\alpha^2 = \frac{E_1^2 + E_3^2}{8a^2} - 1 \quad \text{(3b)}$$

This method obviates scanning, but the wavelengths need to be predetermined, and the calculations are not as simple.

The program was elaborated to compute "measured" reflectivity from Ref. 2 using the envelopes for the real partial clapotis already set up in the computer. A comparison of these results proved difficult because of the various possible locations of the three probes (16 positions per wavelength were computed).

Most results showed a many-valued relationship between "actual" and "measured" reflectivity, with largest deviations at near clapotis, and especially for steepnesses in excess of about 0.01. Because the actual location of the three probes in the envelope will not generally be known, this method cannot be used to compute the actual reflectivity for such situations.

CONCLUSION

Experimentation frequently results in a compromise between small steepness parameters, to preserve linearity, and finite amplitudes, to facilitate observation.

To estimate the incident wave amplitude and steepness, Eq. 2 provides a reasonable first method. If the steepness is small enough to assume Airy theory, either of the two methods previously mentioned can be used to analyze the partial clapotis. If the steepness is significant, say greater than 0.01, with high reflectivity, say greater that 0.5, the three-point method gives an unreliable many-valued relationship between measured and actual reflectivity. The loop and node method should then be used, and the actual reflectivities read off a family of curves such as those in Fig. 1.

A paper by WILLIAM JAMES*, BSc(Eng), Dipl HydEng, PhD (Associate Member)
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NUMERICAL COMPUTATIONS FOR TIDAL PROPAGATION IN THE ST. LUCIA ESTUARY

SYNOPSIS

THIS paper describes a finite difference scheme for the solution of the equations of motion in partial differential form for the St. Lucia Estuary. The boundary conditions are tidal flow at the mouth and wind-set at the lake at the upstream end. Thus the basic input is tidal and wind data, as well as the physical hydrography of the estuary. It is shown that the unsteady water levels can be predicted to an accuracy better than 0.1 ft throughout the eight-mile estuary. The method was used to indicate the success of dredging operations, and can be used to predict the effect of any channel-widening or deepening before such operations are introduced. The method can also be built into a comprehensive numerical model for the entire St. Lucia Lake system.

Introduction

THE object of this study was the estimation of the net inflow of salt water into the lake system during each tidal cycle. Provided that the net precipitation, evapotransporation, and fresh water runoff into the lake has been recorded, the effect on lake salinity of the unsteady tidal propagation of seawater can then be assessed. An additional requirement is a knowledge of meteorological data, such as winds.

From the results of this work it is hoped to construct a numerical model of the entire system, such that an immediate decision could be taken to either disrupt the tidal flow, or augment the fresh water supply (assuming this to be possible) to the lake to maintain the salinity level within any limits established by ecologists. A further benefit of a numerical model is that the effect of any large-scale physical alterations to the estuary, e.g. through major dredging operations, can be examined in advance, and this will lead to maximum operational efficiency and reduction in expenditure.

The numerical scheme formed to solve simultaneously the partial differential equations of flow was one-dimensional in form, and consequently it was not feasible to account for detailed effects such as sediment transport, local obstructions caused by bridges, or salinity stratification. However, the scheme could indicate sections that require more detailed analysis and physical models of such areas could be constructed to a larger scale than hitherto.

The basic data that were necessary for the solution of these equations included friction parameters, windset, tidal range at the mouth and the physical dimensions of the estuary. In order to estimate the friction parameters, certain observations were used to optimise the results. In other words the numerical model was considered proved when it had reproduced observed tidal curves over a long time interval. Thereafter these friction parameters were used with a new schematization to suit the channel existing at that time, to predict the water fluctuations and mean velocities of flow at any point in the estuary and at any particular instant of time.

The equations of motion

The basic one-dimensional unsteady hydrodynamic equations of continuity and motion are:

$$\frac{\partial Q}{\partial x} + b\frac{\partial z}{\partial t} + q = 0 \qquad \ldots\ldots\ldots\ldots \quad (1)$$

$$\frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + g\frac{\partial z}{\partial x} + U\,|\,U\,|\,\Phi(h) - W(h) + \frac{gh}{2\rho}\left(\frac{\partial \rho}{\partial x}\right) + I = 0 \qquad \ldots\ldots \quad (2)$$

where b is the mean surface breadth
g the acceleration due to gravity
h the channel depth
I the bed slope term relating section datums
Q the discharge
q the lateral unit discharge
t is time
U is depth mean velocity in direction of x increasing
$W(h)$ the wind force term
x is distance along thalweg of estuary, measured from the mouth upstream
z the elevation of water surface above cross-section datum
ρ is depth mean water density
$\Phi(h)$ the frictional function.

The nine-mile section of the estuary used in the calculation, from the tidal recorder at the mouth to the recorder immediately beyond the confluence with the Impati River near where the estuary widens out in a delta to join the lake, is shown in Fig. 1. The average breadths over a tidal cycle at these points were 1,324 ft and 839 ft respectively, and the corresponding depths 5.78 ft and 1.34 ft respectively. The average spring range varied from 3.12 ft at the Shark Basin to 0.43 ft at the Impati. Flow from the Impati River, if any, was neglected and the channel was regarded as a closed branch of the estuary. (Water samples taken in the Impati half a mile above the confluence with the estuary were found to be of a similar salinity to that of seawater during the period of the study.)

Velocity profiles were recorded during a new moon, spring tidal cycle, by final year undergraduates in the Department of Civil Engineering at the University of Natal. These profiles revealed an abrupt change in the direction of flow after slack water, instead of the

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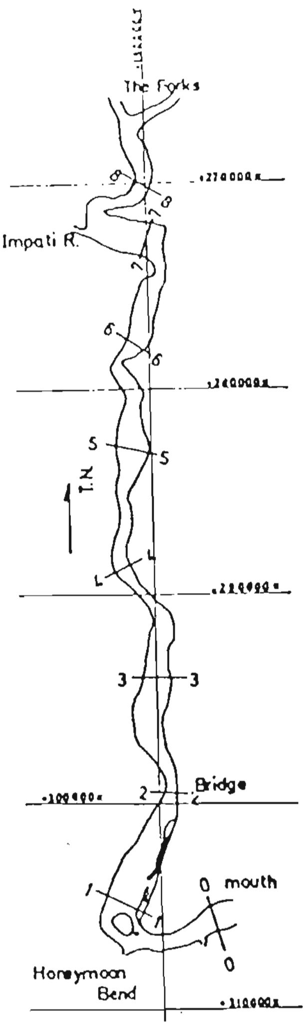

Fig. 1

usual exchange flow encountered in mixed estuaries, and so a constant fluid salinity and density was assumed; term 6 in equation (2) was ignored. Lateral fresh water discharge was therefore also considered to be negligible. The channel bed was assumed to be approximately horizontal and thus term 7 in equation (2) was neglected.

The general equations were thus simplified to:

$$\frac{\partial Q}{\partial x} + b\frac{\partial z}{\partial t} = 0 \quad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

$$\frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + g\frac{\partial z}{\partial x} + U\,|\,U\,|\,\Phi(h) = 0 \quad \ldots\ldots \quad (4)$$

Note the use of the modulus term in bed friction to ensure that the head loss always opposes motion. The finite difference form of these equations with reference to Fig. 2, is:

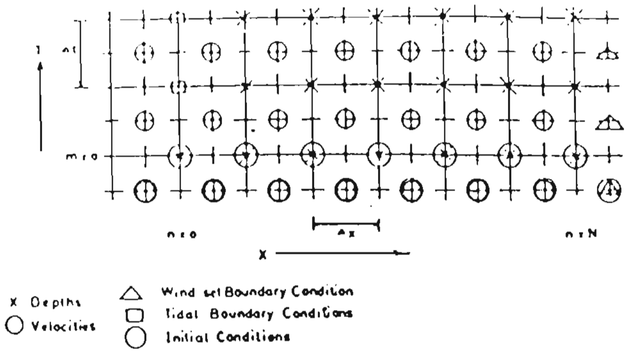

Fig. 2

where A is the cross-sectional area
Δt is the time interval between successive computations
m is an integer
Δx the distance between adjacent stations
n is an integer
Q, U, A and h are all evaluated at the centre of each section at time intervals of Δt
b was measured at integral sections at intervals of Δt

The usual order of computation of the difference equations was used,[1] and satisfied the grid-mesh shown in Fig. 2.

The estuary was divided into eight intervals, each 5,964 ft long to coincide with the existing tide gauges as shown in Fig. 1. The time increment was fixed at 450 seconds to satisfy the stability requirement (assuming $h = 6$ ft max.).

Stability

To ensure that the final difference solution converges towards the solution given by infinitesimally small values the grid size must satisfy:

$$\frac{\Delta x}{\Delta t} \geqslant \sqrt{\left(\frac{gA}{b}\right)} \geqslant \sqrt{\left(g\,h_{\max}\right)}$$

The latter term is the celerity of a long wave in the estuary.

Boundary conditions

The terminal boundary condition at the estuary mouth is the tidal range in water surface elevation. These values are interpolated from tidal records at the Shark Basin. The velocity at the mouth is obtained by extrapolation from the neighbouring sections. The terminal boundary condition at the Impati River is

$$z_{m,n} = z_{m-1,n} + \frac{\Delta t}{\Delta x}\,\frac{1}{b_{m-1/2,n}}\left\{(AU)_{n-1/2} - (AU)_{n+1/2}\right\}_{m-1/2} \quad \ldots\ldots\ldots\ldots \quad (5)$$

$$U_{m+1/2,n+1/2}\left[1 + \Delta t\left|U_{m-1/2,n+1/2}\right|\Phi\left(h_{m,n+1/2}\right) - \frac{\Delta t}{4\Delta x}\left(U_{n-1/2} - U_{n+3/2}\right)_{m-1/2}\right]$$

$$= U_{m-1/2,n+1/2} + g\frac{\Delta t}{\Delta x}\left(z_n - z_{n+1}\right)_m + \frac{\Delta t}{8\Delta x}\left(U^2_{n-1/2} - U^2_{n+3/2}\right)_{m-1/2} \quad \ldots\ldots\ldots \quad (6)$$

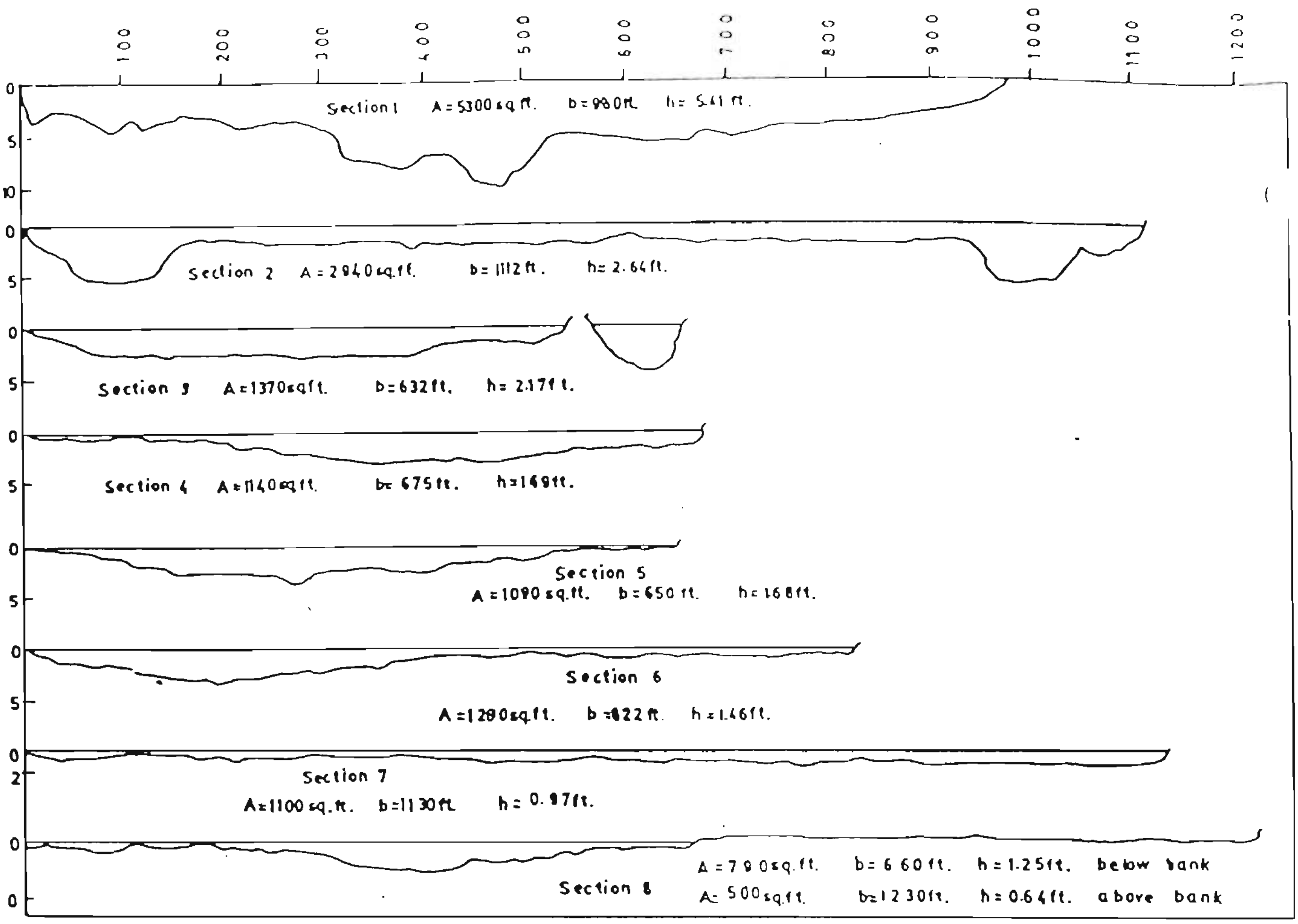

Fig. 3

that of flow to or from the lake depending on the water level at Charter's Creek.

Equation (5) can be modified for the final section in order to calculate z:

$$z_{m,N} = z_{m-1,N} + \frac{\Delta t}{\Delta x}\frac{1}{b_{m-1/2,N}}\left\{(AU)_{m-1/2,N-1/2} - Q\right\} \quad \ldots \quad (7)$$

where N is defined in Fig. 2.

Wind set arises whenever wind blows upon a free water surface, and the effect of these tangential stresses is to generate a surface current in the direction of wind, which is counterbalanced by a return flow at depth. In shallow water this return flow is retarded by bottom friction and the water sets up to leeward until the pressure head is sufficient to restore equilibrium.

Hence wind set is a function of water depth, wind speed and boundary roughness, its magnitude in shallow water being satisfied by the formula[2]:

$$\Delta h = \frac{V^2 L}{1400\, d}\cos\alpha$$

where V is the wind velocity in m.p.h.
L is the fetch in miles
d is the depth in feet
α the angle between the wind and the tidal axis.

The wind set at the estuary inlet for the study period was calculated from the anemographs for Charter's Creek, in the southern section of the lake.

The variation in mean water level at the Impati gauge was derived from both the computed and the observed curves by a simple system of harmonic filtering. In a Fourier series fitted to the lake levels at Charter's Creek, the components with frequencies that are harmonics of the frequencies occurring in astronomical tides are eliminated systematically.

The difference between the filtered computed mean and the filtered observed mean was the *wind tide*.

Roughness

Cross-sectional areas and free surface breadths for each increment were obtained from cross-sections interpolated every 1,000 ft from hydrographic plans compiled by the Natal Provincial Water Engineer. In regions where deep channels were flanked by shallows, the cross-sectional area was adjusted to allow for the lower velocities on the banks:

$$A = A_1 + A_2\sqrt{\left(\frac{h_2}{h_1}\right)}$$

where subscript 2 refers to the shallow regions and 1 to the main channel. The sections are illustrated in Fig. 3. Two schematizations were necessary in Section 8 as a large mudbank was exposed by the receding tide. The cross-sections beyond the Impati were weighted by a small factor proportional to the equidistant cross-sectional areas to account for tidal flow diverted into the Impati River.

It is clear that the flow regime is dominated by friction because the estuary comprises a very wide and shallow channel.

The following form[3] of the Manning friction formula was found to be suitable:

$$\Phi(h) = \frac{g\, n_m^2}{1.812\,(h_{m,\,n+1/2})^{3/2}}$$

where n_m is the Manning friction factor.

Optimization by trial and error of the friction parameters proved to be laborious as any alteration to the n_m appropriate to the particular channel section affected all other increments upstream. Consequently n_m was determined successively in each incremental section proceeding from the mouth, such that the mean square error (the deviation of the computed curves from the corresponding observed curves) was a minimum. Table I contains the final values of the Manning's n_m for each section.

Results

The basic difficulty encountered in explicit unsteady calculations is the necessity to work close to the limits of accuracy of the tide gauge observations. The time origin was chosen to coincide with slack water at the mouth, when the initial mid-section velocities were fairly small. A twelve-hour run-in period was used to overcome the effects of erroneous initial data, and it was found that such data had little effect as the calculation proceeded. The velocity at the lake was originally assumed to be zero, but from an extrapolation of computed and observed results this was amended to a value of 0.322 of the mean flow in the preceding section. When the lake level was above that of mean sea level the net outflow was calculated from a modified Manning equation assuming uniform flow conditions in each section:

$$U = \frac{1.346}{n_m} R^{0.75}\, i^{0.5} \text{ ft/sec.} \qquad \ldots\ldots\ldots \quad (8)$$

where R is hydraulic radius

i is the energy gradient.

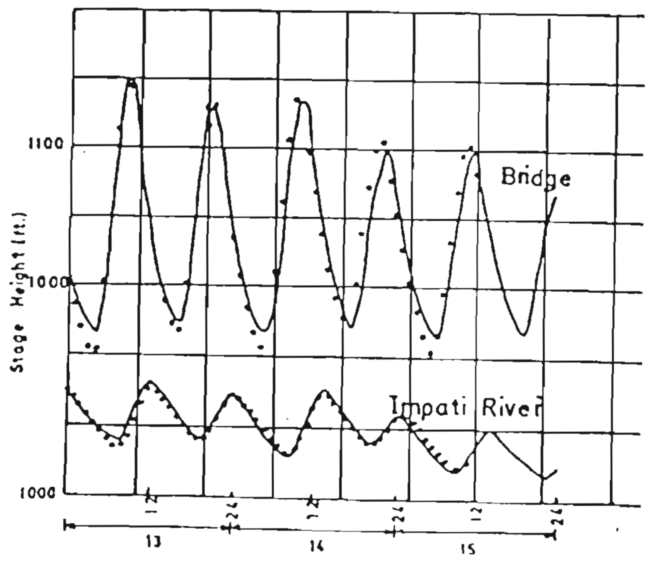

Fig. 4

Typical results are shown in Fig. 4 where the full line represents the observed curve and the individual points, computed values. A maximum difference of 0.1 ft between computed values and observations occurs at the Impati tide gauge, and this is approximately within the tolerances of the sum of the observation errors. Calculations for the scheme after dredging activities resulted in the new friction parameters listed in Table I. These reveal a significant improvement.

The maximum tidal discharge into the lake was found to be of the order of 300 cusecs.

TABLE I

Manning's n_m

Year	Sect. 1	Sect. 2	Sect. 3	Sect. 4	Sect. 5	Sect. 6	Sect. 7	Sect. 8	Sect. 9
1961	.085	.030	.020	.019	.024	.026	.038	.055	.050
1965	.044	.018							

Conclusions

The flow regime was not influenced by density currents during the period of study and the lake level was maintained either by run-off or inflow from the sea. As the latter was unlikely to exceed 300 cusecs, an appreciable drop in lake level could be expected during a prolonged drought. The lake salinity was thus dependent primarily on meteorological conditions at the time of the study.

The tidal wave propagated up the estuary was usually completely damped before it reached the lake. Hence any tides recorded at Charter's Creek were probably generated either by the wind or by astronomical forces.

Damping of the exchange flow seemed to be higher in those sections containing bends. This was aggravated by the large obstruction such as Honeymoon Bend Island in Section 1 and the mud bank in Section 8. The increased hydraulic efficiency in 1965 of the section between the mouth and the bridge was caused by dredging operations.

The accuracy of reproduction of the physical conditions is sufficient for this method to be used to analyse the probable response of the estuary to natural and artificial changes. It is further submitted that a mathematical interpretation of this nature allows a more rigorous explanation of the factors involved in the tidal regime than a physical model.

Acknowledgments

The Authors wish to record their deep appreciation of the kind help and encouragement afforded by the then Provincial Water Engineer, the late Mr. E. A. Middleton.

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CHAPTER 132

SPECTRAL RESPONSE OF HARBOR RESONATOR CONFIGURATIONS

William James*

ABSTRACT

An outline is given of methods devised recently by the author to predict the spectral response of rectangular resonators, and to improve the response of resonators generally. A simple small scale acoustic model of the ocean-resonator-harbor configuration was developed and is described. The acoustic "ocean" is effectively decoupled from the rest of the system by means of sound absorbent material placed along the "infinite" boundaries, and standard audio-frequency equipment is used. The results demonstrate that the open end contraction for rectangular resonators may not differ significantly from the contraction for resonators of similar geometry placed in a semi-infinite (or effectively decoupled) wave channel, at least if the wavelengths are not smaller than the width of the harbor entrance channel.

INTRODUCTION

It is desirable to design harbor entrances to eliminate (as far as possible) those bandwidths in the incident wave spectra that cause difficulties such as range action, high mooring forces, unreasonable wave impact, wave overtopping and drift of littoral sediments into the harbor. In many problems these difficulties are functionally related to approximately the second (or higher) power of the wave height, and hence any reduction in the incident wave height will produce real benefits.

Resonators placed at the harbor entrance can be tuned to radiate back into the ocean those frequency bandwidths considered to be harmful without hindering navigation[1]. Readers are cautioned against using the blanket quarter-wavelength recommendation[2,3,4]; the initial design should accord with the fact that the tuning of individual resonators is considerably dependent on the width of the harbor entrance channel[5,6]. Readers should also note that resonators are not effective

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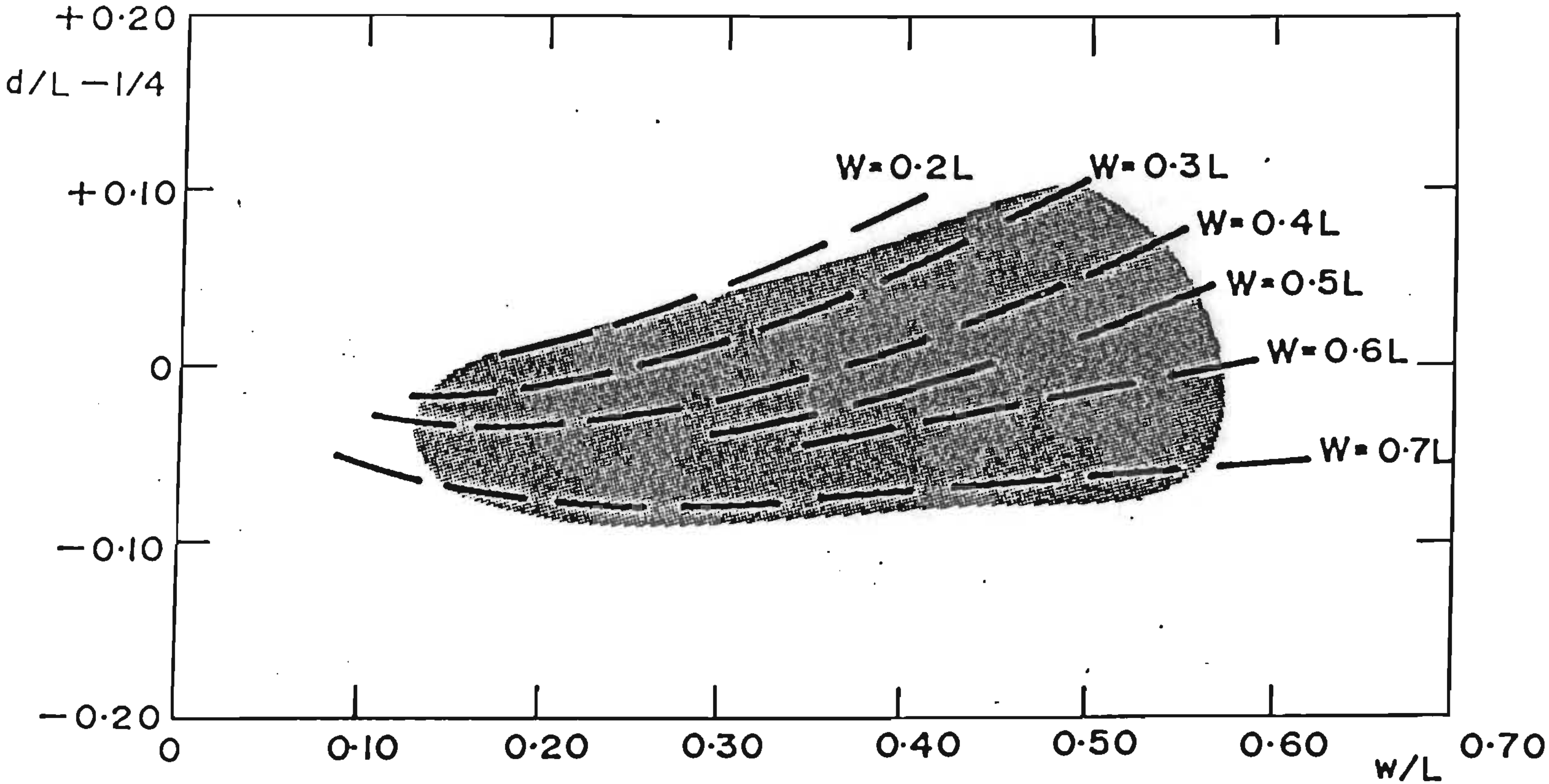

FIG. I GEOMETRY FOR FIRST RESONANT MODE

TR2

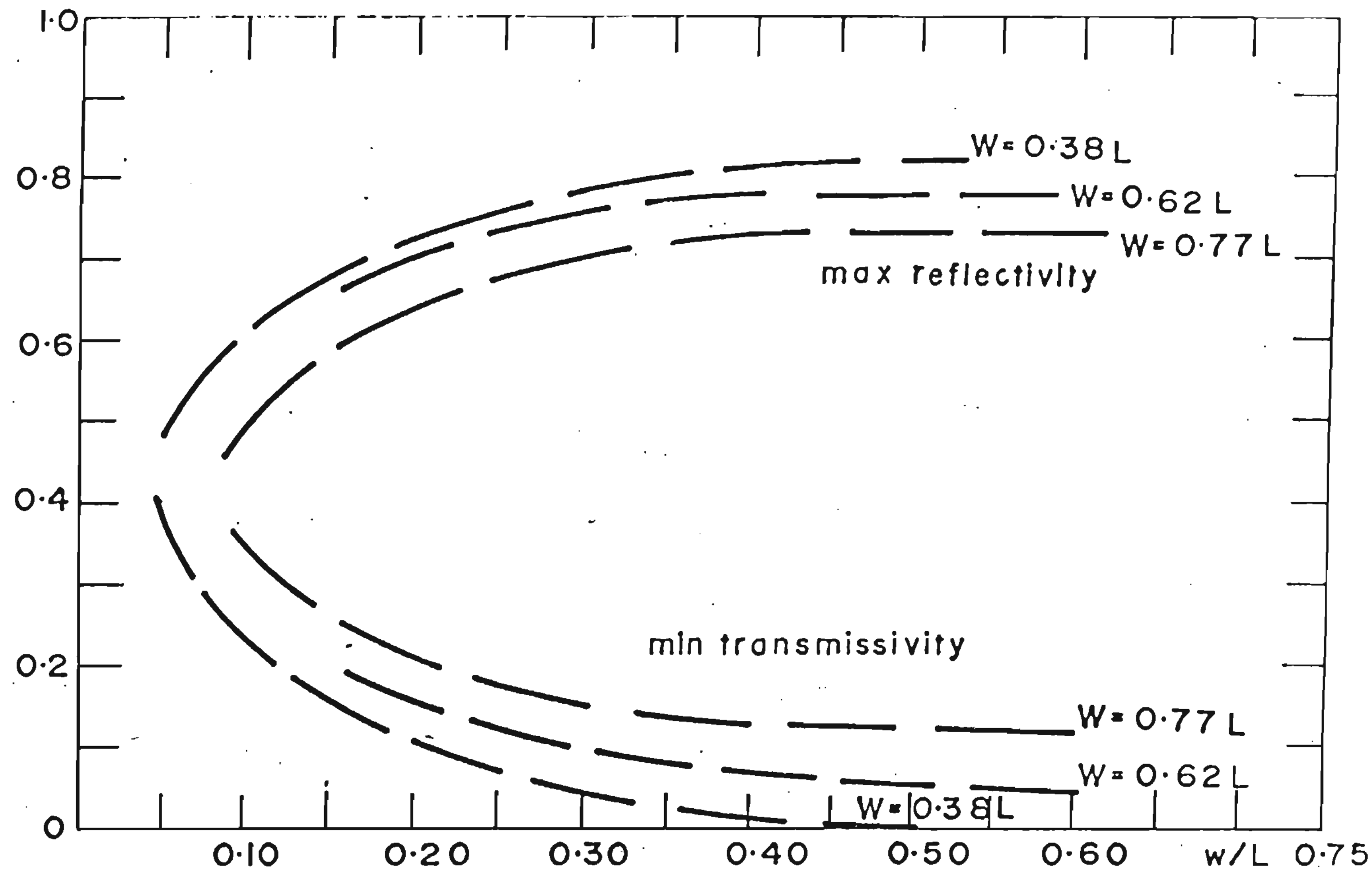

FIG. 2 RESONANT MAXIMA AND MINIMA

for wavelengths smaller than the width of the harbor entrance channel. Hence this paper relates to incident wavelengths that exceed the horizontal distance between the leading edges of symmetrically opposed rectangular resonators.

The transmitted and reflected spectra can be predicted for given geometry and incident spectra, assuming a linear system[7], and this procedure is outlined below. Certain innovations[8] that both broaden the tuning of the resonators and decrease the cost of construction are also briefly described in this paper. An acoustic model has been devised[6] and this is used to check the ocean-resonator-harbor coupling. The results of the latter tests constitute the major contribution of this paper.

SPECTRAL RESPONSE

The method devised for computing the spectral response of individual rectangular resonators is based on experimental results. It is usual to plot frequency response curves in the frequency domain[9] but in this study observed transmissivity and reflectivity were plotted against the tuning parameter d/L (i.e., in the ka domain). For details of the wave measurement and wave analysis procedure, readers are referred to earlier publications[10,11]. The geometrical conditions for distinct resonance are summarised in fig. 1.

Approximate rectilinear apexes were fitted into the ka domain resonance curves, and the resulting maxima and minima are summarised in fig. 2. The tuning parameter bandwidths at the half resonant values were measured and found to be reasonably constant for various values of W/L. The information is summarized in fig. 3. Sufficient data are available in these three figures to allow the computation of the spectral response of any rectangular resonator to given incident spectra, *but only for the first resonant mode.*

Briefly, the procedure is to transform the incident spectrum into an amplitude/wavelength relationship for the particular harbor entrance. The dimensions of the resonator are chosen such that distinct resonance obtains for the dominant wavelength, using fig. 1. For discrete wavelengths the ratios W/L, w/L and d/L are calculated. The resonant tuning parameter is obtained from fig. 1, and the resonance curves "reconstructed" graphically or digitally using figs. 2 and 3. Values of transmissivity and reflectivity are then interpolated for various tuning parameters.

Finally, the incident spectra are transformed using these values. Two examples are presented elsewhere[7], and it is shown that a standard quarter-wavelength resonator may reflect *only one quarter of the peak energy,* nearly all of which is reflected by a resonator designed according to fig. 1.

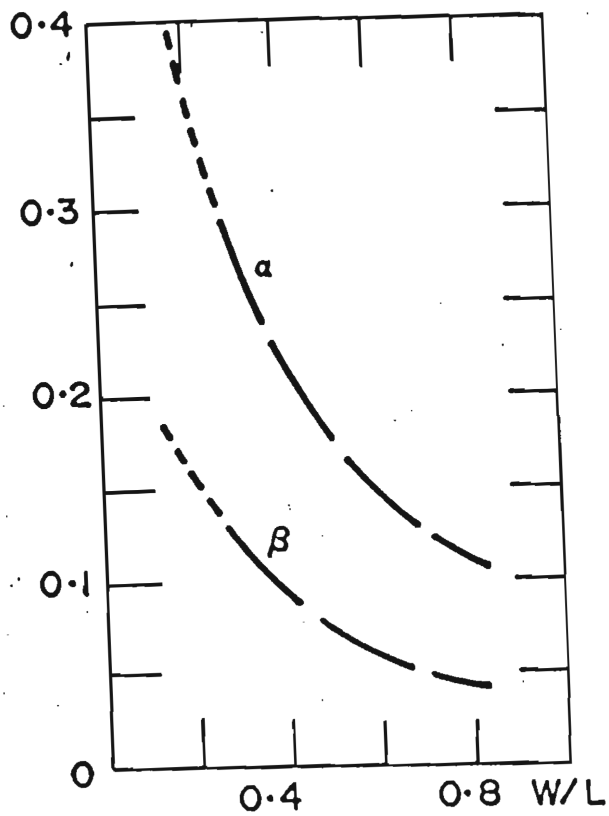

FIG. 3 TUNING PARAMETER BANDWIDTH AT HALF RESONANT VALUE

TWO INNOVATIONS

Experiments were performed on:

(a) variable distances between three resonators in a battery configuration, and

(b) non-uniform depths in a resonator/harbor entrance area.

Triple Resonator Batteries.

Rayleigh[12] recommended that the ratio of the fundamental periods of the individual resonators in a battery configuration should be $1:\sqrt{2}$. In this study resonators were constructed from 3/4 inch thick "perspex"; geometries are shown in fig. 4.

Reflectivity and transmissivity were measured at points approximately one wavelength upstream and downstream of the battery respectively. In these tests the water depth was held constant, and wavelengths were systematically varied.

The results showed resonant peaks at those individual resonator frequencies obtained in earlier tests on single resonators, as is to be expected. Hence fig. 1 can be used to predict the response of battery configurations: superimpose the equation (a straight line) for each battery geometry (e.g. $w/L = 1.61\ d/L$) on fig. 1 and scan the line for the location such that the harbor entrance and resonator geometry satisfy the geometrical conditions for resonance. This identifies the tuning parameter for resonance, and performance can be estimated using the same computational procedure for spectral response described above.

By increasing the distances between the resonators, the water mass in the entrance channel contiguous with the leading edges of two adjacent resonators is brought into the system response. This effectively broadens the overall tuning of the battery configuration and hence improves the spectral performance of the battery. Because of the end-contractions, the distance should be significant, e.g. commensurate with resonator dimensions, and chosen by careful *ad hoc* model tests (acoustic models preferably).

Non-Uniform Depths.

In the experiments on non-uniform depths an artificial invert was inserted into a single resonator, as depicted in fig. 5. The still water depth and wavelength were held constant and the resonator planform was varied systematically (by gradually retracting the rear wall outwards). Reflectivity and transmissivity were measured as in early tests[1].

The results indicated a reduced efficiency for resonators of comparatively small still water depth, but less upstream agitation

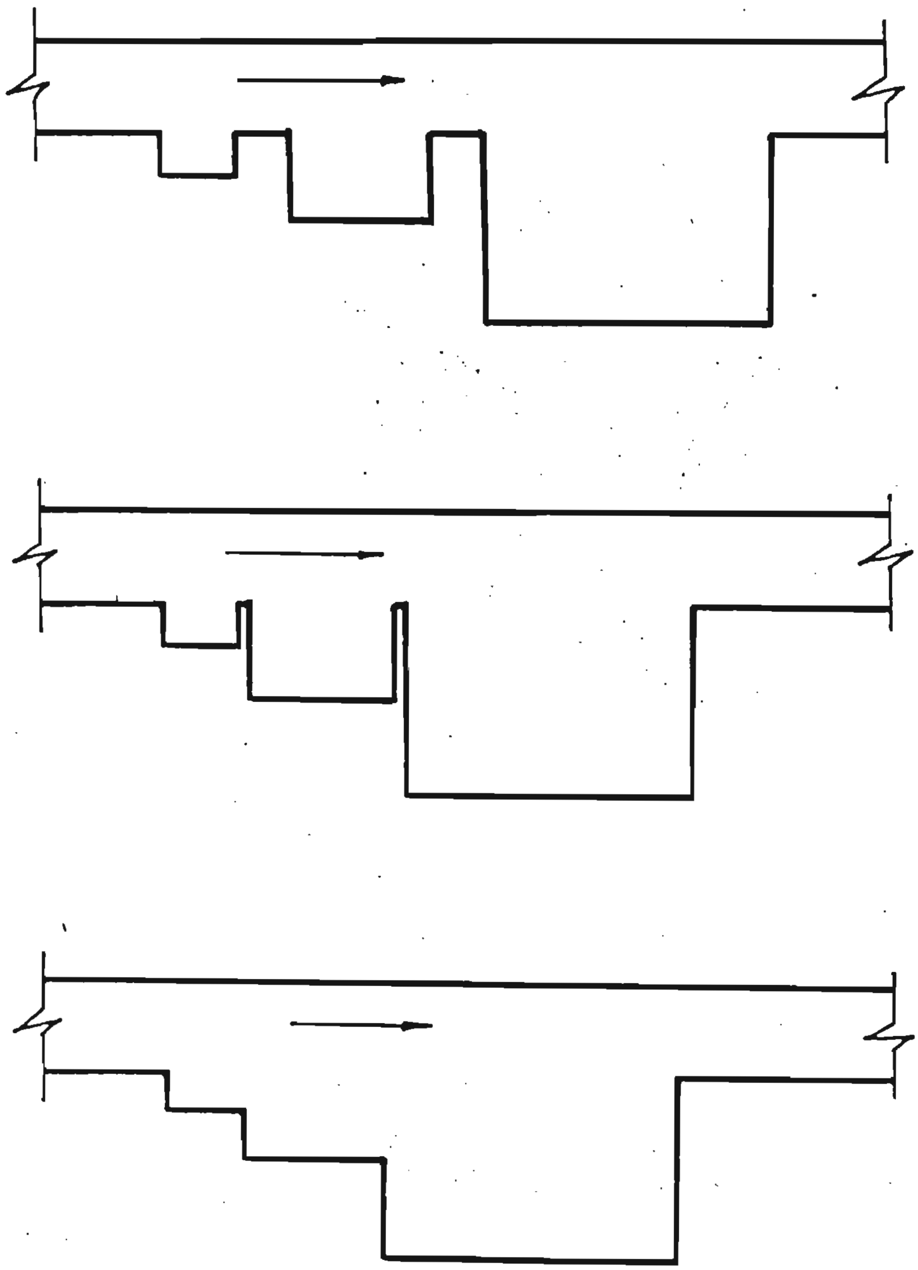

FIG. 4 BATTERY GEOMETRIES EXAMINED

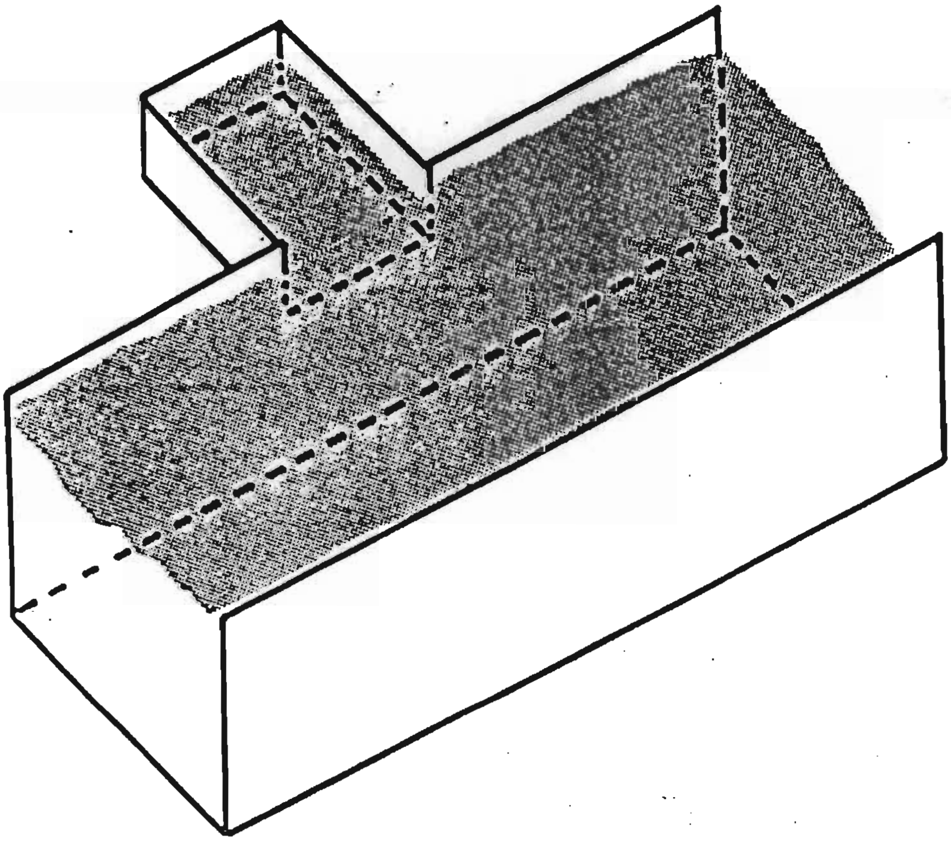

FIG. 5 NON—UNIFORM DEPTHS

(reflectivity) was associated with given transmissivity.

It was found that the length of the resonator could easily be halved, if the still water depth in the resonator is reduced to a fraction of the harbor entrance channel depth, a point of considerable practical significance. However, tidal ranges will detune the resonator, the effect being related to the proportional reduction in depth and to the actual tidal range.

THE ACOUSTIC MODEL

An acoustic model evidently has certain advantages over a hydraulic model; wave generation and absorption are easier, the fluid does not have to be isolated, wavelengths are generally shorter, and measuring equipment, speed and accuracy are generally better.

To test the model, earlier experiments on resonators in a water wave channel were reproduced. The variable geometry was built up from 3 inch movable timber walls and placed on a glass plate on top of a desk. A second glass plate was placed on top of the walls, and a loudspeaker was connected to an oscillator and placed against sponge rubber at the entry to the main duct. A microphone was placed against the sponge rubber absorber at the harbor end of the main duct, and was connected through a small pre-amplifier to an oscilloscope. By setting the widths of the main entrance duct and of the resonator, and by holding the oscillator frequency constant, the length of the resonator was adjusted until resonance occurred. This was monitored by a minimum signal on the oscilloscope.

The effect of imperfect sound absorption was checked and found to be unimportant. Tests on scale effects were also negative, although these were not exhaustive. In addition the effects of sound waves entering the duct along its length were also found to be insignificant. The signal-to-noise ratio was easily improved by means of the amplifier on the oscilloscope.

The results showed slight differences from those of the water wave experiments. Reasonably accurate predictions of harbor resonance modes for frequencies can be obtained in this way, but further development of the method will be necessary if absolute values of harbor amplification are required.

OCEAN-RESONATOR-HARBOR COUPLING

Fig. 6 shows the experimental apparatus used to examine the coupling problem. In this case the model was set up on a reasonably clean office floor, again using 3 inch high timber walls. The cover to the ocean domain was supported on 3 inch iron nails. Cotton batting (reflectivity about 5%) was arranged along the "infinite"

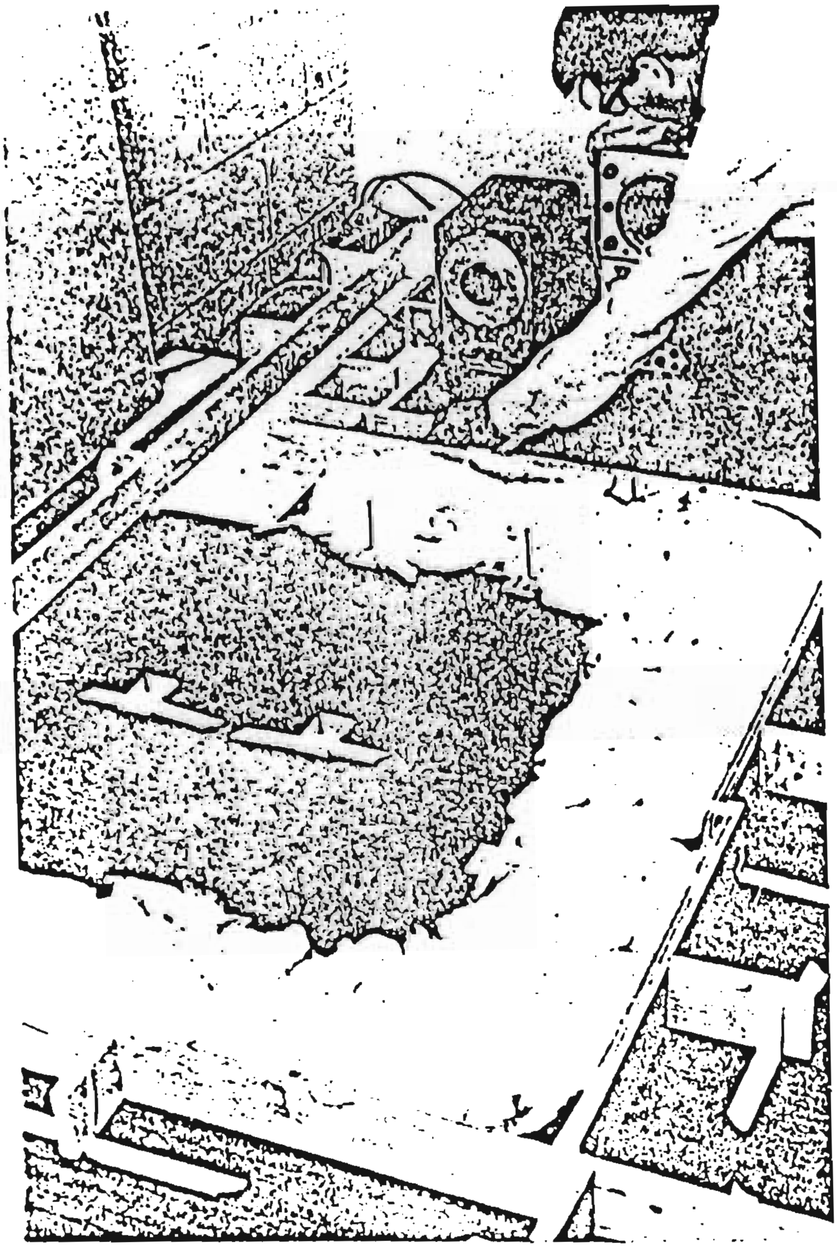

FIG. 6 ACOUSTIC MODEL: OCEAN-RESONATOR-HARBOR

boundaries, and this effectively decoupled the ocean domain. Total cost of the model excluding audio equipment was $8.50.

Acoustic waves were generated from a virtual point source, and so the ocean domain had to be long enough for the radiating incident waves to achieve an approximately parallel wave front at the harbor entrance. It follows that re-reflections from the harbor off the loudspeaker were negligible.

The experimental procedure adopted was similar to that above: resonance monitored by a minimum acoustic pressure signal in the harbor, and achieved by systematic variation of resonator geometry. This method was better than incident wave frequency variation, since system attenuation, transmissivity and reflectivity, parasitic vibrations, and also wave generation and recording were frequency dependent.

Geometrical conditions obtained for distinct resonance are presented in fig. 7. Results for semi-infinite ocean coupling and semi-infinite wave channel coupling almost coincided; consequently both results could not be plotted on the figure. However the ocean coupling curves were slightly flatter, as indicated by the dashed curve.

Evidently, then, harbor resonance studies may be carried out at the end of water wave channels (provided that the incident wavelength exceeds the width of the harbor entrance channel) without material loss of accuracy.

Further tests on the acoustic model confirmed that resonators should be located at the ocean end of the entrance channel, and at an amplitude antinode in the wave envelope, if the harbor is reflective at the resonator tuning frequency. Since partial re-reflections occur off the ocean end of the entrance channel, the downstream harbor and channel oscillations are generally not amplified by resonators. For *transparent* frequencies there is no effect and for *opaque* frequencies there is no penetration.

CONCLUSIONS

Three figures are presented for estimating the spectral response of individual rectangular resonators, *but only for the first resonant mode*. No account is taken of the second resonant mode even though this is important when considering the response of a battery of resonators.

Individual resonators in a battery respond in an additive manner, and hence this response can also be predicted. The mass of water contiguous with adjacent resonators in the harbor entrance channel can

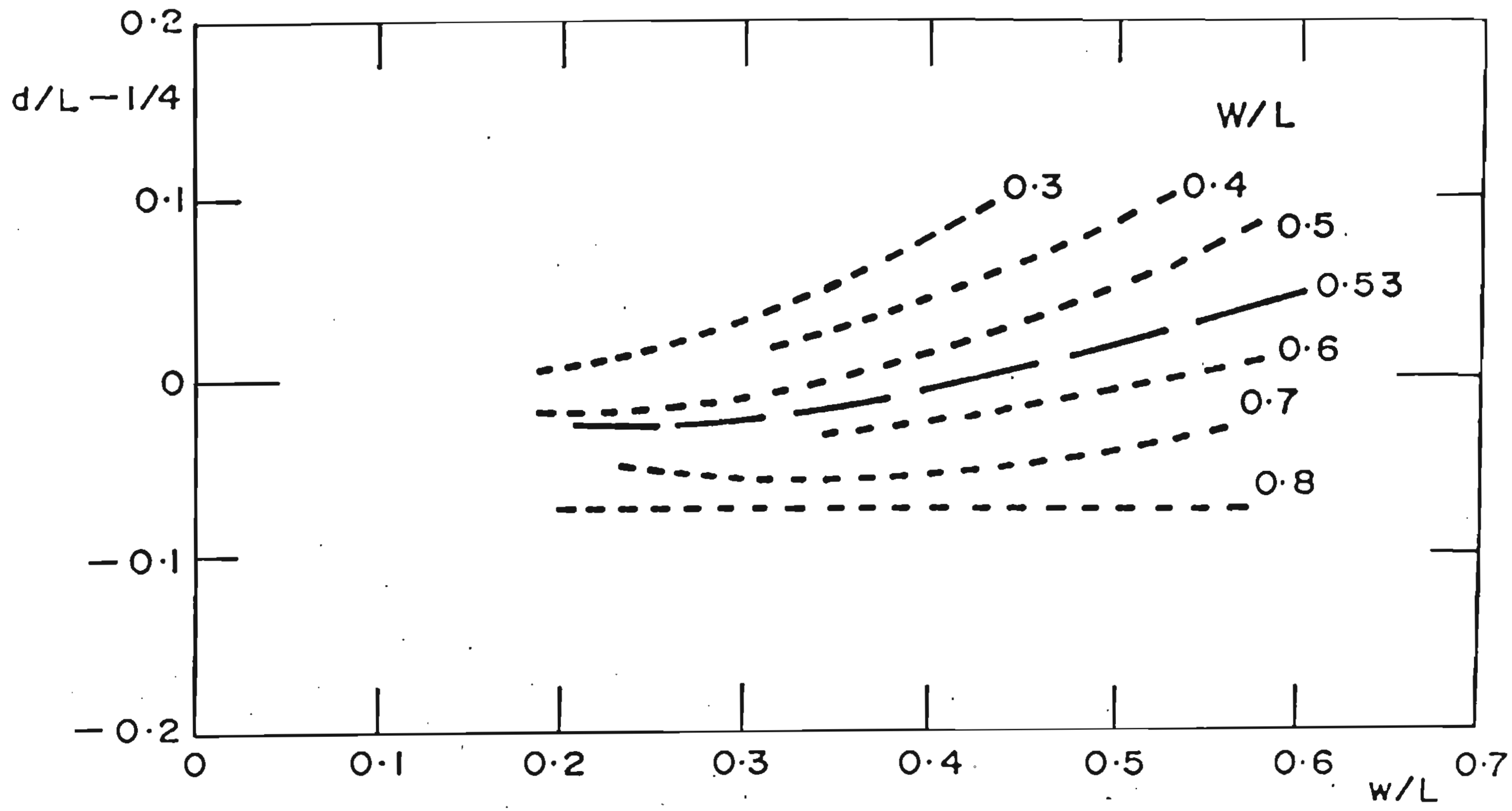

FIG. 7 OCEAN—RESONATOR—HARBOR COUPLING

also be incorporated in the system response. For this reason the resonators should not be contiguous, if at all possible.

Comparatively shallow still water depths in individual resonators detunes the resonator downwards, i.e. towards smaller frequencies. Hence, for a particular tuning, non-uniform depths, with smaller depths in the resonator, result in smaller geometry, and concommitant savings in construction and excavation costs. Large tidal amplitudes would effectively detune such systems, however.

The acoustic analogy is an extremely fast and cheap method for evaluating eigen frequencies for any harbor planform. The method could probably be elaborated for estimation of orbital velocities and even of mooring forces, for uniform water depths in the harbor and harbor entrance. No scale effects were detected in the tests reported.

An acoustic model was constructed to check the ocean-resonator-harbor coupling. *The results indicated that the end-effect does not differ significantly from the contraction for resonators of similar geometry placed in a semi-infinite wave channel, at least for wavelengths greater than the harbor entrance width.*

This result validates the experimental results obtained in the wave channel; the method for predicting the spectral response of resonators for a real situation (i.e. on the edge of a semi-infinite ocean) is evidently reliable to the first order.

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NOTATION

a = lineal dimension
d = length of resonator
k = wave number
L = incident wavelength
w = width of resonator
W = width of harbor entrance channel
α = reflectivity
β = transmissivity

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CHAPTER 98

RECTANGULAR RESONATORS FOR HARBOUR ENTRANCES

by

William James*

ABSTRACT

A complete theoretical analysis of the behaviour of resonators built into the breakwaters of a harbour entrance would be extremely difficult. This paper describes an approximate development for extreme values of resonator geometry and shows how a combination of these cases explains certain observed phenomena for intermediate shapes.

The paper also describes comprehensive laboratory tests on resonators and draws attention to certain significant discrepancies from present design practice. In particular, the paper indicates the extent to which optimum resonator geometry depends upon the harbour entrance width and demonstrates that such resonators can be designed to prevent any penetration of waves into harbours, without restricting shipping.

INTRODUCTION

A resonator is taken to be a short rectangular branch canal, completely closed on three sides, built orthogonally on to the entrance breakwaters of a harbour (see Fig.1). Only the action of such resonators in parallel straight breakwaters has been investigated and this geometry is identical to that studied by Valembois in 1953 (ref.1). In this present paper the effect of distributed reflections from the harbour is not considered; this study pertains only to the geometry of such resonators and their action on waves entering harbours.

The dimensions of the resonator should be optimized such that a mass of water contained within the resonator and that part of the entrance channel contiguous with the resonator mouth (here termed the "junction element" as shown in Fig.1), has a resonance frequency equal to that of the harmful incident wave that should be totally reflected back into the ocean. For the purposes of this paper the lowest frequency eigenvalue is used; for batteries of resonators tuned to cover a wide frequency band, resonance will also occur at higher harmonics.

Limiting the treatment to semi-infinite oceans and harbours results in simpler mathematics but introduces complications in the experiments, since the wave generator channel and harbour must be completely decoupled. Further simplifications in the mathematical treatment result from the use of first order linear wave theory and the assumption of irrotational motion. Such development will accord closely with the tsunami-type waves likely to excite range action in real harbours but experimental waves are necessarily Stokian and, for accuracy, non-linear effects must be accounted for. Consequently, in this study, wave measurement was effected by micrometers and corrections were applied to render the readings correct to the nearest 0.005 inch.

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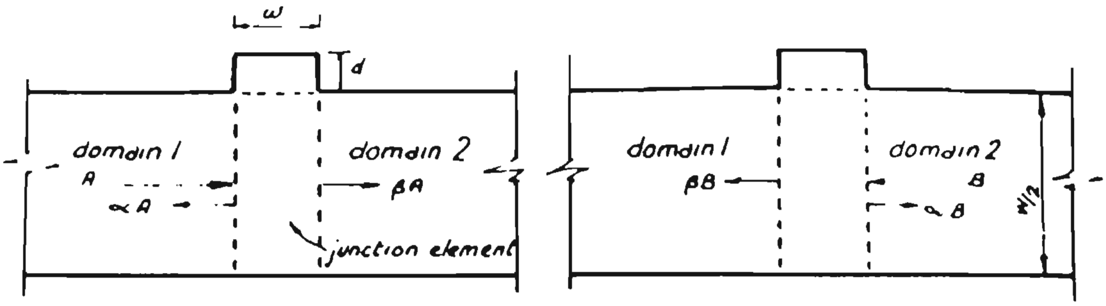

Fig.1 Wave components

MATHEMATICAL TREATMENT

Evidently the zone of protection provided by a resonator in the downstream direction, at resonance, is limited to a width of one half the incident wavelength (ref.2). Hence wave energy transmitted into the downstream domain will increase directly with increase in channel width above one wavelength, for the case where resonators are placed on both sides of the entrance channel. It follows that a resonator investigation of this type may be limited to one-dimensional propagation. Note that, though the assumption of plane waves is invalid near the junction element, two-dimensional motion becomes one-dimensional at distances of the order of twice the depth from the discontinuity, for these channel widths. Moreover, researchers have found that this limitation is not at all serious, provided that no transverse resonance occurs (ref.3). Transverse resonance is likely to occur when the basin width is close to an integral number of half-wavelengths.

The resonator geometry may be intermediate in shape between extremes that are either long and narrow, when the motion is in a direction normal to that of the waves in the entrance channel, or short and wide, when the motion is parallel to that of the entrance channel. These cases are denoted mode I and mode II respectively (see Fig.2). It is assumed that the complicated motion for intermediate geometries can be approximated by superposition of these unidirectional components. Each mode is considered in turn and then a combination is shown to accord with observations for an intermediate case. The effect of turbulent dissipation is also discussed.

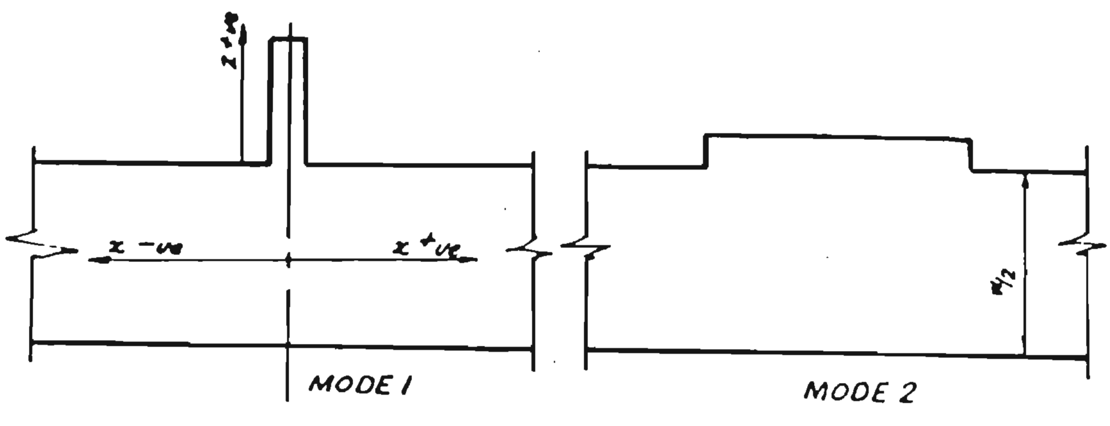

Fig. 2 Extreme cases.

Mode I

Notation is defined at the end of this paper.

For mode I,

$$wW << \lambda^2$$

Hence: (a) the water surface of the junction element remains approximately horizontal and (b) the origins for the resonators and the channel may be assumed to coincide.

Using complex representation, the reflected wave is written:

$$\eta = \bar{\alpha}\, a e^{j(kx+\sigma t)}$$

The water surface for the original incident wave, the reflected wave, the partial clapotis in the resonators and the transmitted wave may be matched for each contiguity:

$$\bar{i} + \bar{\alpha} - \bar{\beta} = (1+\bar{R})\bar{\theta} \tag{1}$$

The equation of continuity for the junction element is:

$$\sum_{1,2,R} \left\{ \int_{-h}^{0} \left(-\frac{\delta\phi}{\delta x}\right)_{x=0} dy \right\} = wW \frac{\delta\eta}{\delta t}$$

or:

$$kwW\bar{\beta} = -j\left\{ W(\bar{i}-\bar{\alpha}-\bar{\beta}) - 2w\bar{\theta}(1-\bar{R}) \right\} \tag{2}$$

for equal depths throughout.

Equations (1) and (2) can be solved for $|\alpha|, |\beta|, |\theta|, \hat{\alpha}, \hat{\beta}, \hat{\theta}$ in terms of $|R|$, $\hat{R}$, and, in this study, an Elliot 803 computer was used to tabulate the results. All results were checked against the expression for the conservation of energy developed below.

Mode II

In this case the $\bar{\theta}$ and $\bar{\theta}\bar{R}$ waves are parallel to the main channel. The waves are matched for discharge and surface continuity again at each channel discontinuity as follows:

$$\bar{i} + \bar{\alpha} = \bar{\theta}(1+\bar{R})$$
$$\bar{\beta} = \bar{\theta}(1 + \bar{R}e^{2jkw})$$
$$W(\bar{i}-\bar{\alpha}) = \bar{\theta}(1-\bar{R})(W+2d)$$
$$W\beta = \bar{\theta}(W+2d)(1-\bar{R}e^{2jkw})$$

These equations were solved, checked against energy conservation and tabulated using an I.B.M. 1620 computer.

Energy conservation

The energy of the reflected and transmitted waves is equal to that of the original incident wave less the energy dissipated in the resonators and junction element.

Proceeding from $E = \frac{1}{2}\rho \iint g\eta^2 dx\,dz + \frac{1}{2}\rho \iint \left(\phi\frac{\delta\phi}{\delta y}\right)_{y=0} dx\,dz$ (3)

we get, for no dissipation, $|\alpha|^2 + |\beta|^2 = 1$

For tsunamis and the experimental waves, viscous attenuation is negligible in the resonators. However, turbulence at the upstream edge is considerable and a lumped head loss can be postulated at this point:

$$H_L = a(|\theta| - |\theta||R|) = K\frac{u|u|}{2g}$$

Equation (3) then leads to:

$$|\alpha|^2 + |\beta|^2 + |\theta|^2(1-|R|^2)\frac{2w}{W} = 1$$

This allows H_L to be computed from observations of $|\alpha|$, $|\beta|$ and $|\theta|$.

U should include allowances for:

(a) local high velocities at the upstream re-entrant,
(b) the overall flow pattern in the junction element,
(c) the vertical velocity distribution.

The problem can be simplified by considering a steady-state flow pattern applicable to resonant conditions only and ignoring the local effects of surface deformations. Thus the transverse velocity profile along an equipotential emanating from the upstream edge of the resonator provides a good basis. Extreme velocities close to the edge can be reduced to manageable proportions by ignoring velocities within a nominal boundary layer and the vertical velocity distribution can be assumed to be the same as that given by Airy theory under a node in a standing wave. Using Schwarzian transforms for the upper half-plane the co-ordinates for given incremental values of ψ can be computed along the equipotential starting from the sharp edge. The velocity $\bar{q}$ is given by these differences and K can be calculated from

$$\int_{-h}^{0}\int_{0}^{l}\bar{q}|\bar{q}|\,ds\,dy = \frac{1}{K}\,2ga(|\theta| - |\theta||R|)$$

Intermediate geometry.

Any general combination of the type

$$(\alpha,\beta)_{total} = (\alpha\,,\beta)_{I}f_1(w,d) + (\alpha\,,\beta)_{II}f_2(w,d)$$

where the subscripts refer to the results obtained from computations for modes I and II and $f_1(w,d)$ denotes a simple function of the type $d/(w+d)$ and $w/(w+d)$ respectively, would reveal that:

(a) $\beta_{minimum} \neq 0$ and $\alpha_{max} \neq 1$ generally.
(b) for large w, there is an increasing tendency for the resonant value of d/λ to exceed a value of 0.25.
(c) for large w, there is a second type of resonance.
(d) resonant values of d/λ depend upon both w and W.

PRELIMINARY VERIFICATION.

A suitable experimental programme could not be devised without preliminary pilot studies. The results of these tests are now described and refer to an isolated single resonator.

Apparatus. The narrow "perspex" flume and timber housing for opposed adjustable rectangular resonators is shown in fig.3.

The rear wall of each resonator cantilevered vertically downwards from a system of clamps, which traversed along both side walls for the full length of the resonator. Hence any variation of resonator width, which was effected by means of a false (upstream) side wall, necessitated a change in width of the rear wall. This was achieved by means of twin thin overlapping galvanised iron sheet lips. An intermediate perspex wall was used to reduce the width of the entrance channel. For such tests filters were arranged at the upstream sharp edge of the intermediate wall and on the seaward side an efficient wave absorber was built.

Where the theoretical domain is semi-infinite, it is axiomatic that wave re-flections off the paddle are NOT allowed and wave filters were used for such decoupling devices. These filters were not effective at low wave steepnesses. Nevertheless, results were frequently obtained under stop/start conditions, i.e. before the initial waves would re-reflect and hence were of some value.

The desideratum that domain 2 be semi-infinite caused difficulty, because of insufficient space (in which to absorb all the energy of the transmitted wave.) Of necessity the writer adopted an absorber similar to that developed by Hamill (ref.4).

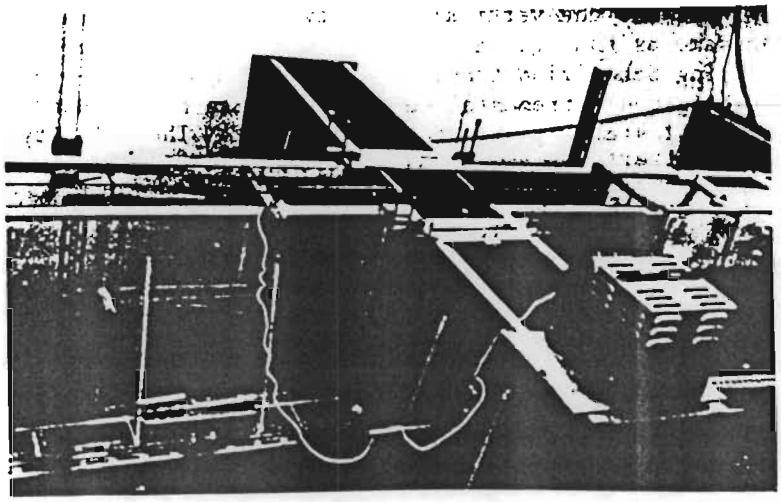

Fig.3 Preliminary apparatus.

Procedure. Envelope heights were directly measured to 0.01" by means of a simple steel pointed probe of 0.10" diameter, detected with the aid of a standard CRO, the time base adjusted to give a stationary signal. The actual reading recorded was such that only 5 signals were obtained in any 10 consecutive waves.

Envelope heights were measured for each value of 'd', the length of the resonator. 'd' was varied up to 8 times per value of resonator width, which was varied up to 7 times for each value of width of entrance channel. In all, four different entrance channel widths were tested. The wavelength was generally held constant. Wave periods were measured by timing 100 consecutive waves and wavelengths were then calculated by substitution into the first-order equations.

Every experimental "run" commenced from still water conditions and measurements were conducted as soon as the partial clapotis in each domain had stabilised. Wave periods were measured both at the start and the end of the run.

Results.

All experimental readings were plotted. Nodes and loops were measured off these plots and the coefficients of reflection and transmission were computed. In this pilot study no allowance was made for attenuation, non-linear effects, or experimental error.

The coefficients were then all plotted against the dimensionless tuning parameter d/λ together with relevant results from theory (see Fig.4).

A number of photographs were taken of the entrance conditions at the resonator.

Discussion.

The photographs indicated extreme turbulence throughout the resonator and junction element. Velocities of this nature will occur very seldom indeed in any full-scale structure but the photographs provided an obvious warning of the navigational hazards resulting from an irrational design.

An obvious result is the discrepancy of the resonating values of the tuning parameter from that prediced by theory, not only for the fundamental mode but also for higher harmonics. Similar experimental results have in fact been obtained in analogous systems and have led to the formulation of "end corrections." The results of tests on resonators with curved entrances revealed further reductions in both the tuning parameter and the distance of the domain 1 partial clapotis from the resonator at resonance. This, of course, was to be expected.

FURTHER EXPERIMENTS

Introduction.

The pilot study provided reasonable agreement with the theoretical development, with the reservation that the "end correction" for the tuning parameter be evaluated empirically. However, the results obtained for large resonator widths were inconclusive and it was apparent that the main objectives in the final programme should necessarily include:

(a) a substantial reduction of non-linear effects,
(b) improved measurement technique, with regard to both accuracy and speed,
(c) vastly improved decoupling devices,
(d) a study of the effect of entrance channel width,
(e) a study of the effect of large resonator widths.

Perhaps the most important object of this second programme was to check whether a single resonator behaved as a single-degree-of-freedom oscillator, or, if not, to delineate the conditions under which this postulate did hold. Another important aim was to ascertain the validity of the linear theory used.

Apparatus. Experiments were carried out in the narrow flume, shown in Fig.5. It was hoped to shorten the upstream domain considerably in order to provide sufficient space for efficient measurement and absorbtion in the downstream domain. Eventually, however, a compromise was reached by the construction of the bend seen in Fig.5.

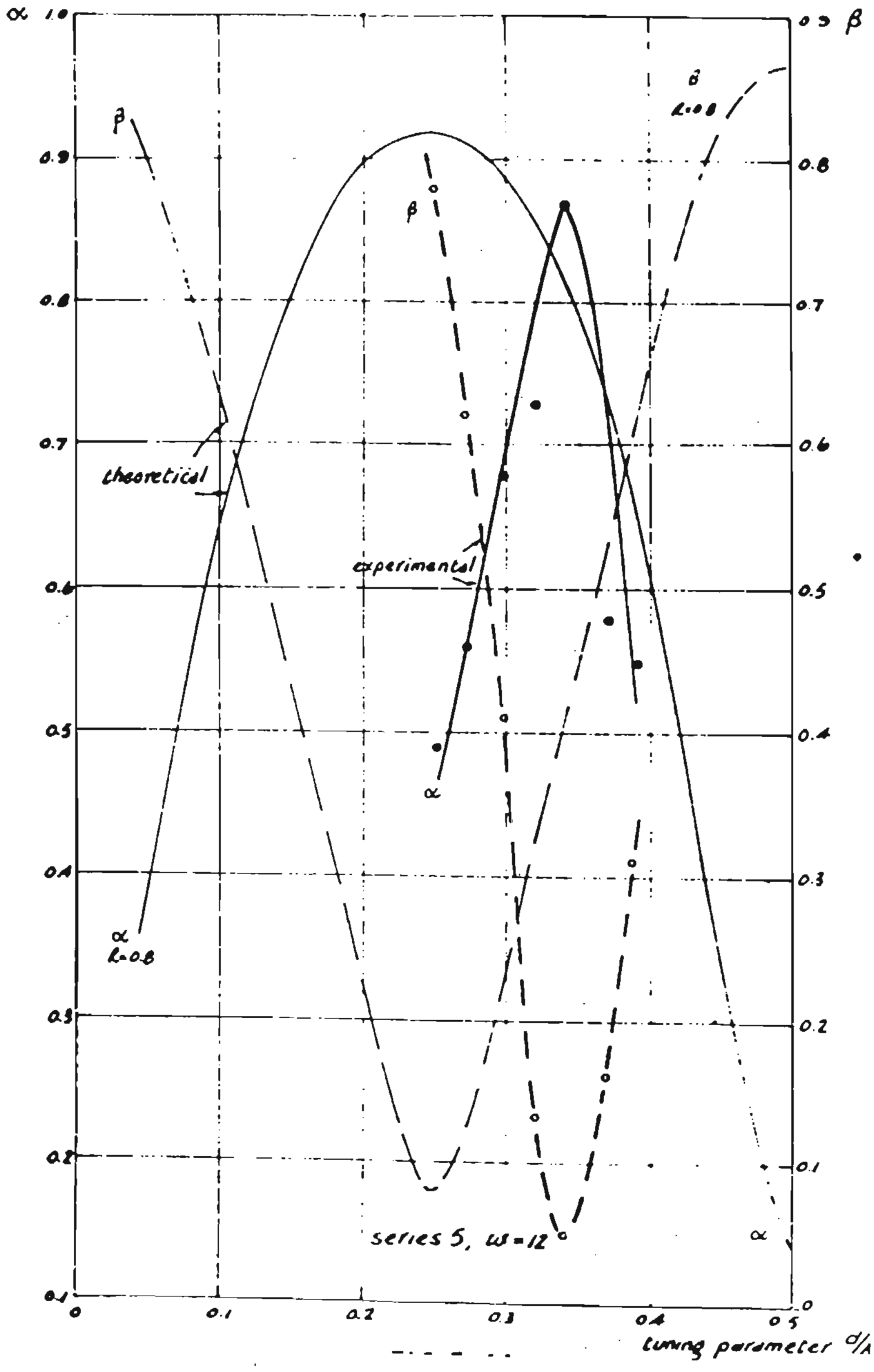

Fig. 4. Typical Preliminary Results.

Transverse oscillations were avoided by prohibiting such troublesome frequencies. No low frequency vibrations greater than 0.0001" were detected. Nevertheless a special instrument was attached, in case parasitic oscillations developed from other sources(see p.10).

Despite the smoothness of the perspex walls, side effects were easily visible under suitable low-level lighting. Disturbances radiated from the loops of the partial clapotis. Other sources of similar pseudorandom surface undulations were the filters, inconsistencies along the boundaries, vibrations inherent in the generator mechanism, and external vibrations, especially due to laboratory pumps and turbines and passing vehicular traffic. Surface tension was generally minimal, because of dispersive agents used in the measuring procedure, and consequently the "noise" resulting from meniscus inversion was probably of the order of 0.001" in amplitude.

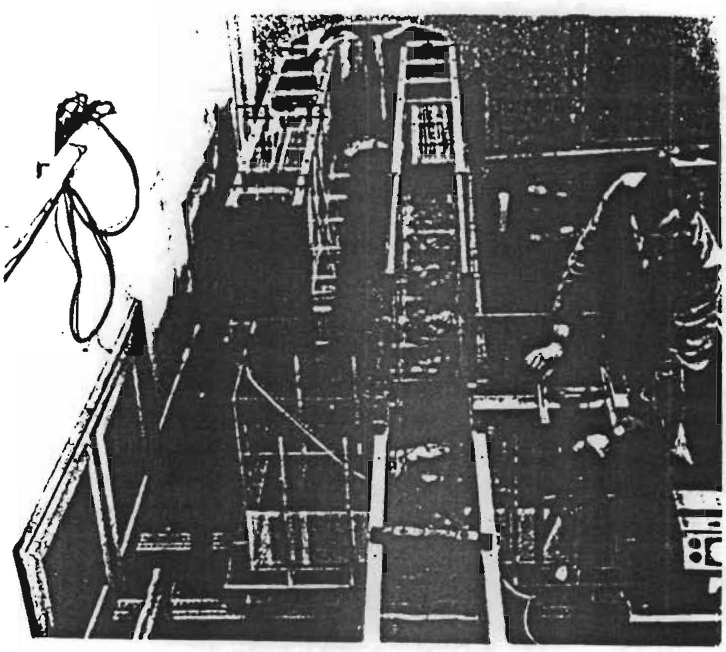

Fig. 5 Final Apparatus.

The incident wave periods and still water depths were chosen such that consistent waves were delivered at the resonators. For $T<0.45$ secs. waves became too unstable and it was found that the standard deviation of wave heights varied inversely with still water depth. For reasonably shallow depths, this instability rendered wave measurement exceedingly difficult. Shallow still water depths also necessitated long probes which increased measuring errors and consequently the smallest depth tested was 5".

Standard deviations were measured by means of a "dekatron" counting unit and the probes described below. The count recorded the "number of crests greater than" (or "number of troughs less than") for each micrometer setting. Such a count was taken for batches of 100 consecutive waves at setting increments of 0.0005" throughout the range.

Histograms were plotted from the differences in the counts for sequential settings, of which a sample is given in Fig.6. and these revealed symmetrical relative-frequency distributions. The computed standard deviations were of the order of 0.0015" for a = 0.25" and under the conditions obtaining. The coefficient of variation was less than 1%. This compares very favourably with standard deviations quoted by other workers.

The symmetrical bell-shape shown in Fig. 6. greatly facilitated measurement; the mean reading is given by approximately half the number of a fairly large sample of waves, say 20.

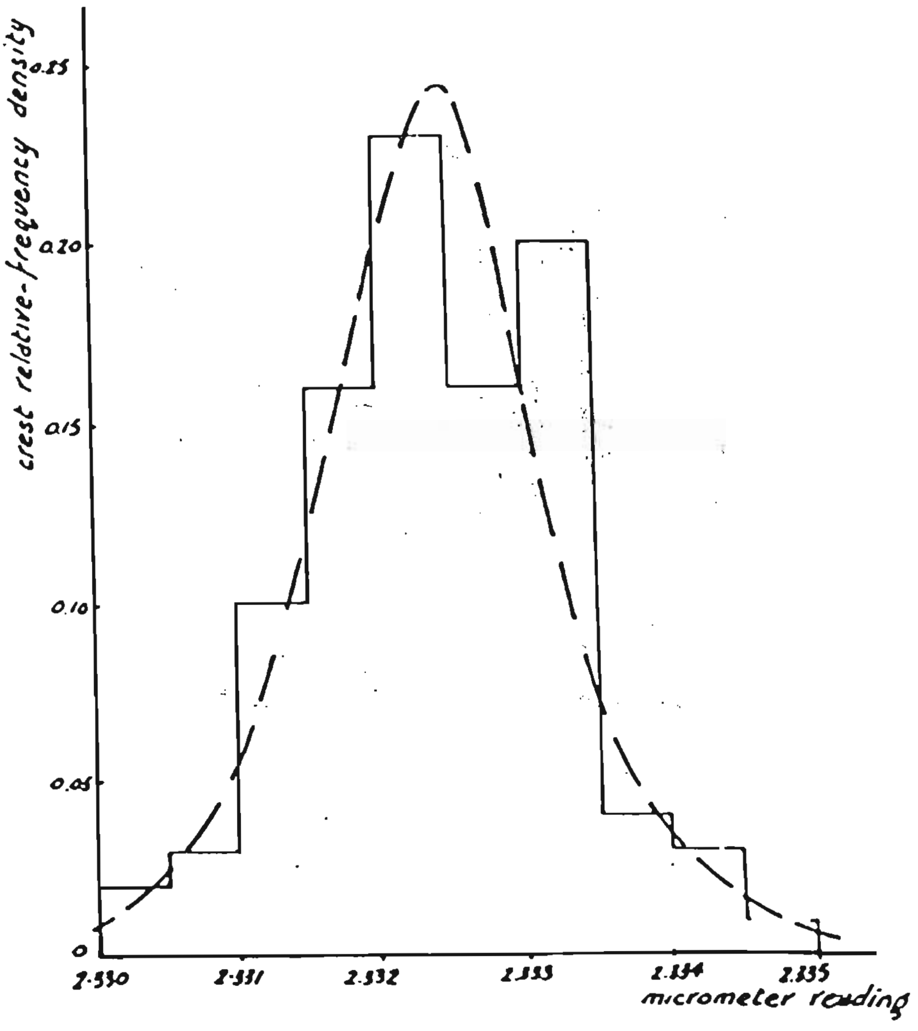

Fig. 6. Wave measurement histogram.

The writer did not investigate the reflection/transmission characteristics of the bend alone; the overall effect of the absorber downstream, the filters at each end of the bend, as well as the bend itself was examined. The filters were constructed of perforated zinc plate, 16 gauge (similar to those shown in Fig.7), 36" long, at 0.5" pitch, extending throughout the depth of water and measured coefficients of transmission were of the order of 0.4 - 0.6, depending upon wave length.

At least two filters were arranged in each domain, whence the total "decoupling coefficient" for each domain was at worst 0.06, assuming total reflection off the paddle and absorber.

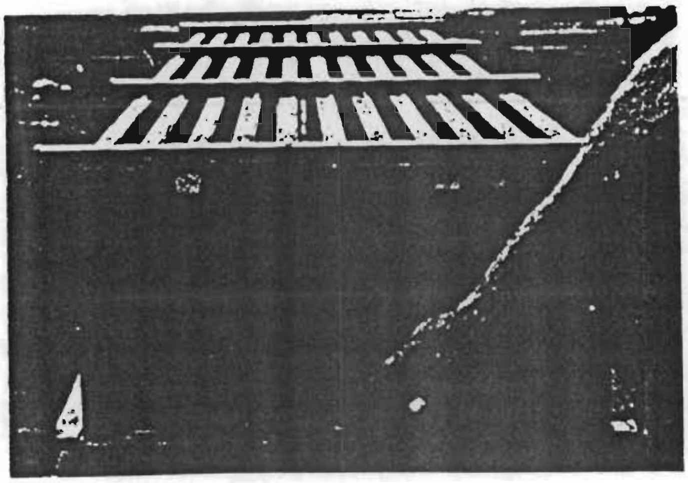

Fig. 7. Wave filters.

The reflectivity of the absorber used in the pilot study was found to be about 0.1. Although this is considerably better than rubble-mound absorbers - for which the value would have been about 0.3, a desirable value would be $\alpha \ngtr 0.02$. The requirements of linear theory dictate that wave steepnesses be kept minimal and this in turn demands a highly efficient absorber.

Constructed in perspex, the final absorber was 6'0" x 9" and the perforations were 1" dia. at 2" centres. When fixed in position downstream of the bend, the overall gradient was less than 0.007. The reflectivity of the entire domain 2 set-up was measured and the readings adjusted for experimental errors and non-linearity. The results demonstrated two points:

(a) decreasing transmission (from domain 1 to domain 2) with decreasing wavelength, due entirely to the dissipative forces.

(b) increasing reflectivity with increasing wavelength.

In an effort to detect long period fluctuations in water level, a 3 l. separating flask was attached to the end of the flume. A column of water was positioned in a short horizontal section of capillary tube attached to a large diameter rubber bung and the lower end of the flask was connected to the flume through large diameter glass tubing. A change of water level in both the flume and the flask of 0.001" would therefore displace the column of water several inches. The indicator had to be replaced regularly, as evaporation in the main flume caused sufficient reduction in water level. Short period fluctuations were ineffective because of the overall impedance of the arrangement. Within limits, the impedance could be adjusted by stopcocks. The instrument was attached to the flume behind the absorber, downstream of the bend.

Major sources of error are surface tension effects. When these effects are reduced, e.g. by using very fine probes and limiting instantaneous contact to a depression of the crest surface rather than actually puncturing it, it is usually impossible to see any meniscus effect. Probes were fashioned out of fine stainless steel sewing needles, nominal diameter about 0.015". The needles were coated with a thin layer of a P.V.C. akraline non-wetting paint, to ensure that no column of water drained down off the probe as it emerged from a crest. Probably because of galvanic corrosion, it was necessary to maintain the sharp points by careful abrasion.

Probes were attached to micrometer heads, capable of being read to 0.0001". The micrometer heads were securely mounted on heavy crossbars, clamped to the flume and the signal relayed to the cathode ray tube. A newer improved instrument uses a dial gauge, as shown in Fig.8. Another possible source of error is the sum of

(a) the depth to which the crest is actually depressed before a signal is obtained
(b) the height to which the surface of the crest is raised before contact is broken.

A dispersive agent was added continuously to reduce this and numerous very careful measurements were made on still water. The maximum correction was found to be 0.007". The extent of dynamic effects was not established.

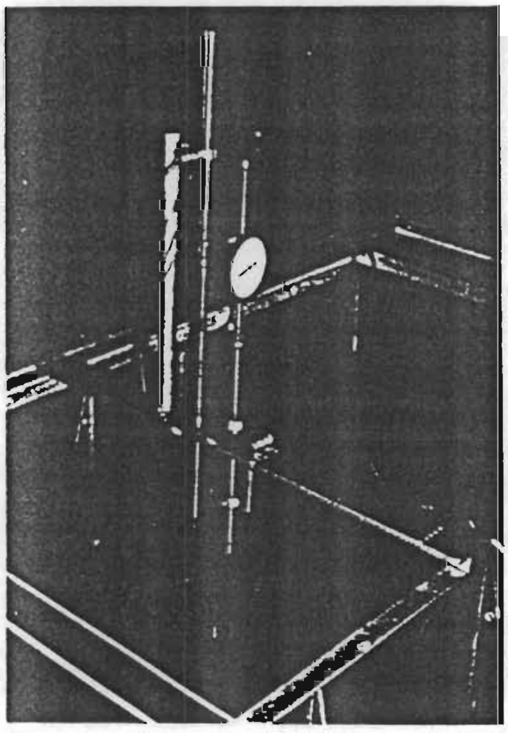

Fig. 8. Wave measurement.

General Procedure.

Wave periods (for at least 100 consecutive waves) were measured both at the outset and at the conclusion of the experiment. Both transverse oscillations and seiches were successfully avoided, so that the generator was stopped only at the conclusion of each experimental "run" when the resonator geometry was adjusted. Experiments were controlled in the laboratory by simultaneous curve-plotting. In this way the parameters were chosen to yield the family of curves plotted in Fig.13.

Usual methods for evaluating the two primary components of the partial clapotis in both domains have two basic assumptions in common:

1. The fundamental waves have harmonic wave-form.
2. Only two such constituents are present.

Consequently, disregarding measuring errors, the accuracy of both methods increases with decrease in both amplitude and wavelength of the incident wave (because of reflectivity). It was found that values of α calculated by the 3-point method were usually unreliable and the method was soon discarded in favour of the loop-and-node method. This enabled on-the-spot plotting of characteristics and thus greater flexibility and control of each experiment, a most important consideration.

On the other hand accurate scanning demands between 8 and 20 measurements of envelope height for each experiment, a somewhat protracted and tedious labour.

Data Processing. Resonator antinodes and loops and nodes for both domains were punched-up and a program was then written to correct for the measuring errors described above and to compute, inter alia, α and β.
These results were plotted as shown in Figs. 9 and 10.
The effect of attenuation and non-linearity is immediately apparent, from the fact that the plots do not tend to 1.0 or 0, as the case may be, as $d/\lambda \rightarrow 0$.

Discussion.

In assessing the accuracy of the results, the errors have been evaluated as follows:

(a) Systematic errors, due to attenuation and non-linearity. These errors do not affect the derivation of the geometry for resonance but are of paramount importance in establishing absolute values of reflectivity and transmissivity. Corrections for both these errors can be applied easily:

(i) Attenuation. All values of α and β should be corrected by multiplication by the reciprocal of β_o, the transmissivity at $d/\lambda = 0$.

This operation yields even larger values of α_o. The effect of domain 2 reflectivity at resonance is very small, i.e. usually $< 0.01A$ in amplitude in domain 1. It is incorporated in the random errors discussed below.

(ii) Non-linearity. The analysis of real partial clapotis produces low values of α, and negligible effect on β, when α is not very small. Corrections for this can be applied easily.

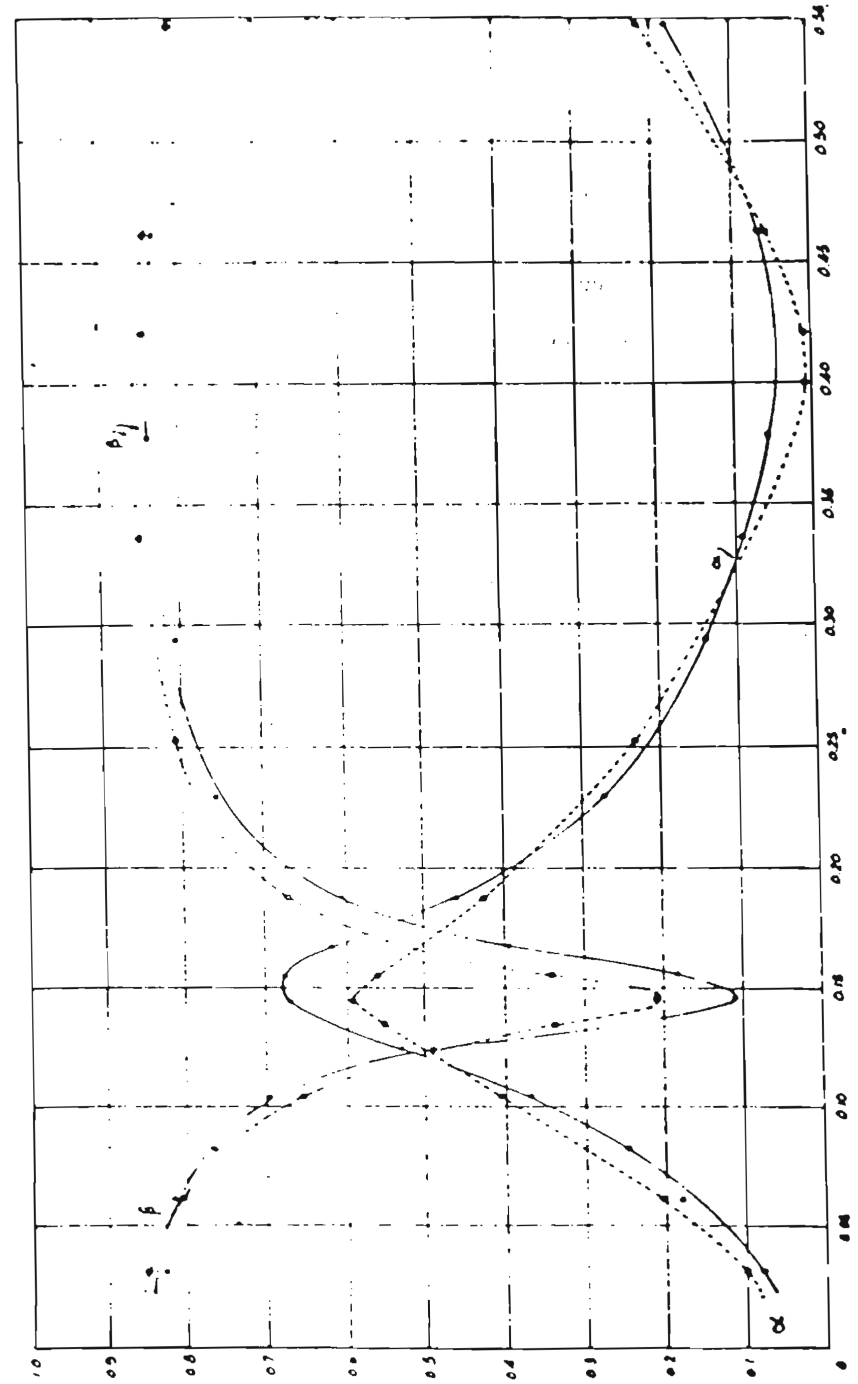

Fig. 9. Single Resonator Results, 1st mode.

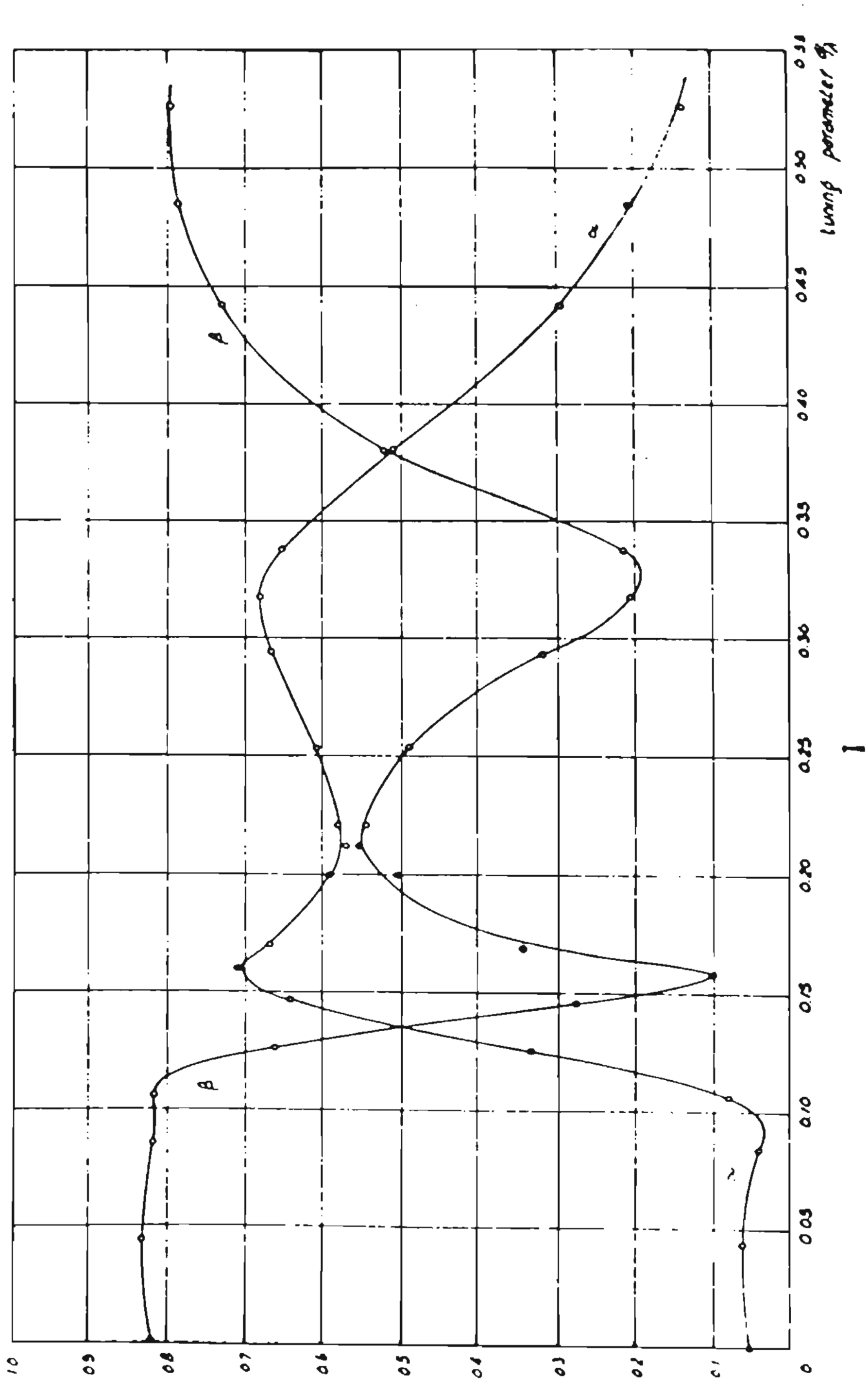

Fig. 10. Single Resonator Results, 2nd mode.

(b) Pseudo-random errors, attributable to generator instability, surface "noise", negative waves from domain 2 and the errors in the processes from measurement to plotting.

The writer did not attempt to assess individually such imponderable errors, as these are best calculated from the scatter obtaining in the graphs.

In error analyses, only graphs with a relatively high density of experimental results are admissible. As little scatter was found during the laboratory control of the experiments, the number of experiments was reduced to a minimum and, ipso facto, most of the curves plotted are unsuitable for error evaluation. However, certain curves were analysed and the probable error on the plot having the largest scatter of all the tests performed in the final programme was found to be 0.0146. The writer therefore submits that the random error was not much more than 1%.

The parameters of resonator geometry were rounded off to the second decimal place. The calculation of wavelength to this order of accuracy required an accuracy of measurement of wave period of 0.001 secs. and of still water depth of 0.05".

Figures 9 and 10, clearly indicate that the combined resonator and junction element acts as a two-degrees-of-freedom oscillator, under certain conditions. An appraisal of the α and β curves reveals a gradual transition from the single-degree system to the two-degrees system and an eventual preponderance of the second mode of resonance as w/λ increases beyond a value of 0.5. Further appraisal reveals that this "critical" w/λ increases with increase in W/λ .

The modes of oscillation are illustrated in figs. 11 and 12 respectively.

Note that the water surface sketched is exaggerated in the vertical scale; amplitudes were small and consequently the nodal lines sketched are inaccurate. Nonetheless, the sketches elucidate the mechanisms at resonance and illustrate the transition between these states.

Results for the first resonant mode were finally combined in Fig.13.

GENERAL CONCLUSIONS.

For the sake of simplicity, only the points of primary importance are enumerated below:

1. Resonators can be applied in harbour engineering for the elimination of waves causing short period agitation, influx of sea sediments and/or range action.
2. The results corroborate the results of the analytic theory developed and verify that the response of a short rectangular branch canal to periodical plane gravity waves propagated across the mouth is analogous to that of a single degree of freedom oscillator within certain limits:

For small values of W/λ and when $w/\lambda > 0.5$, the resonator becomes a two degrees of freedom oscillator. For larger values of W/λ this limiting value of w/λ increases.

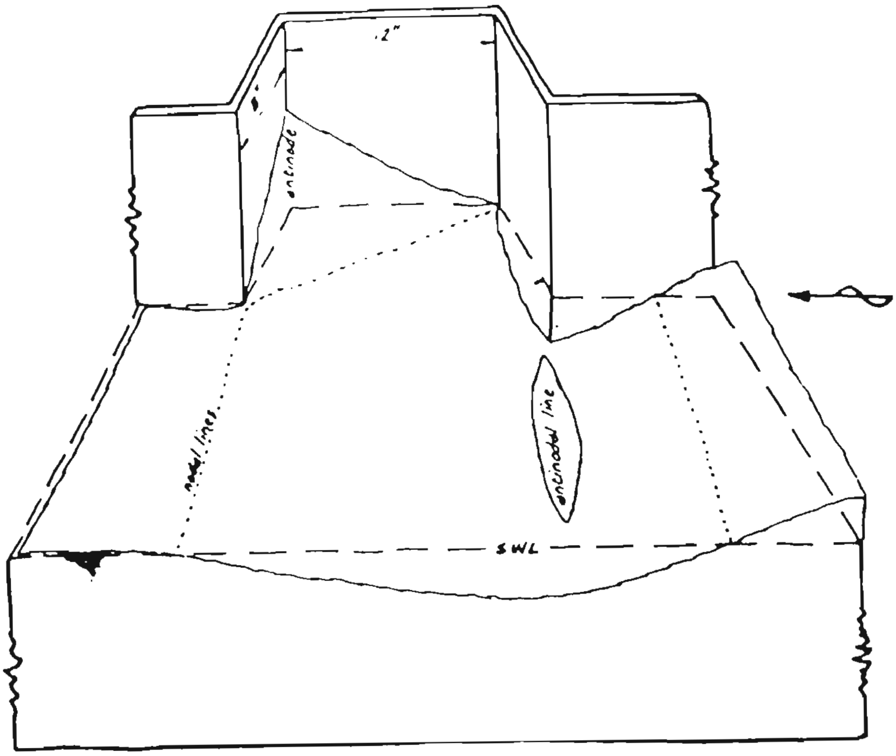

Fig.11. First mode of resonance.

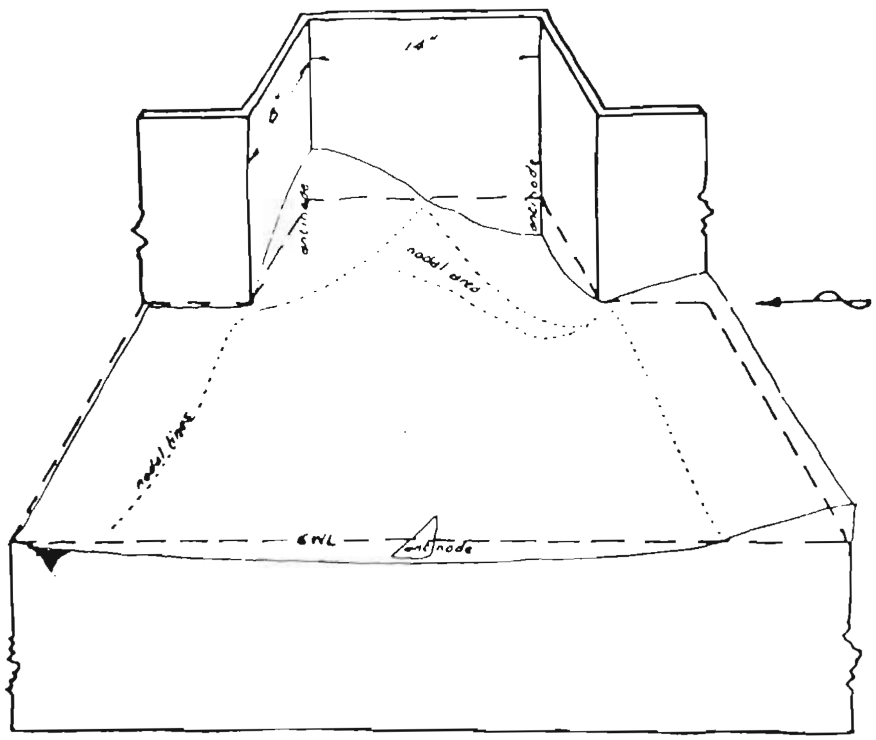

Fig.12. Second mode of resonance.

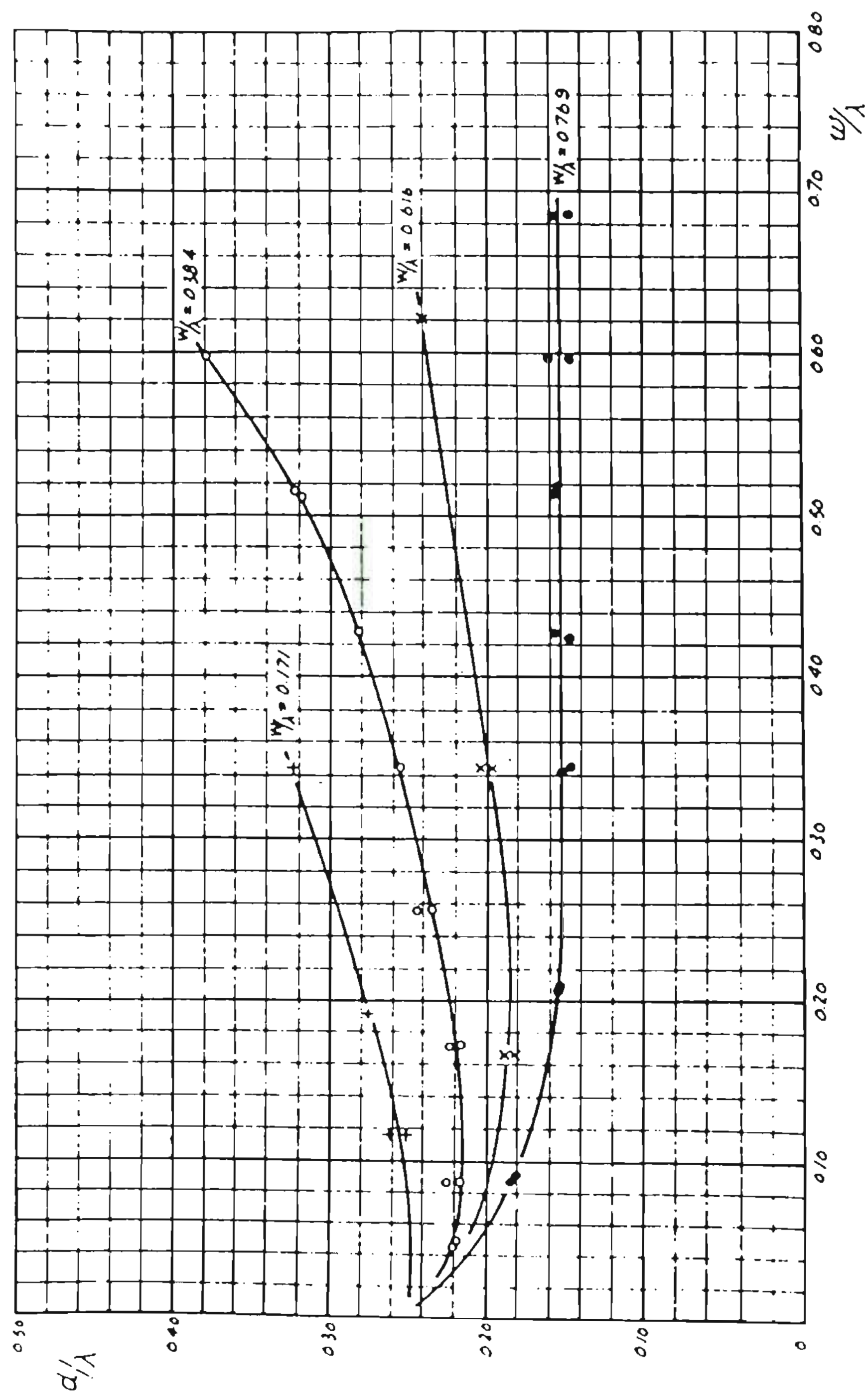

Fig. 13. First Resonant Mode.

Waves of the characteristic resonance frequency are not transmitted past the resonator but are reflected upstream as negative waves.

3. The zone of protection provided by a resonator is limited to a width of $\lambda/2$ in the downstream domain. Consequently, the benefits of resonators vanish as W increases beyond λ .

4. Resonant amplitudes of vertical oscillation in a resonator are finite, irrespective of viscous dissipation.

5. The cardinal point established in this investigation, is that resonator geometry for resonance differs greatly from earlier recommendations. Of utmost significance is the width of the entrance channel. Harbour entrance geometry at resonance is accurately related to incident wavelengths by the family of curves in Fig.13, which has immediate application in design.

6. The frequency band width for which β is less than any prescribed value is a function of w/λ (and increases directly with increase in w/λ) as well as a function of W/λ .
The dependence of the tuning parameter d/λ on w/λ at resonance is such that a minimum exists at $w/\lambda < 0.2$. Generally, however, the tuning parameter increases with increase in w/λ .
These two points indicate a compromise in the selection of w and d.

7. Optimum resonator performance is related to the energy dissipated in the resonator. At resonance, $\alpha = 1 - \beta$ approximately. Optimum performance appears to be relatively sensitive and further experiments, in which viscosity is carefully controlled, may be useful to fix absolute values. (In these tests there was no temperature control and changes of viscosity of up to 20% were involved.)

8. Finite wave heights reduce the effectiveness of the resonator. The effect appears to be small, however. Wave height has no influence on resonator geometry for resonance.

9. Small values of w/λ are accompanied by both decreased efficiency and high resonant velocities. Such velocities could prove a serious navigational hazard.

10. Where there is partial reflection in domain 2 the resonator should be located at $l = n\lambda/2 (n = 1, 3, 5 \ldots)$, or at the position of a loop if reflections are distributed. Ad hoc model tests would then be advisable.

11. Still water depth is of little or no consequence in the resonator geometry for resonance.

12. Non-uniform depths effectively reduce performance. However, the tuning parameter could be substantially reduced in the case of long waves and considerable savings in financial outlay could then accrue.

13. Current techniques used for the reduction of finite partial clapotis incur unreasonable errors, because of the trochoidal waveform. Suitable corrections can be applied easily. Current wave-flume measuring

techniques also incur systematic errors through viscous attenuation and menisci effects. Corrections for these can be easily applied. The random error encountered in similar investigations need not exceed 1% if a similar measuring procedure and experimental apparatus is adopted.

ACKNOWLEDGEMENTS

Guidance from Prof. J. Allen and financial assistance from the S. African C.S.I.R. and the University of Natal is gratefully acknowledged.

NOTATION

w	resonator width	l	length of an equipotential
W	channel width	$\|R\|$	transmissivity for resonator
λ	wavelength	$\|\Theta\|$	reflectivity in resonator
η	surface ordinate	y	vertical ordinate
a	semi-amplitude	h	still water depth
$\bar{\alpha}$	complex vector	ϕ	velocity potential
$\|\alpha\|$	reflectivity	E	total energy
j	$\sqrt{-1}$	ρ	mass density
$\hat{\alpha}$	phase angle	g	gravitational acceleration
k	wave number	z	transverse co-ordinate
x	horizontal abscissae	H_L	equivalent head loss
σ	circular frequency	U	a generalised velocity
t	time	K	dimensionless coefficient
$\bar{i}$	unit vector in x direction	d	length of resonator
$\|\beta\|$	transmissivity	$\bar{q}$	velocity vector
c	a constant	s	distance along an equipotential.
ψ	stream function		

Subscripts 1, 2, R, refer to semi-infinite ocean, downstream domain and resonators respectively.

Subscripts I,II refer to mode I and mode II respectively.

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PART 3: COMPUTER AIDED DESIGN
(8 papers)

All papers in this part were written and produced by me. Four papers were co-authored with Mark Robinson, my research assistant and the other two co-authors were former graduate students (P. Zachar was not directly supervised by me). Again, in all cases, the original idea, the lines of research, and the theoretical and experimental procedures to be followed, were laid down by me, and a considerable proportion of the basic software engineering carried out by myself. The co-authors were responsible for developing the computer code, and undertaking the computation under my continuous supervision.

EXCEPTION:

Paper CAD6: Although this paper was based on the M.Eng. dissertation by Robinson, the idea (shallow-buried, small-diameter, high-pressure, flexible pipe networks with distributed storage, delivering water on an intermittent basis) was uniquely my own. It is still regarded as an ingenious solution to drinking water distribution in low-income areas prone to disease. In the High Arctic this was complicated by precautions taken against freezing. Mark Robinson was a highly effective problem-shooter in this work.

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Interactive processors for design use of large program packages

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Interactive processors for design use of large program packages

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Civil engineering design methodology is changing as a result of emerging computer hardware and software. Highspeed, interactive, user-friendly systems are more common, whereas formerly batch submission of batch-oriented programs was the only common method. However, a variety of large, complex program packages are still widely used for urban drainage design problems. These packages are usually executed on mainframes with a large memory capacity and high computing speeds; users need a good working knowledge of FORTRAN input requirements and mainframe job control language, especially when the packages are used in a multi-program, multiprocessor environment. Furthermore, long learning times are associated with the implementation and execution of complex packages on batch-oriented mainframes at remote sites.

This paper discusses the design of an interactive package of pre- and post-processors to facilitate input data preparation, job execution, output interpretation, and job resubmission in a multi-program, multiprocessor, remote batch environment. The pre-processors create input data files for job execution; the post-processors operate on the output from the simulation package, synthesizing, analyzing, and plotting to accelerate the users' interpretation at their terminals. The important advance in this work is that the original coding for the main, externally supported, batch-oriented package is not altered. Furthermore, local text processing using previously prepared, disc-based outline reports and the post-processor output allows direct production of draft reports.

Le perfectionnement continu des moyens informatiques, équipements et logiciels, modifie profondément la méthodologie de l'ingénieur civil. Les systèmes interactifs à haute vitesse et simples d'emploi se répandent de plus en plus et prennent le pas sur les programmes couramment employés jusqu'ici, qui étaient conçus dans l'optique d'un traitement par lot. Néanmoins, une gamme variée de produits programmes d'application complexes sont largement utilisés dans le domaine de l'hydraulique urbaine.

De tels produits programmes sont habituellement exploités directement dans une mémoire centrale qui doit offrir une grande capacité et une vitesse élevée d'exécution; l'usager doit avoir une bonne connaissance des exigences d'entrée FORTRAN et du langage de contrôle, surtout lorsqu'il s'agit d'un système d'exploitation à tâche multiple. Bien plus, il faut prévoir un temps considérable pour la mise en oeuvre des produits programmes complexes dans le cas d'un traitement à distance au moyen d'un terminal lourd.

Cet article décrit la mise au point d'un programme interactif par pré- et post-processeur qui doivent faciliter la préparation des données, l'exécution, l'interprétation des résultats et la reprise du travail dans le cadre d'un système à tâche multiple permettant le traitement à distance.

Les pré-processeurs constituent des fichiers de données, tandis que les post-processeurs organisent les résultats de façon à les rendre utilisables par l'usager qui les exploite à son terminal. Un progrès important est réalisé grâce à cette étude: il est possible, sans modifier leur codage initial, d'exploiter les logiciels existants, conçus dans la perspective du traitement par lot, sur des terminaux interactifs utilisant des pré- et des post-processeurs. Bien plus, on est en mesure d'obtenir une version préliminaire de la solution du problème étudié grâce à un traitement local du produit fourni par le post-processeur, selon des instructions emmagasinées sur disque.

[Traduit par la revue]

Can. J. Civ. Eng., 9, 449–457 (1982)

Introduction

It is important to realize that the civil engineering profession is currently experiencing a major revolution in design, brought about by advances in computer hardware (i.e., equipment) and software (i.e., programming techniques). Four phases may be identified in this revolution:

1. more "number crunching," where limited access to batch-oriented mainframes allows more calculations ("number-crunching"), of the same type, than could previously be carried out on more elementary machines;
2. better "number crunching," where new programs incorporating advances in techniques of numerical analysis allow more design options to be explored, typically in a remote-batch environment, using, for example, design-office terminals;
3. new kinds of "number-crunching," where widespread access to inexpensive minicomputers allows wholly different problems to be investigated and resolved; new programs, techniques, and machines increase the scope of the engineering design; computing is an essential and a naturally accepted basis for the design.
4. much more than "number-crunching," where en-

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tirely new approaches to the design problem are taking root; for example, where communication with comprehensive models is through interactive color graphics that allow the design engineer to focus on difficult problem areas using a single keystroke on the terminal.

Few municipal engineering design offices have reached phase 4; most are somewhere between phases 2 and 3. The intent of this paper is to describe software techniques that can ease the transition.

It may be helpful at the outset of this paper to distinguish between the following terms: routine, program, package, model, and processor. A routine is a subprogram, a segment of coding designed to perform a simple task. A program is source coding, normally including several routines, designed to solve equations based on a mathematical derivation of concepts of real physical processes. A package is a collection of programs and routines, together performing related design-oriented calculations. A program normally requires a short systems program, written in machine-specific job control language (JCL), and a datafile before it can be executed on a machine. The JCL controls various system-oriented aspects of the computer run, for example, job priority, accessing of appropriate data files, and printing of program results. Part of the datafile (or input) relates to a parameterized physical description of the prototype, e.g., a discretized schematic of an urban drainage system. Another part of the datafile relates to a hydrometeorological time series, such as rain, etc., that forms the input to the model. The model, according to our usage, is the program coding, comprising generalized algorithms, plus that part of the datafile that parameterizes and describes one particular physical prototype drainage system. A processor is a program, usually unrelated to physical concepts of real hydrometeorological processes, but related to computer-oriented activities. The term processor is also a short form for central processing unit; thus a multiprocessor environment denotes several different computers used in conjunction, perhaps interconnected in some way.

Current practice in North American urban drainage design studies involves the use of a large variety of computer packages for modelling the components of an urban drainage system to varying degrees of complexity, e.g., the Stormwater Management Model (SWMM) (Huber *et al.* 1975) and HEC-2 (U.S. Army Corps of Engineers 1979). As well, comprehensive hydrometeorological data bases (rainfall, streamflow, and water quality) are being maintained by various government agencies. A typical problem faced by many consultants is how to efficiently implement these design packages and integrate them with the available data bases.

In this problem, civil and municipal engineers are faced with certain problems.

1. They cannot afford the time, people, or money to develop and/or maintain independent design packages. Thus they use programs specified and devised by others. Usually this means implementing a package whose input requirements are cumbersome and whose output is unsuited to the engineer's computing environment. Good documentation, seldom available, only partly offsets these difficulties.

2. They do not own a computer with sufficient memory and peripheral equipment to implement the package and data base. Often it is necessary to use a remote commercial computing facility where batch facilities are more economical.

3. At a large batch-oriented computing centre a great deal of time is spent simply waiting. The actual run-time involved in executing a job on the computer is a very small fraction of that required to create the data files and job-stream, advance the job through the machine's input and output queues, return the output to the user, digest the large amount of output typical of batch-mode computing, and prepare the subsequent run. Very often travel or transport times must be considered too.

4. Long delays in turnaround (i.e., the time that elapses between job submission and return of program output) and mundane, rigidly formatted, batch-oriented data preparation result in long learning times and in slow model verification, sensitivity analysis, calibration, and validation, before the model can be used to get results.

Evolving computer environments

Most potential users of batch-oriented design packages could be classed as casual users (Cuff 1980), i.e., they have a good grasp of the physical concepts being modelled and consequently know which packages are appropriate for solving a given problem. However, since they use the packages infrequently, they do not easily recall how to execute them on the computer system. Hence casual users tend to shy away from fully exploiting the available modelling resources and system capabilities. For example, a casual user may avoid interfacing two or more models, each describing an individual process in detail, if their joint use requires working knowledge of system JCL. A casual user would favour using a single, general model that deals with all the relevant processes in a more superficial way but requires little involvement with the computer system. Kennedy (1975) identified this problem as a major roadblock to the acceptance and full implementation of a computer system.

It is common to blame poor partnership between the user and the machine on the computer centre management. However, with very few exceptions, program input and output (I/O) controls are not conducive to developing a smooth working arrangement between the user and the "computer" (in reality, the model being run on the system). The fault often lies in the fact that the originator of the program is an experienced batch-user whereas most users range between novice and intermediate (Embley *et al.* 1978). What may be an obvious technique to a professional programmer may bewilder a user unfamiliar with system JCL or program I/O control. Moreover, large packages were often developed by organizations with good access to batch facilities, e.g., the Hydrological Engineering Center of the U.S. Army Corps of Engineers in Davis, California. Embley *et al.* (1978) succinctly defined the purpose of using a computer system: to allow a user to communicate directly with a machine to accomplish a goal with the minimum expenditure of time and effort. Casual users today tend to access design packages using remote slow-speed terminals (up to 1200 baud or about 120 printed characters per second).

Recently, significant developments have taken place in the fields of computer hardware and software. Microprocessor technology has made highly efficient microcomputers affordable by many users. Versatile, sophisticated packages are flourishing. However, most original computational packages currently in use in urban drainage design practice have a poorly designed man–machine interface, e.g., HEC-2, STORM, SWMM, ILLUDAS, and many others. Some, such as HYMO, have a good batch-mode I/O design. Nevertheless, there is an obvious need to improve this interface in order to permit the user to compute efficiently, develop a positive attitude towards computing, and fully exploit the computer as an information tool, i.e., to render the big batch packages user-friendly.

Interactive demand-mode computing

In batch processing a job is submitted to a computer through a peripheral device such as a card reader or keyboard terminal, the job is executed, and the output returned either immediately to the terminal or printed on a line printer at the central site and delivered at some later time to the user. There is no communication between the user and the model being run from the time the job is submitted until receipt of the output. Interactive or demand computing may also be initiated by either type of input device, but usually it will be done from a remote terminal. However, the package halts execution at predetermined points to permit the user to influence the remainder of the calculation, based on interim results output to the terminal.

An extension of demand computing is conversational computing. In this mode a dialogue is carried on between the model while it is being run and the user in an effort to prompt or guide the user in making run-time decisions. Such packages are usually embedded in a user-activated procedure that automatically generates the necessary system JCL. This is particularly helpful to casual users.

The relative merits of batch-mode processing and interactive demand mode computing have been examined (Miller and Thomas 1977); differences in programs have been found to be a more important source of performance variability than system mode differences. For instance, a programmer–user would probably find poorly designed, interactive processing less efficient than batch-processing for achieving the same goal.

Assuming that two different but equivalent packages exist, the decision to use batch or demand computing depends on two issues: whether or not it is useful for the engineer to interact with the calculations in progress, and the amount and type of data required to be altered when setting up the various runs of the model. The question is compounded by the limited objectives of a casual user unfamiliar with the advantages of demand computing. For example, there is an undeniable case to be made for systematic verification, sensitivity analysis, and validation when developing and/or using any model in a design application. Scant attention has been paid to this in typical batch-mode design applications. A demand-mode environment clearly favours a more responsible attitude to modelling (James and Robinson 1981).

Usually urban drainage design engineers are faced with exploring a number of alternatives for feasibility and/or cost effectiveness. They make subjective evaluations of available information to decide on the best solution. Thus an interactive computer procedure should not be an inflexible method in which the user plays only a passive role, for example, where he may merely read the result of an optimization process. The procedure must be a tool with which the user interfaces with the machine. Ultimately, all design decisions should be made by the engineer. A properly designed, interactive procedure should not make inferences; rather, it should ensure that the user receives full information regarding all options in a multi-objective problem and in such a form that the user is most likely to make the correct, or an acceptable, decision.

Design of dialogue

If a program operates in a cumbersome or inefficient manner the user could be limited to a small set of options. Newstead and Wynne (1976) and Roy (1980) found that suitably designed, interactive procedures assist users in making the judgements necessary for the solution of multi-objective problems by providing more complete information. Interactive computing would appear to be preferable for most complex designs but the package must be efficient to be viable.

The man–machine dialogue is the central issue. The importance of making the numerical model work for the user has been recognized (Fitter 1979; Cuff 1980; Kennedy 1974, 1975; Treu 1975; Miller and Thomas 1977; Gaines and Facey 1975; James 1978; Shneiderman 1980; Martin 1973) and cannot be overemphasized. If the method of communicating with the computer is complicated and exacting, or the dialogue ambiguous, the positive aspects of computing will be nullified. The dialogue must flow naturally. The knowledge and processes for which the users and the "computer" share a common understanding should be made clear (Fitter 1979). The users understand that they are to supply information for a certain calculation technique and the machine will "understand" (i.e., is so programmed) that it will be required to interpret the information necessary for that technique.

Fitter (1979) observed that users often assume that the computer incorporates unrealistic intelligence and powers of inference (i.e., it is able to solve problems for which the requisite process-algorithms are not present in the package being used). The dialogue must be designed to prevent this tendency. Where possible the package warns the user of the limitations of the component process-algorithms and routines. Treu (1975) identified a range of languages that could be applied to interactive systems. These range from a computer-oriented artificial language that is difficult for the casual user to recall (Kennedy 1975) to a language very similar to the spoken word that is essentially impossible for the computer to comprehend. A flexible, user-oriented, limited language has evolved in our conversational procedures. A specific set of allowable commands is used that can be interpreted at any point by the computer; these are therefore unambiguous, yet have an obvious meaning for users and thus are more easily remembered.

In creating interactive procedures, it is necessary to consider the exact nature of the interface between the user and the machine. Miller and Thomas (1977) identified four basic types of interface.

1. System-guided, where the user has limited choice; the user is taken step by step through a fixed procedure and must choose data from a specific list of items. This form of interface provides quick processing and minimizes error.

2. System-guided, where the user has free response; the user is taken step by step through a fixed procedure but can supply data in any form he chooses. This form of interface has the potential, especially in numerical applications, for ambiguity and error.

3. User-guided, where the user has limited choice; the user has the option at all times of instructing the system in which direction to proceed, but can only choose data from a given list of items. This type of interface would be appropriate for users who have some familiarity with the potential of the system. It could result in increased exploration of the possible solutions while at the same time minimizing errors.

4. User-guided, where the user has free response; the user selects the direction of the system and supplies data in no prescribed form or order. This interface has the potential for the most error and ambiguity and should only be used by experienced users.

Although we have experimented with all four types of interface, our current interactive procedures are tailored to a user evolving from novice or casual into a sophisticated user, i.e., combining interface types 2 and 3. The user selects a specific calculation option or combination of options and the system will guide him, using brief switchable prompts, and switchable echoes of dependent parameters, to supply the required data. Experienced users in repetitive design situations switch prompts and echoes on and off randomly. The values that may be supplied for the required data are left to the user's discretion but extreme values cause cautionary responses, and default values are used. This maximizes the number of alternative methods that the user can explore, while minimizing potential error.

There are three levels at which attention should be paid to user psychology: the functional, the procedural, and the syntactical levels (Robinson 1977).

At the functional level, it must be made clear which functions are to be carried out by the user and which by the computer. A brief introduction at the beginning of the procedure is important. The user is made aware that the computer will prompt, warn, and echo when necessary. The user can devote his time and attention to interpreting the design problem. As well, the interactive software should accord with the ongoing calculation. For example, the function of the pre-processor is to create the input data file required by the analytical package. The prompts for input coincide exactly with the documentation of the input for the external package, written and supplied by the original authors. Source documentation remains directly applicable.

At the procedural level, the software is designed to provide alternative methods. The one chosen will direct the program logic along a certain course. The designer determines the course by selecting the appropriate method.

At the syntactical level a number of guidelines for achieving effective conversational software can be proposed. The manner of presenting prompts for input and displaying output is the area in which the greatest scope for improvement exists. The manner in which data are formatted can have a profound effect on the user's efficiency and error rate.

Treu (1975) defined mental work as the concept of certain procedures, tasks, or commands being more or less difficult than others. Further, he defined a concept called the user transaction time as being the time from the output of the last system character to the input of the last user character. The duration of each component of the user transaction time varies, depending on the length and complexity of the system message, the ease of selecting an appropriate command, the length of that command (number of terminal keys depressed), and the user's reading and typing skills. One objective in designing an interactive system should be to keep the user transaction time to a minimum while maintaining a clear understanding of what is required of the user.

Criteria for design dialogue

For the design of a well-behaved interactive system we have established nine criteria from our own experience as well as others' (Fitter 1979; Cuff 1980; Kennedy 1974, 1975; Miller and Thomas 1977; Gaines and Facey 1975; James 1978, 1979; Robinson 1977; Shneiderman 1980; Martin 1973). (If these criteria appear obvious, consider the number of systems and programs in widespread use that violate them.)

1. The dialogue should be terse, coherent, and unambiguous, yet should be conducive to a cooperative attitude. The behavior of the machine should be consistent with the dialogue, which should be structured to flow smoothly from one concept to the next. The users should comprehend the purpose and logic; they should know what end result they are seeking, i.e., all their pieces of input should follow a logical sequence. A prompt for input should carry a tone indicative of the form of response required. The prompts should always be presented in the same manner. The signal that input is required (e.g., the use of three dots) should appear in the same position, say 40 spaces from the left-hand edge of the paper, so that the user becomes accustomed to the manner in which input is requested and entered. This minimizes the chance of error and develops a reasonable tempo for the session.

2. Prompts should be concise to minimize the chances of ambiguous interpretation by the user. Each prompt should be prefaced by a number that allows the user to refer easily to a pocket manual for aid. In particular, experience has shown that concise, complete, easily readable, portable documentation for a program is often a much more effective aid than a large output file of HELP commands displayed at a terminal. Use of a manual containing examples, in conjunction with an informative diagnostic message presented at the terminal can speed problem resolution. This is especially useful when numerical quantities are to be entered. The manual should outline values that the parameter in question could take in a variety of circumstances. (The pre-processor also should check input against outer limits beyond which the input is clearly absurd.)

3. The user should not be required to respond to more than one idea at a time. This reduces confusion and frustration and minimizes error.

4. The input translation routine should accept free format data. Users not familiar with a particular computer language become confused when suddenly required to enter data in a specific esoteric format, for example G10.3. As well, the translation routine should accept both abbreviated and full commands, instructions, comments, and data. This accommodates users with a range of experience. An obvious example is the entry of a blank string for data elements, where the interactive system would automatically substitute default values for such blank data.

5. The computer should always respond to the user. Some indication should be given that the user's response is being processed, or has been accepted or rejected. This may be a specially designed echo, message, or another prompt. All related input prompts, i.e., for a particular phase of the design, should be made in proximity to one another; during lengthy calculations users are periodically informed "calculation proceeding."

6. The user must be able to observe and control the procedure and be able to abort the current state of the system and/or reset the procedure to the initial state, an earlier state, or a new, user-specified, local state.

7. Data entered should be validated by checking syntax and comparing with reasonable limits. Care should be taken not to necessarily reject data outside the "normal" range; it is surprising how much data is not "normal." Rather, a diagnostic message should be flashed on the screen; this does not antagonize the user but rather invites him to reconsider his response.

8. Error messages should be designed to convey information in a manner that is concise yet does not antagonize the user. Carefully worded error messages will encourage the user to try again without giving the impression that errors will be tolerated all the time.

9. Results relayed to the terminal should be ordered and easily read using graphics or clear printer-plots. This output must convey only essential data yet be clearly labelled to indicate its significance. It must be remembered that further input will likely depend on these results. If the user cannot decipher them, he cannot supply additional data and the design sequence breaks down.

Dialogue designed around these basic criteria will be efficient, user-friendly, and reliable.

Control of user errors

Users will make mistakes. It is essential that the interactive system detect as many errors as possible before the engineering design can be affected, and inform the user politely. Martin (1973) provided an in-depth examination of the control of user errors. Design

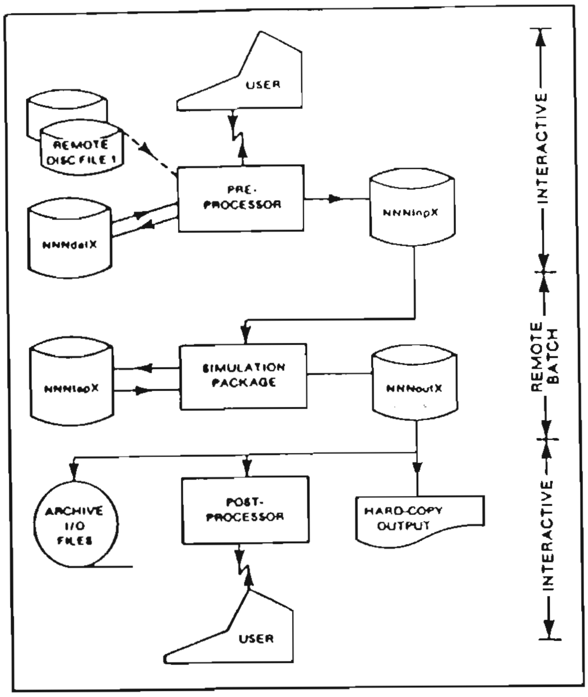

FIG. 1. Interactive processing of a batch-oriented simulation package.

criteria for the control of errors (Martin 1973; Robinson 1977) include:

1. structuring the dialogue to immediately detect as many errors (syntactical and numerical) as possible;
2. providing the user with an unambiguous description of the error and the reason why it occurred; and
3. providing a facility for immediate correction of errors.

Generally, error messages should provide information and encourage the user to try again, avoiding repetition of the mistake.

Interactive processors for design packages

Two large packages known as FASTSWM and FASTHEC2 have been developed by the authors. In each case the interactive system comprises three parts as shown in Fig. 1: the pre-processor, the large externally supported package, and the post-processor. The functions of the pre-processor are to:

1. solicit and accept design-control input for administrative routines for file auditing and manipulation;
2. solicit and accept user-directed input required by the external package, in a conversational free-format mode;
3. solicit and accept design-oriented input data (if any), such as a rainfall or temperature data base, from input devices or units as directed by the user for the evolving design;
4. check the validity of the input and return values of certain important, dependent parameters to the user's terminal;
5. convert the units of the input data (often SI or metric) to units required by the main simulation package (often Imperial if originating in the U.S.A.);
6. convert the free-format user input to the formats required by the external program, for example, from a line-oriented free format to a rigid, arbitrary, fixed format;
7. construct a file of system job control language to submit the main program to the batch CPU together with these properly formatted input data; and
8. save uniquely identified input files in the user's catalogue to be subsequently modified, archived, or destroyed.

The function of the external package is to model the requisite hydrologic processes in accordance with the original documentation.

The post-processor's function is to:

1. return selected output to the user's terminal upon program completion and after reconverting (if necessary) to appropriate units;
2. calculate various correlation functions between simulation results and observed or expected data (if any), to evaluate the current solution;
3. printer-plot the results if requested; and
4. reinitiate pre-processing for further exploration of the solution once data modification has been made.

By arranging these processors outside the main external package, the latter coding remains untouched. An update is simply substituted for the current version. The processors are easily modified to incorporate any I/O changes.

Dialogue procedure

Our dialogue procedures are primarily concerned with alphanumeric data entry, but the user may take control of the procedure by entering a command. The following is the minimum subset of commands, used in place of data:

BACKSTEP (start this step over again)
DEADSTART (start the pre-processor over again)
PROMPT (alternately switch prompting off/on)
ECHO (alternately switch echo checking off/on)
DROP (terminate procedure)

Although other commands are frequently used, this minimum subset will achieve the objectives listed in this paper.

Commands are expressed in at least the first three columns of the data line; columns 4–80 are generally used for numeric data, keywords, and alphabetic comments. The procedure commands are started in the first space and may be curtailed after the third letter; only three letters are necessary to identify the command.

When prompts for entering data are printed at the terminal they are prefaced by an alphanumeric code. This code uniquely identifies the data group expected as user response to the prompt. A diagnostic message may refer to data item EX5.4, indicating the fourth item in data group 5 (formerly "card groups") of the EXTRAN block of SWMM. In general, the term group corresponds to cards in batch computing although line may be an alternative term. Again, reference to documentation is facilitated. A prompt for data in the SWMM EXTRAN block may be prefaced by the code EX 5. This indicates to the user that the input being requested is for the EXTRAN block and that data group 5 is expected. The user can easily refer to both the processor pocket documentation and the comprehensive user's manual for the original design package. The two sets of documentation and the prompt codes correspond. A further extension to this prompt code is in error checking. A data item is uniquely identified by the number following the decimal point in the code. Spaces or commas are used to separate data. When all data corresponding to one prompt are entered, the line is transmitted via a carriage return command.

The data can be written in any format, leaving at least one blank space or comma between data items. If two commas occur together, for example, the processor will assume and insert a zero in default of the expected data item, a convenient type of shorthand for the experienced user.

A decimal is required for numbers containing fractions, but not for whole numbers. Alphabetic keywords and comments including special characters can be interleaved with the data to describe individual data items. The input translation routine will disregard non-numeric input where it expects only numeric data. This is convenient for annotation for future archiving.

Implementation of the FASTSWM and FASTHEC2 procedures

Figure 1 depicts the typical operation of the interactive system. Through a log-on procedure, the computer system on which the procedure resides is accessed and the procedure is activated using a "CALL, procedure name" command. Besides simple editing, this is the only system command the user needs to know and presents no difficulty for the casual user.

Upon invoking the procedure the user will be asked to provide the following three pieces of design-control or job-related information:

1. a three-character identifying code, *NNN*, for this particular user job session, where the *NNN* can be any three characters, usually the user's initials;

2. a single identifying case number, *X*, to differentiate unique trials for this particular user or job session, where *X* can be any character; and

3. whether a new data file (NEW) is to be created from scratch for this job session, or whether an existing, modified data file is to be reprocessed (OLD); All NEW files need a new case number *X* unless an existing file case number *X* is to be overwritten (i.e., replaced).

The pre-processor creates two files: (1) *NNN*dat*X* is an echo of the free-format input provided by the user and may be easily modified and resubmitted to the pre-processor as an OLD data file. If requested, the pre-processor accesses remote design-oriented disc files and amalgamates them with the free-format input to produce a complete design data set. (2) *NNN*inp*X* is a data file that is specifically formatted and is in the units required by the main simulation package, and which will form a portion of the input to the package.

The pre-processor builds a JCL file that, if requested, submits the main program package, together with file *NNN*inp*X* and any peripheral data files generated by previous runs of this or another simulation model, to the batch CPU where the calculations are carried out.

The results generated by the CPU are returned to the user's terminal. In addition to *NNN*dat*X* and *NNN*inp*X*, files created include:

(a) a listing of results that need to be presented to the user to form part of the decision making process — file *NNN*out*X*;

(b) a machine-readable code comprising pertinent results in a form suitable for input in subsequent runs of this or another simulation model — file *NNN*tap*X*; and

(c) a job dayfile recording CPU times for each constituent task, for program maintenance.

The procedure activates the post-processor after the main package has been executed and assigned to disc storage. Key results are returned to the terminal in summary form and, if requested, as a printer plot. The user may then request further analysis of this output including various correlation functions. Based on the results, the user decides whether to continue to explore the solution further. If so, the original free-format data file *NNN*dat*X* is modified, using the system editor, to formulate a new approach to the problem. Additionally, the user decides whether to invoke a new segment of the program package, in which case a NEW job session or case number may be implemented. The design proceeds in a cyclical, iterative fashion. The user is freed from the need to learn system JCL to create disc files for subsequent runs, and can run an unlimited number of jobs from the outset. The system has also been implemented on a multiprocessor system, as shown in Fig. 2, in such a way that it remains comprehensible to novice users. The procedure evidently reduces considerably the complexity of job submission; users are able to focus on the processes modelled. No knowledge of input formatting or of systems control language is required of users.

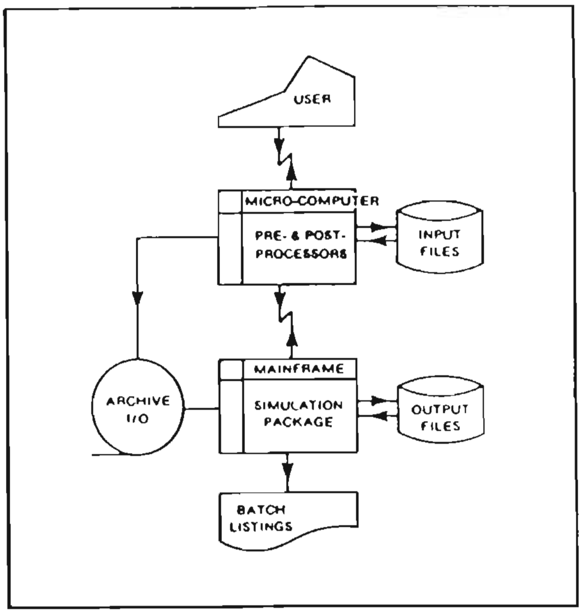

FIG. 2. A multiprocessor environment for interactive processing.

During the past three years, about a dozen professional seminars using this system have been held in Canada (Toronto, Edmonton, Niagara Falls, Hamilton) and in Sweden and South Africa. The system is also regularly used in graduate and undergraduate courses at McMaster University. This method has been found to reduce learning times, minimize user errors, and reduce total design turnaround times. As such it has proved especially valuable in an instructional or training environment; in fact, hands-on instruction, including up to 100 job submissions per novice user, is now possible in a 3-day professional seminar on packages such as SWMM and HEC-2. Novices carry out model verification, sensitivity analysis, calibration, and validation runs in this time.

Conclusions

Emerging computer hardware and software is affecting civil engineering design methodology in urban drainage design. Engineers are making more use of interactive, user-friendly systems to fully exploit the design opportunities offered by large-scale program packages. The transition to interactive computing from batch-mode computing is aided by a package of pre- and post-processors described in this paper. Design time, including user transaction time and program turnaround time, is greatly reduced when an interactive approach to computing is taken. Various requirements for a good man–machine dialogue are reviewed. Concise, unambiguous, man–machine dialogue promotes a reduction in user transaction time, minimizes the potential for error, increases user confidence, and encourages model verification, sensitivity analysis, validation, and the investigation of different design alternatives. Transmitting only the key output to the terminal speeds up the final design.

Conversational, interactive, pre- and post-processing procedures greatly facilitate the preparation of input data, job submissions, output retrieval, and the interpretation and subsequent investigation of possible solutions to multi-objective problems. In the method described, the original coding of the central or core external package remains untouched, thereby facilitating incorporation of new versions and/or updates whenever available.

The post-processor is easily interfaced with a text processing system to reduce the time involved in producing a draft report.

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Standard terms of reference to ensure satisfactory computer-based urban drainage design studies

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Standard terms of reference to ensure satisfactory computer-based urban drainage design studies

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The main purposes of modelling are a reasonable understanding of the physical processes involved and a careful evaluation of reasonable alternative designs. The central concern is credibility.

Inreasingly, government and municipal engineers involved in urban drainage design supervise studies in which sophisticated computer models are being used by specialists. Because of the large variety of programs available for modelling, the rapid evolution of the models, and because of their complex structures, the studies are becoming increasingly difficult to manage.

In the initial study terms of reference, the following component study activities could be specified in detail when bids are requested: problem review, study objectives, performance criteria, requisite accuracy, review of available programs, available data and study resources, program selection criteria, model verification, model calibration, model validation, minimum level of discretization, sensitivity analysis, data preparation and output interpretation, documentation of the modifed program actually used, and preparation of machine readable input and output files for archiving. Many of these activities will be carried out by a reputable engineer in any case.

By including these terms of reference, all consultants bidding will be assured that their costs will be met, and clients will be insuring their investment in design engineering with a modest premium. Each of these points is discussed using examples appropriate to stormwater management modelling studies. However, a number of the concepts can be applied generally to other disciplines using computer-based modelling.

Les principaux buts de la modélisation consistent en une compréhension satisfaisante des phénomènes physiques et une évaluation soignée des variantes d'un projet, le souci majeur étant la fiabilité des études.

De plus en plus, les ingénieurs des administrations engagés dans l'étude des drainages urbains dirigent des études au cours desquelles des spécialistes exploitent des modèles complexes et raffinés de simulation sur ordinateur. En raison de la variété des programmes disponibles en ce sens, de la rapide évolution des modèles mathématiques et de leur structure complexe, de telles études deviennent toujours plus difficiles à diriger.

Au niveau des études préliminaires, la description du mandat peut comprendre les éléments suivants: définition du problème, buts de l'étude, critères de performance, précision exigée, revue des programmes disponibles, critères de choix d'un programme, vérification, étalonnage et validation du modèle, étude de sensibilité, préparation des données et interprétation des résultats, justification documentée du programme modifié tel qu'utilisé, préparation des intrants et des extrants sous une forme lisible à la machine et se prêtant à l'archivage. Plusieurs de ces opérations sont effectuées par un ingénieur de bonne classe.

En incluant tous ces éléments, les consultants soumettant des propositions d'études s'assurent d'oeuvrer à l'intérieur des coûts prévus, et les clients ont l'assurance d'un travail d'ingénieur de qualité à un coût raisonnable. L'article étudie chacun de ces éléments en les illustrant par des exemples empruntés au domaine de la gestion des eaux pluviales. Un certain nombre des concepts utilisés présentent un caractère général qui les rend applicables à d'autres disciplines qui font appel à la simulation sur ordinateur.

[Traduit par la revue]

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Introduction

In this paper we discuss some special requirements of *computer-based* urban drainage studies. These requirements are supplemental to those relating to the specifics of a particular study. The purpose of these additional requirements is to guarantee credibility and confidence in the computer modelling. It is suggested that these special computer-modelling requirements could be set out in the initial study terms of reference or request for proposals. If these requirements are excluded therefrom, it is suggested that consultants submitting study proposals recommend inclusion of these items to their clients. The suggested additional terms of reference are summarized as part of the conclusions to this paper.

Several factors have incerased the complexity of computer-based urban drainage studies over the years:

(1) more and more programs are becoming available;

(2) the models are including more processes;

(3) the variety of computing hardware is increasing;

(4) the cost of computing is decreasing;

(5) software capabilities, especially of the smaller computers, are becoming more sophisticated;

(6) computer communication between design offices and remote main-frames is becoming easier; and

(7) there is a professional drive within most engineers to improve their understanding of computer modelling,

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despite the longer learning times associated with this methodology, higher salaries, and related costs.

These developments increase the pressure on regulatory agencies to approve sophisticated design technology which they may not have had time to study fully themselves. Some of these factors are further described subsequently.

Individual studies often differ greatly in their objectives and available resources. Consultants usually review the complete study problem and available study resources at the outset in order to establish a mutually agreed scope and basis for the model study. The problem review should not only be done in a general and descriptive way. It should be directed so as to purposefully list all those characteristics of the study problem that will aid in the selection of computer programs. It should attempt to list specific problems that ought to be considered in any model study undertaken. For example, if hydraulic jumps or energy dissipation at manholes are important considerations in a drainage study, these should be identified.

The study objectives and relevant objective functions should be listed and related. All questions to be answered must be documented in order to explain how the numbers generated by the computer model will relate to the general objectives of the study. For example, urban runoff computer models often produce only hydrographs and pollutographs resulting from a certain limited hydrologic time series. Simple parameters based on these response functions (hydrographs) include: peak values, time-to-peak, and total runoff or loadings. On the other hand, the study objectives often require a least cost alternative to a specific flooding or drainage problem, such as the production of toxic or other harmful pollutants as a result of a certain hydrometeorological time series. This concise explanation of the logical relationship between the model study results and the objectives of the design is important for the rest of the study.

Model complexity

Many computer models are capable of describing urban drainage systems to a high level of internal system detail. An example is the stormwater management model (SWMM), described by Huber *et al.* (1975), used in urban hydrology and drainage hydraulics. The strength of these models lies in their ability to take into account the interaction of a reasonable number of hydrologic and hydraulic processes and to report on the status of certain important variables within these processes throughout the simulation. Of course, there would be little point in using such a program if the system under study were simple enough to be modelled with the use of less expensive calculating machines than the main-frames still commonly used. The main benefit in using such a complex model therefore appears to be that many, if not all, of the relevant processes and their interactions can be sufficiently accurately investigated, at a large number of points in the study area, to allow the system ultimately to be sufficiently well understood by the engineers responsible for the design. Thus, programs requiring main-frame support should provide a thorough evaluation of reasonable alternative designs for complex systems. In the next few years many of these programs will be adapted for use on mini- and microcomputers (Patry 1979; Thompson and Sykes 1979).

It is evident that the next decade will see continuing advances in software development, especially applications programs. For example, recent studies (Chung and Bowers 1977; Huber and Heaney 1979; Hinson and Basta 1979; Croley 1977) list several hundred computer programs currently available for solving problems associated with water resources development. These programs will require guidelines for evaluation of their applicability and reliability.

While most of the popular programs are being enhanced to better describe constituent processes, or to include more processes, other programs describe some of these constituent processes in greater detail. Hence the problem that now arises is the selection of the correct sequence of systems and process models to be used in a study in order to best represent the physical system being analysed. Similarly, if a program adopted is very complex, it may not be clear which processes may be safely ignored, or that the data set used is the best of many possibilities. It may be better to use one of the system models that take into account similar processes, but do so inaccurately. Evidently few guidelines exist for this model selection process. On the other hand, as a client organization gains experience with one model, there is a strong tendency to prefer studies based on that same model. Problems such as these, which occur in the field of urban municipal drainage, have been reviewed in a recent paper (McPherson 1979).

The engineers conducting the study should attempt to show why selected models are deficient or sufficiently accurate for their purposes in the light of the problem review and special hydrologic and hydraulic processes that are required for their particular study. The purpose here is to show how models may be used to support one another, or to establish a reasonable sequence of model use in the study.

There is a large amount of published literature and data on the structure and performance of various models. In water resources, see for example Colyer and Pethick (1977), Huber and Heaney (1979), and Hinson and Basta (1979). Hopefully, a simple review of the documentation will indicate the accuracy with which selected, required important processes are modelled. In their published guidelines, the Association of Professional Engineers of Ontario (APEO) has stressed the fact that program documentation should provide information on versions, theory, and software to prospective users (APEO 1977).

Most tried and tested programs are considerably enhanced each year. For example, the latest versions of ILLUDAS (Terstriep and Stall 1974), SWMM (Huber *et al.* 1977), and HSPF (Johanson *et al.* 1980) may operate in the continuous mode, including water quality processes. Since these programs have been under development for ten years or more, the precise stage of evolution of the program must be correctly identified when the program is used. Many consultants and network computer centres maintain their own versions of large program packages such as SWMM. Often these versions incorporate substantial modifications, usually aimed at core reduction, improved turn-around, simplified data preparation, or more relevant output interpretation. Program documentation seldom matches the capabilities of the program at any point in time; consequently it becomes important that the actual program capabilities are accurately and carefully recorded.

Modelling effort

A certain amount of time should be invested in searching out existing data. It is much simpler to seek out and review the available data than to collect new data from scratch. Such data could form part of the model selection criteria; to make maximum use of available data the study should not be constrained to use a model unsuited to these (unknown) existing data. For example, water levels could be directly computed and plotted rather than tediously converting these data to hydrographs.

Accuracy is of overriding importance. If the required accuracy can be established early in the study, it would make the selection of the models and determination of the level of discretization for the model study an easier task. In a sense the accuracy level predetermines the type of model to be used. There seems to be a mismatch between the accuracy of the results demanded from the computer and those obtainable from field observation or laboratory analysis. Both the systematic and the random sources of error should be investigated, as part of the selection process, for both the model (concepts, solution methods, computational accuracy, input data, etc.) and the field equipment. There is little point in producing simulation results for validation that are many times more accurate than the observations which are used to validate the model. Consider for example the accuracy of rain gauge sampling of moving storms, and the selection of design storms. High accuracy may not be necessary in view of the often tenuous relationship between the study objectives and the performance criteria.

The study resources are limited to the available manpower, time, and money; the management of a study is an attempt to maximize the level of detail subject to these constraints. Collection of field data is a primary drain of these resources. The field program must be dovetailed to complement the modelling effort. The key is sensitivity analysis. Model selection criteria should be clearly stated in the study report. It is easy to criticize a study at a later date for not using a model which would describe processes to a much higher level of system detail if it is not made clear that the simpler model was used because of too little expertise, time, or money to meet the effort required for higher accuracy or detail.

Of course, in many cases the client may not have the time to study the results of these extra requirements, especially if they result in voluminous reports. In acquiring design services from a reputable engineer, the client expects the study to be carried out properly, as indeed is almost always the case. The activities listed herein, then, apply more to those cases where the programs used are complex or not well known. Hopefully engineers familiar with modelling will naturally carry out many of these activities automatically.

Special steps to establish model credibility

A series of verification, validation, and sensitivity tests can be designed to produce sufficient information to answer a wide range of questions about a model's performance. Evidently such tests will not materially increase the workload for the consultant, nor will it greatly increase the cost of the study. The tests should also help to detect certain types of error. This is probably the most important point of this paper.

Verification tests

Verification tests use some specific conditions for which the model response can be exactly predicted to check if indeed the model has been structured and coded as intended. Verification tests are not conducted by comparison of model responses with those of the actual system to be modelled; rather, comparisons between model responses and theoretically anticipated results are made in as many cases as possible. The input data need not be physically reasonable.

Of course it will not normally be necessary for the design group to verify the same model in the same way for every design study. Summary information presented in the program documentation will normally be sufficient for subsequent studies involving essentially the same model and the same processes. Verification must be done at least once on receipt of the model, ahead of the first design application.

It would be useful to create a standard data file for

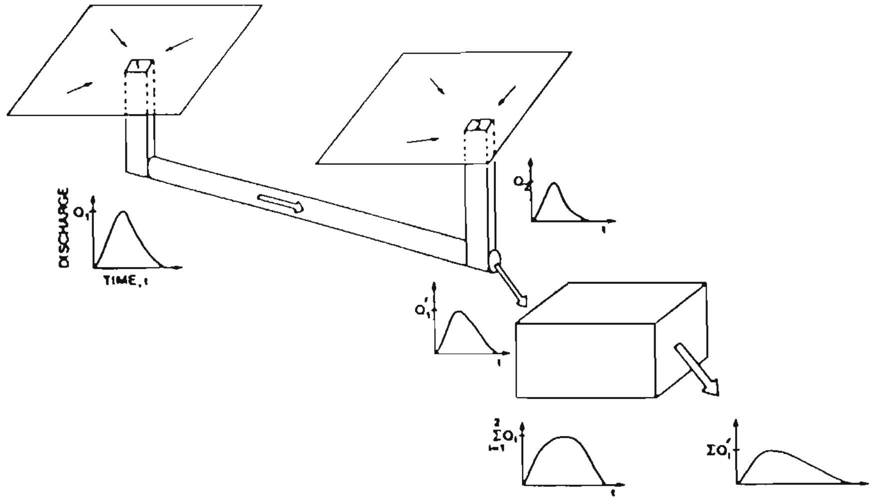

FIG. 1. Hypothetical system for model verification.

verification tests. This data set could be applied to any model put into use in an urban drainage study. For stormwater models, for example, a hypothetical system comprising two simple, square subcatchments, of say one acre (0.4 ha) each would be suitable. The subcatchments could be joined by a pipe of standard diameter and simple form. The hydrograph from the first subcatchment would be attenuated in the pipe. At the downstream end of this pipe the hydrographs from both subcatchments would be superimposed. At the outlet of the pipe the combined hydrograph could be routed through a simple, standard, perhaps rectangular, storage tank. A proposed system for model verification is illustrated in Fig. 1. The purpose of such a verification data set would be to test the algorithms for:

(1) the generation of the overland flow hydrograph;
(2) routing in the pipes;
(3) the superposition of two hydrographs; and
(4) storage routing in storage tanks.

Each of the above could be examined in more detail using the following tests:

(1) zero rain to ensure that runoff is not generated artificially within the model;

(2) very steep catchments to ensure that the hydrographs generated are very similar to the input hyetograph;

(3) light rain and high infiltration rates on low percent impervious areas to ensure that no runoff occurs;

(4) completely impervious catchments to ensure that the total volume of runoff is equal to the total volume of rainfall applied to the catchment;

(5) low continuous infiltration rates and high continuous rainfall to ensure that the correct volume of infiltration is subtracted;

(6) very flat pipe gradients to check surcharge calculations;

(7) similar tests with small diameter pipes and high pipe roughness;

(8) high and low initial abstractions; and

(9) tests on the storage routing parameters, viz. the outlet rating curve and the storage curve.

In all verification tests particular attention is paid to the quantitative summary output: for example, total precipitation, total gutter flow, total snowmelt, total infiltration, etc. This is an essential check to ensure that the numbers generated by the computer are sensible. The user must check the results by hand and categorically demonstrate that the results are sufficiently accurate.

A special algorithm and data file could be built into a program such that when the "auto-verify" option is requested the model automatically carries out a series of verification tests.

Calibration and validation tests

Calibration implies the comparison of model simulation results to field measurements, to another model known to be accurate, or to some other adequate criterion to ensure that the model of the system is producing accurate information. If these comparisons indicate that the model results are not sufficiently accurate, the model of the system is altered, usually by adjusting one or more program parameters, and the procedure is repeated. This process generally involves several iterations before a satisfactory confidence level is achieved. Techniques used in calibration include:

(1) comparison of results against field observations;

(2) cross-correlation of continuous model results with those of another proven (usually discrete event or process) model; and

(3) some combination of field observations and modelling.

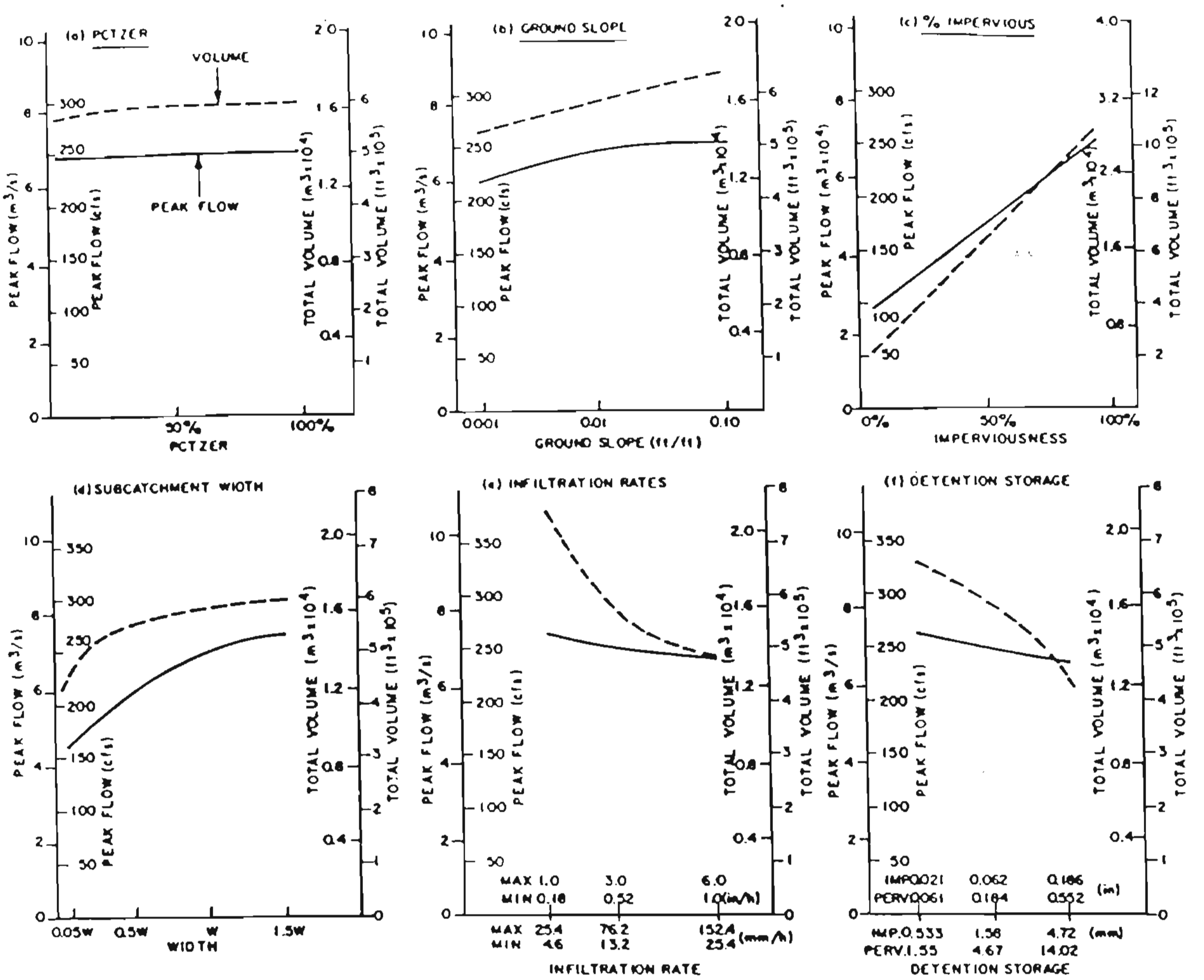

FIG. 2. Sensitivity analysis of SWMM—RUNOFF, flow.

Validation implies testing the model of the system with a data set not used in the calibration procedure. The most accurate method of validation is the comparison of output from the calibrated model against a corresponding set of independent field measurements. Tests which can be used in assessing the degree of fit of the system model to the physical system are outlined in the literature (Jacoby and Kowalik 1980; Garrick *et al.* 1978; Overton and Meadows 1976).

The calibration and validation process should be limited to the most important or sensitive parameters, and to the range of parameter values applicable to the normal operating conditions of the system. First, acceptable tolerances must be established. They should be related to achieveable field observations and to the accuracy of the field equipment. The degree of fit must take into account the errors associated with field measurements. Then, emphasis should be placed upon those critical parameters that have the greatest effect on the performance of the system model. Reasonable assumptions may be satisfactory for less critical parameters. Hence, calibration tests are closely related to sensitivity analysis, discussed subsequently.

Once verified, calibrated, and validated, the model can be applied with confidence to the evaluation of the real system. A recent paper describes the process for combined quantity and quality modelling (Jewell *et al.* 1978).

Level of discretization

The procedure for systematic disaggregation has been described in an earlier paper (James 1972). Disaggregation implies modelling more subspaces of smaller size using a finer time step. For coupling the time increment to the size of the subspatial elements, the concept of an impulse response function is useful, e.g., an instantaneous unit hydrograph. It may be accurate enough for our purposes to represent this response function by a time vector of 20 elements. This effectively sets the time step to 1/20 of the time base of the impulse

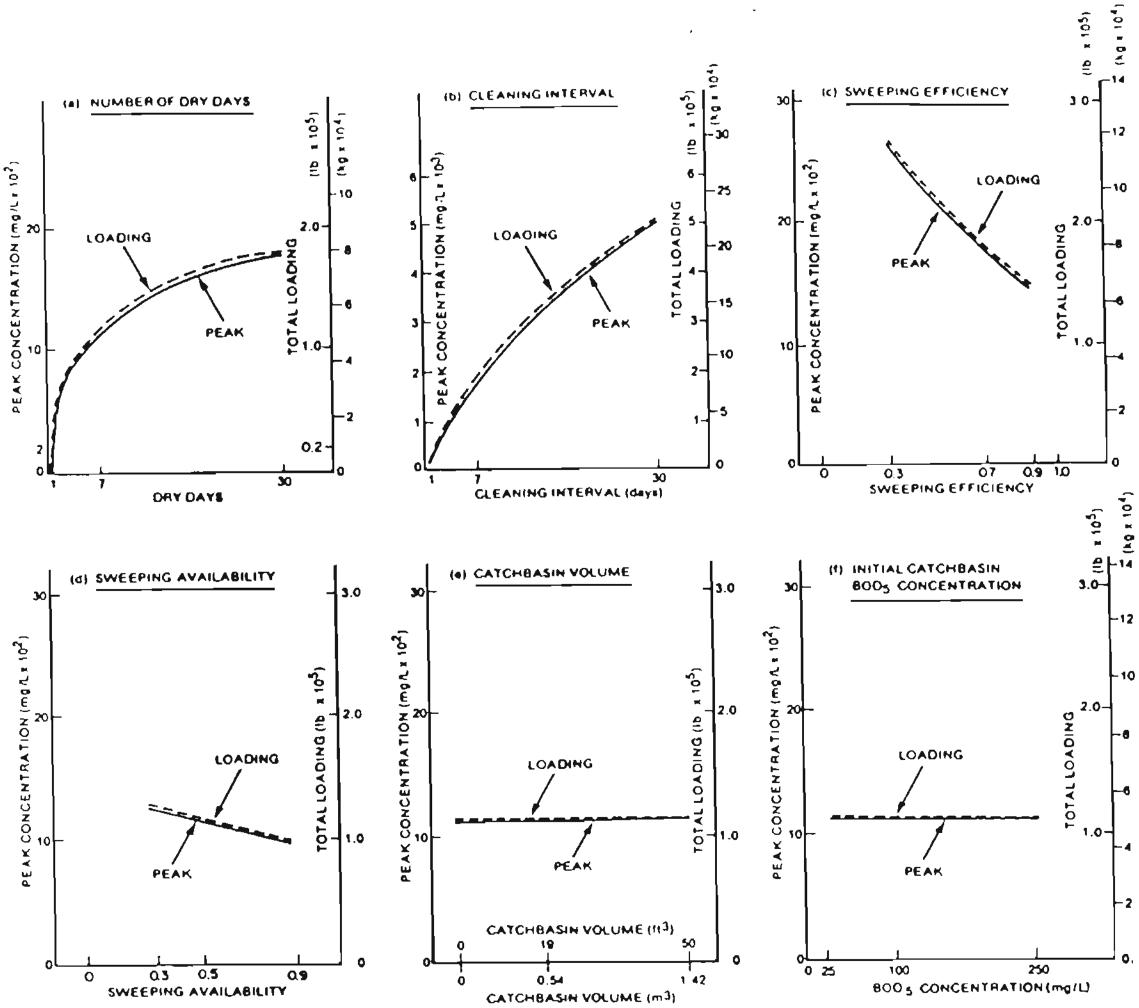

FIG. 3. Sensitivity analysis of SWMM–RUNOFF, suspended solids.

response functions. Sensitivity tests in which the time step is systematically changed may also be appropriate. In practice, disaggregation and sensitivity analysis proceed simultaneously (James 1972). Often, careful disaggregation and appropriate selection of the time step will produce better results than extensive optimization of empirical factors such as Manning's roughness for pipes and overland flow. A recent paper (Alley and Veenhuis 1979) provides some guidelines.

There is an obvious conflict in the selection of the level of discretization. Consultants' costs increase rather rapidly with an increasing number of subspaces. More data has to be abstracted and prepared, and more expensive computer runs will result. On the other hand, the client will usually prefer to have a high level of discretization for the agreed fixed study price. It is useful to have, at the outset of a study, an indication of the desirable level. As a guide, it is important to identify all significant elements in the system. These represent locations for which it is necessary to generate a response surface, such as hydrographs at all diversion structures and outfalls in hydrological studies. If these elements can be identified at the time the terms of reference are set out, consultants will have a better idea of the scope of the work. It makes sense to face this possible conflict squarely at the beginning of the study.

Sensitivity analysis

Sensitivity analysis proceeds by holding all parameters but one constant at their expected values, and perturbing that parameter within reasonable expected limits such that the variation of the objective function can be examined. If apparently small perturbations of the parameter produce large changes in the objective function, the system is said to be sensitive to that parameter. The user must obtain a measure of how accurately that parameter must be represented in his model. If the objective function is not sensitive to the perturbed

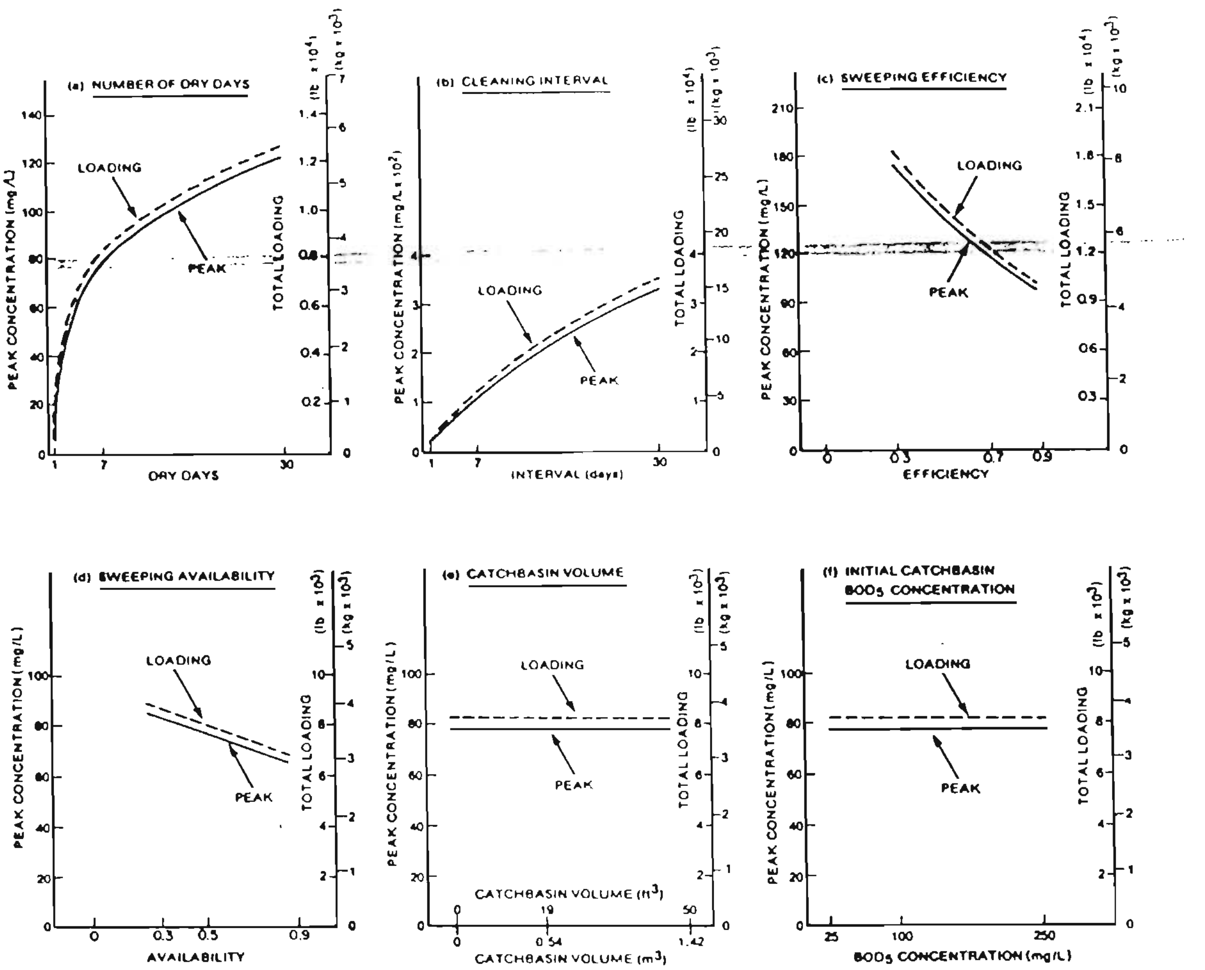

FIG. 4. Sensitivity analysis of SWMM–RUNOFF, BOD_5.

parameter, then the parameter need not be accurately represented. If the system is insensitive to the perturbed parameter, the parameter and its associated process is redundant and the process should be deleted. The tests are done using the full design problem input data set. It must be stressed that the actual values of the constant parameters may affect the sensitivity analysis and so their values should be typical of the conditions being modelled.

As part of a study on the Hamilton urban drainage system (James 1980) sensitivity analyses were made on the quantity and quality models of the RUNOFF block of the SWMM. In carrying out the sensitivity analysis for the quantity algorithm a system of 11 subcatchments comprising a total area of 265 acres (107 ha) and containing a network of 15 flow conduits was used. The subcatchments ranged in area from 10–34 acres (4–14 ha). The conduits ranged in diameter from 2.0–10.8 ft (0.61–3.29 m). Results were based on a 120 min rainfall with a total volume of 1.2 in (30 mm) and a simulation time of 200 min. The results of the analyses are presented in Fig. 2 and apply at the outlet of the system. Note that the results are presented in the same units used by the SWMM program. Scales for SI units are also given on the figures.

In carrying out the sensitivity analysis for the quality section of the model a system of four subcatchments was used. Each subcatchment was assigned one of the following land use classifications: single family residential, multi-family residential, commercial, or industrial. The pollutographs were totalled at the downstream end of the system; they were not routed through conduits. A synthetic rainfall of 0.5 in (13 mm) distributed evenly over 2 h was used for the simulation. The results of the analyses for suspended solids and BOD_5 are presented in Figs. 3 and 4 and apply at the outlet of the system.

The results presented in Figs. 2–4 may be briefly discussed as follows. The quantity and quality model parameters may be listed in order of decreasing significance in terms of sensititivy for the Hamilton urban drainage system:

Quantity
(1) Percentage imperviousness
(2) Width of subcatchments
(3) Initial and final infiltration
(4) Depression storage for both pervious and impervious areas
(5) Ground slope
(6) Percentage of impervious area with zero detention storage
Quality
(1) Number of dry days
(2) Street sweeping interval
(3) Street sweeping efficiency
(4) Exponential coefficient in the washoff equation
(5) Dust and dirt loadings
(6) Insoluble fractions due to suspended solids
(7) Availability factors for suspended and settleable solids
(8) Total gutter length

This rank order may not apply in another study area; sensitivity analysis is desirable for each study application.

Here again algorithms should be available within the program that permit the user to easily conduct a sensitivity analysis. When this "auto-sensitivity" option is selected, the program requests the user to identify the parameter whose sensitivity is to be tested, and the range of perturbed values. The system data file is then automatically rebuilt and the tests carried out. All output response functions, such as hydrographs, should be plotted on the same family of curves in order to present the impact immediately to the user. It is critically important to rank environmental parameters that affect the important components of your response function for your model in this way.

Control of errors

Having carried out the required verification, calibration, validation, and sensitivity analyses at various levels of discretization, the results have to be presented in the report in a way which ensures that the purposes of the tests have been achieved. The verification tests must be shown to produce expected results, thereby demonstrating that there are no errors in the coding of the program. The calibration and validation results must be shown to be reasonably accurate. The trends resulting from the sensitivity analysis must be shown to make sense in the light of the actual design problem, and will indicate whether the level of effort put into estimating the individual parameters is appropriate based on their significance in affecting model results.

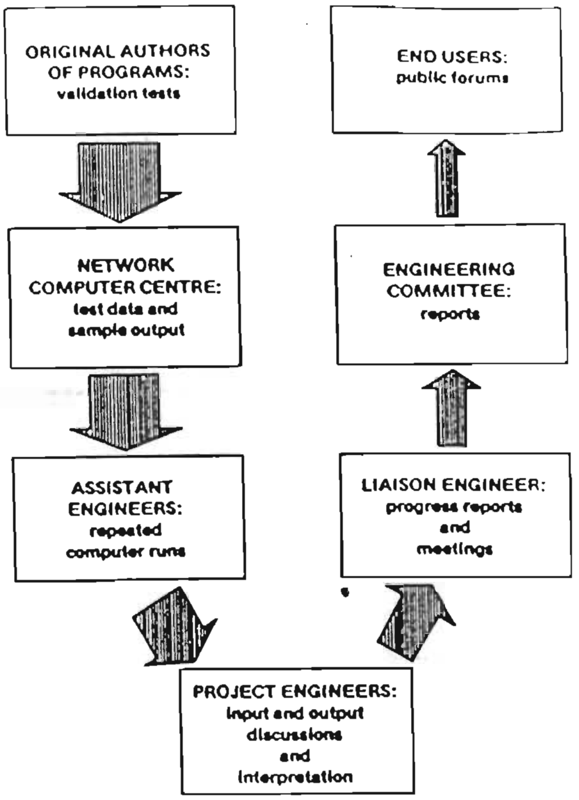

FIG. 5. Flow of model credibility.

Conclusions

In our work we have tried to establish controls that ensure that the best model and program are used correctly. Many of these control measures amplify the guidelines for using computer programs published by the Association of Professional Engineers of Ontario (APEO) (1977). The aim of these measures is to ensure that credibility and confidence in the results is created and transmitted sequentially from the design office to the client; for example, from the junior engineers to the project engineer, perhaps in the consultant's office, and thereafter to the municipal engineers responsible for supervising the study, and ultimately through the engineering committees and other political bodies to the beneficiaries, frequently the public. The process is depicted schematically in Fig. 5. In other words, it is not sufficient that the computer model represent all relevant physical processes of the study accurately enough, or that the results be correct or sufficiently accurate. It is important to assure that the study be carried out in such a way that there is little chance of using a wrong model, or using wrong data, or wrongly interpreting the results, or simply of not understanding the model or its relationship to the design objectives.

We argue herein that certain precautions be taken at the outset of a computer-based study. We recommend that the study terms of reference include the following topics:

Problem review—The problem review should identify all significant elements in the study area so that the model selected can be shown to include all relevant processes, and the model application to be disaggregated to the correct degree.

Study objectives—The study objectives should be reviewed to show how the objective functions, such as pollutographs and/or hydrographs, relate to the design alternatives. New problems may become apparent during the study so it must be possible to redefine the objectives and all subsequent deductions.

Performance criteria—The performance criteria for the comparison of one design alternative with another include peak response, total loadings, time to peak, cross-correlations, etc., and must be correctly identified so that the simplest modelling can be justifiably used to select the best design alternative.

Requisite accuracy—Accuracy of field measurements for validation should be carefully reviewed in order to ensure that the modelling effort coincides in detail with the data available for comparison. Systematic and random errors should be defined.

Review of available programs—Several programs should be suggested or selected for review. The review should consider process models as well as system models, and an appropriate sequence of models. Study resources include time, manpower, money, and information, and these, in turn, will determine which of the models may be selected.

Model selection criteria—All available data must be reviewed. Models should be selected for which expertise is available and ample data easily collected considering the limitations imposed by the study resources available.

Model verification—Verification tests on the model should be required using a standard data set consisting of simplified elements chosen to test each process of interest. The verification tests should check the summary output and demonstrate that the coding is performing as intended.

Model calibration and validation—Calibration tests should be carried out on one or more of the subspatial data sets using specified parameters to demonstrate that the model is being correctly used and that an accurate representation of the system has been achieved. Validation tests should be carried out on an independent data set to demonstrate that the system model is reasonable.

Sensitivity analysis—Sensitivity analyses should be carried out on a minimum number of parameters, for example, infiltration parameters, roughness values, widths of subcatchments, etc., to identify those of greatest significance and to justify the effort put into their estimation.

Minimum level of discretization—The smallest number of subspaces required for modelling the system should be selected commensurate with the objectives of achieving the best design at a reasonable cost. These minimum levels should correspond to the disaggregation necessary to identify response surfaces only at all the significant elements in the system, which should be specified at the outset, so far as possible.

Data preparation and output interpretation—All input and output should be interpreted to demonstrate that the model is performing in a logical way. The magnitude and direction of errors should be estimated.

Documentation—The version of the program actually used in the study should be identified and appropriate documentation sources listed in the report. In addition, the machine readable input and output files should be listed and archived for future use.

The delivery of computer based engineering design services could be improved by enlightened study terms of reference and attentive control by the responsible municipal or local authority engineers. The suggestions in this paper should reduce the chances of inexperienced engineers using an inappropriate model, inappropriately using an otherwise suitable model through erroneous data preparation or output interpretation, and remaining ignorant of the model even when the results are acceptable. Credibility would thus be improved.

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A program package for interactive design of optimal pipe networks for any climatic region in Canada

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A program package for interactive design of optimal pipe networks for any climatic region in Canada

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The program package presented in this paper facilitates the computer-aided design of least-cost pipe distribution networks for any region in Canada. The design algorithms function in an interactive, conversational mode. Three algorithms—for analysing pressure and flow in networks, for computing the 'best' diameter of each pipe in the network, and for computing the best sizes of devices and materials to protect the pipes against freezing—were combined. Here 'best' is taken to be the least equivalent average annual cost.

The pipe distribution network may include a wide variety of pipeline components and environmental constraints. The variables that determine the cost of pipe distribution networks in the extreme climatic regions of Canada are identified and their interaction for the many combinations of environmental, economic, and technological constraints are described.

The computer program package was employed in the design of a minimum-cost distribution system for Broughton Island, Northwest Territories. The performance of the package indicated that it could be reliable, easy to use, and competitive in design cost.

The package is locally accessible through a national computer network to dial-up terminals in or near most cities in North America.

L'ensemble de programmes décrit dans cet article est destiné à faciliter, par le biais de l'ordinateur, le calcul de réseaux de conduites de distribution de coût minimal pour n'importe quelle région du Canada. Les algorithmes de calcul mettent en jeu un mode dialogué. On a combiné trois algorithmes: un pour l'étude des pressions et des débits dans les réseaux, un autre pour le calcul du diamètre 'optimal' de chaque conduite d'un réseau, et un troisième pour le calcul des dispositions optimales devant assurer la protection des conduites contre le gel. Le mot 'optimal' désigne ici ce qui entraîne le coût annuel moyen équivalent le plus bas.

Le réseau de conduites de distribution peut comporter un large éventail de composantes de pipe-lines et devoir satisfaire à des contraintes environnementales. On identifie d'abord les variables qui déterminent le coût des réseaux de conduites de distribution dans les régions canadiennes aux conditions climatiques extrêmes; puis on décrit leur interaction sous différentes combinaisons de contraintes environnementales, économiques et technologiques.

L'ensemble de programmes de calcul électronique a été appliqué à l'établissement du projet d'un réseau de distribution de coût minimal pour l'île Broughton, dans les Territoires du Nord-Ouest. Cet ensemble informatisé s'est révélé fiable, d'exploitation facile et d'un coût compétitif. Il est accessible au moyen de terminaux téléphoniques reliant la plupart des villes d'Amérique du Nord à un réseau national d'ordinateurs.

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Introduction

The design of pipe distribution systems must take into account technological, environmental, and economic constraints: intense cold for extended periods, permafrost, space limitations, intermittent flows, a variety of special pipeline components, material availability, high energy costs, and a wide range of interest rates. The environmental and economic constraints can potentially exist in a wide range of combinations and when they are applied to a system of pipes and pipeline components a unique system cost is produced. For a given set of environmental and economic factors only one combination of network components exists for which the system cost is minimum.

Traditionally, the only criteria kept in mind when designing pipe water distribution networks were the service requirements. This meant delivering a specified flow rate, usually sufficient for firefighting, within an acceptable pressure range, to all points in the area being served by the distribution system. Classical methods used to solve the simultaneous equations involved in the hydraulic analysis of the distribution system included trial and error solutions,

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nomographic solutions, and the Hardy Cross method.

The final design of the distribution system, including pipe sizes and pump capacities, was rarely, if ever, the optimum (i.e., least total cost, capital and operating). This is especially true where existing systems are gradually expanded. Camp (1952) recommended that designing a least-cost system was "unnecessary and unwarranted because relatively large variations in pipe size in the immediate range of the economic size change the total cost but slightly. ..."

With the high-speed computational facilities currently available it is possible to accurately analyse a network, subject to a given set of nodal demands, in a matter of seconds. Thus, a layout of pipes that would satisfy operating requirements could be designed in a day or two. As the demand for modernization and expansion of existing networks grew designers became more aware of the necessity for optimization of the supply systems. Large distribution networks require considerable investments of capital to construct. Designers now realize that ill-designed networks not only increase initial investments but also lead to excessive operating costs.

As computational techniques have advanced, a number of highly sophisticated programs have been developed to aid the engineer resolve the problem of finding the optimum supply network. Although the majority of these programs are computationally efficient they are generally user inefficient, requiring the user to have direct batch access to a computer. In most cases the programs require a significant amount of input data, which must be abstracted, coded, and transported to and from the design office each time the data are to be debugged or the program run. For an optimal design to be found a considerable investment of the engineer's time is necessary.

Use of programs that operate in a time-sharing, interactive mode offers a solution to the problem of long design turnaround times inherent in batch mode computing. A program that operates interactively provides the user with immediate results; he can make immediate decisions and supply the additional input to complete the design, usually resulting in considerable savings of time and effort.

The advantages of interactive computing become evident when one considers the design of water distribution systems for arctic communities. A supposedly finalized design may require modification during construction. For example, a shipment of pipes may arrive at the site with shortages and excesses due to shipping error. Because of time limitations imposed by the harsh environment, construction cannot be delayed while the order is resubmitted. The engineer supervising construction need only dial the proper telephone number, affix the receiver to a portable terminal, and he is able to carry out a new design incorporating the modifications made necessary by the over- and undersupply. The delay in construction is negligible yet the engineer can ensure that the system as constructed will still meet service requirements and be highly cost effective.

In this study a variety of algorithms that, operating individually, can aid in the design of certain water distribution networks, are combined and upgraded. When integrated with interactive conversational software they produce a fast, efficient, easy-to-use package capable of computing an optimum design for a pipe distribution system for any climatic region in Canada.

Review of the Algorithms used for the Design of Pipe Distribution Networks

In the southern regions of Canada current practice is to bury the pipelines sufficiently deep to prevent freezing during the winter. This type of system requires no special protection for use in winter. However, the situation is quite different in the permafrost regions of Canada. Pipelines cannot be economically buried deeper than a few feet, where the ground is permanently frozen. It becomes necessary to supplement the conventional water distribution system with devices and materials that protect the pipelines from freezing and subsequent rupture during the winter months.

Until now, one design procedure has not dealt with all situations. In order to construct a general design package for water distribution systems in Canada it is necessary to provide not only an algorithm for computing the optimum layout of pipes, but also one for computing the optimal sizes for devices and materials providing thermal protection.

Common to both algorithms is the network analysis. An efficient, reliable algorithm capable of handling a variety of pipeline components is essential to a general design procedure. These three main algorithms, necessary for creating a design procedure that is applicable throughout Canada, are described subsequently.

Pressure and Flow Analysis in Pipe Distribution Systems

Programs available for the analysis of pressure and flow in pipe networks range from the most basic trial and error procedures to complex programs (Chandrashekar and Stewart 1975; Donachie 1973) involving the solution of the set of nonlinear algebraic equations describing the pipe, pump, and reservoir characteristics that specify the steady-state operation of the system. These equations are solved using a numerical technique such as Newton's method. The method will converge to a solution but a large computer memory is required and computing time can be several minutes, long enough to be quite expensive if many trials are made.

The program adopted for use in this study was originally developed in the Department of Civil Engineering at the University of Kentucky. The procedure used is an extension of the Hardy Cross method, which allows inclusion of a variety of pipeline components, described by nonlinear equations, without requiring a great deal of computer memory or time. For any pipe system composed of a number of junctions J, terminal energy points TE, primary loops PL, and pipes P, the following relationship always holds:

$$[1] \quad P = J + PL + TE - 1$$

For each junction a continuity equation can be written:

$$[2] \quad Q_{in} = Q_{out} \qquad (J \text{ equations})$$

For each loop the energy equation is written:

$$[3] \quad \sum_{i=1}^{n} h_{Li} = \sum_{i=1}^{n} E_{Pi} \, (PL \text{ equations})$$

If there are no pumps in the loop then [3] states that the sum of head loss around the loop is zero. If there are TE terminal energy points, $TE - 1$ energy equations can be written for paths between any two terminal energy points:

$$[4] \quad \Delta E = \sum_{i=1}^{n} h_{Li} - \sum_{i=1}^{n} E_{Pi}$$

Equations [2], [3], and [4] constitute a set of simultaneous nonlinear equations that, when solved, yield a value of flow rate in each pipe. In order to handle the nonlinear head loss and pump terms a linearization procedure is used to set up simultaneous linear equations that may be solved by one of a number of available matrix methods.

The algorithm developed to perform these calculations has a number of advantages over most traditional Hardy Cross methods and over many of the more sophisticated solutions of the full set of nonlinear simultaneous equations: (i) the algorithm can solve any system configuration (looping or branching); (ii) a variety of network components such as pumps, reservoirs, and check valves can be dealt with without altering the complexity of the equations to be solved; (iii) the reduction of the nonlinear set of equations to a set of simultaneous linear equations ensures convergence to a solution in all cases; and (iv) the facility has been provided for operating in various English units or in metric units, at the user's discretion.

Optimum Design of Pipe Networks

Use is made of a collection of algorithms developed by a Danish firm of consulting engineers, Nielsen & Rauschenberger, for optimizing water distribution networks (Rasmusen 1976). The algorithms pertinent to the optimization procedure have been abstracted and integrated with the previously described network analysis algorithm.

In general, because flows through pipes in a network are highly dependent on one another, an optimal size for a pipeline cannot be computed separately from the rest of the network.

Each pipe in the network connects a pair of nodes and has a known length L, diameter D, and roughness coefficient R. Each node in the network is associated with a demand $Q_j \geq 0 \, (j = 1, ..., J)$, a required minimal service pressure $H_j^{min} > 0$, and an actual service pressure $H_j \geq H_j^{min}$. Each source of supply has associated with it a fixed flowrate $Q_k \leq 0 \, (k = 1, ..., S)$ and a natural pressure H_k^{nat}. Note that by this sign convention abstraction is positive and supply is negative.

The costs of the network can be expressed as the sum of the initial investment in pipe material and the cost of operating the pumps over the expected service life. The assumption is made that all pipeline costs can be expressed as a capitalized present value that is a function of the pipe diameter only:

$$CP_i = \Psi(D_i)$$

Similarly the recurring energy costs resulting from the operation of the pumps can be expressed as a capitalized present value E_k representing the cost of lifting a unit volume of water a unit length per second continuously over the service life of the system. For a given set of nodal demands the least-cost layout may be found by minimizing the following objective function with respect to diameter:

$$[5] \quad \text{cost} = \sum_{i=1}^{P} CP_i L_i + \sum_{k=1}^{S} Q_k E_k (H_k - H_k^{nat})$$

subject to the following constraints.

(i) Continuity, i.e., the abstraction from node j must be equal to the difference between all flow into and out of node j:

$$[6] \quad Q_j = \sum_{i=1}^{T} Q_{ij} - \sum_{i=1}^{F} Q_{ij}$$

where T = total number of pipes carrying flow into j, and F = total number of pipes carrying flow out of j.

(ii) The minimum pressure requirements:

$$[7] \quad H_j \geq H_j^{\min} \qquad (j = 1, ..., J)$$

(iii) The conditions at the source of supply:

$$[8] \quad H_k^{\text{nat}} \leq H_k \qquad (k = 1, ..., S)$$

where H_k^{nat} = the natural pressure on the pump intake.

(iv) The physical relationship describing the flow in a pipe:

$$[9] \quad Q = \Phi(D, L, R, H_j, H_{j'})$$

where j represents the upstream node on the pipe, and j' represents the downstream node on the pipe.

For example, using the Hazen-Williams equation:

$$[10] \quad Q = 1.318C_H\left(\frac{\pi}{4}D^2\right)\left(\frac{D}{4}\right)^{0.63}\left(\frac{H_j - H_{j'}}{L}\right)^{0.54}$$

Equation [9] is nonlinear. Pipeline costs are generally also nonlinear. However, the complexity of the problem is reduced somewhat by realizing that only diameters having standard manufacturer's dimensions need be considered.

The problem breaks down into two distinct components: (i) an hydraulic network analysis problem that, given any fixed system and set of nodal demands, can be solved; and (ii) a problem in modifying diameters towards an optimal solution. This implies an iterative solution where diameters are set, the hydraulic analysis carried out, and diameters modified towards an optimal solution. The optimum layout is arrived at by computing a series of gradually improving feasible solutions.

In carrying out several designs it was found that the optimization procedure gave layouts that had a smaller total cost than the initial assumed layout provided that the assumed layout was in the vicinity of the optimum layout. When the assumed layout was not in proximity to the optimum the procedure often failed to converge to the optimum layout. In this case error messages are printed at the user terminal.

Optimum Design of Materials and Devices for Protecting Utilities Against Freezing

Algorithms incorporating the concepts of network analysis and network optimization discussed in the previous sections would be sufficient to compute an optimal layout for a water distribution system that would function in the southern regions of Canada. However, in order to generalize the design procedure, provision must be made for computing the optimum sizes of devices and materials to prevent pipe freezing. This would be necessary for buried systems that are to operate in the permafrost regions of Canada, and for all above-ground systems operating in the winter.

The pulsed supply (intermittent pumping) system for water distribution has been shown, in a variety of reports (Grainge 1965; Heinke 1974; Suk 1975), to eliminate a number of the disadvantages inherent in other systems. Use is made of small diameter pipes, in the range of ¼–3 in. (6.4–76 mm), to convey the water. Small diameters are all that is necessary because the volume of water being distributed is not great, owing to the small population in each community. The pipelines are fitted with heat tracing tape and are insulated. The pipes are laid in a shallow trench (up to 2 ft (0.6 m) deep) and fill is placed over them to protect against physical damage from vehicles.

The pulsed supply system operates on a well-defined cycle each time a pulse of water is to be delivered to the dwellings in the community. Initially the pipes contain a small amount of residual water, not in sufficient quantities to damage the pipes when frozen however. Heat, in the form of electrical current, is supplied to the surface of the pipes through the heat tracing tape until the pipe surface reaches a temperature slightly greater than 32°F (0°C). Water is then pumped, under pressure, through the system filling storage tanks housed in each dwelling. When all the tanks have been filled the pump is shut down and the air compressed in the house tanks blows through the system forcing out the remaining water. Valves at the drainage points in the system are opened after the pumping ceases. Once most of the remaining water has been drained the power is shut off and the empty pipe is left untouched, yet in no danger of freezing, until it becomes necessary to refill the tanks.

There are several notable advantages of the pulsed supply system. It provides an adequate supply of clean water. The water is not touched by hand until it has been delivered to the tanks; hence contamination enroute cannot occur. The shallow burial of the pipes is inexpensive, keeps the community's transportation system uninterrupted, and allows easy access for repair and maintenance. Because the system does not operate continously running costs are less than for a utilidor system (where all utilities are carried in a heated duct above the ground). The design of an optimum thermally protected network requires optimization of the following: (i) thickness of insulation; (ii) capacity of the heat tracing tape; and (iii) warm-up time for the system. The three decision variables are interrelated such that altering the dimension of one necessitates recalculation of the other two to ensure that in-service requirements are met. Heat tracing tape with a higher capacity allows the system to be brought up to the operating temperature in less time, which results in a shorter period of time over which heat is lost. High wattage heat tracing tape draws more power for a shorter period of time than low wattage tape; however, low wattage tape requires a smaller initial investment of capital. The optimum design requires a trade-off between the two extremes. Thick insulation requires more heat and hence more power be input in order to bring the system up to the system operating temperature; however, thick insulation retards heat loss to a much greater extent than does thin insulation. Once again a balance between the two extremes must be sought. Finally, the selection of heat tracing tape capacity and insulation thickness requires that the whole network reach the desired system operating temperature within the optimal warm-up time. In order to arrive at a system producing the minimum annual cost a balance between the decision variables must be achieved.

The algorithm searches for the combination of system parameters producing the minimum total annual cost by trying all combinations of insulation thickness and design wattage for each pipe for each warm-up time. Two options have been provided in regard to warm-up time. The first option has each pipe in the network considered as being on a separate circuit. The pipes with the largest diameters requiring the longest warm-up time would be activated first, then the pipes with the next largest diameter, and so on. The activation series is timed so that all pipes reach the system operating temperature at the same time. Thus an optimum warm-up time is not computed; rather a 'dummy' value is assigned to it in order to satisfy program requirements. The second option has all pipes begin warm-up simultaneously. In this case an optimum system warm-up time is computed.

In general terms the objective function may be stated as "Minimize the sum of annual cost of borrowed capital and annual operating costs." The initial investment will be made up of the cost of the pipelines (with insulation) and the heat tracing tape. Operating costs will be made up of the cost of supplying power to the heat tracing tape and any pumps, and maintenance of the system, which is assumed to be a percentage of the total capital cost of the system (Heinke 1974). The objective function will be subject to the following constraints: (i) the rate of heat loss from the liquid in a pipe must not be greater than the wattage of the heat tracing tape for that pipe, and (ii) the time required to bring a pipe up to the operating temperature must not be greater than the assumed warm-up time. Only the current optimal set of design variables is stored. Each time an improved solution is found the previous solution is discarded. This approach to optimal sizing is novel in that optimization is based on total annual costs rather than total initial investment. This is more relevant for small communities in the arctic because the town councils often have limited budgets with which to work. A system that has been optimized on the basis of annual cost will provide them with an estimate of the cost to the town yearly and with it they can more effectively decide if the system can be afforded.

The algorithm is somewhat inefficient in that the cost of every combination of decision variables must be computed and checked. However, there is difficulty in finding the optimal solution by other methods because only discrete sizes are available for each material or device.

Development of the Design Package

Design of an effective computer-aided design package is a three-stage process. The first stage involves design of the overall procedure, i.e., methodology, for solving the problem under consideration. The second stage involves design of an effective man–machine dialogue. The third stage involves design of the details of the computer program, i.e., a series of FORTRAN statements to carry out the first two stages. Combining the end products of the three design stages results in a package that should generally find an acceptable solution to the problem specified by a set of input data defined by the user.

Design of the Interactive Software

A computer uses information input to compute results, which are returned to the user. The ease with which the user communicates with the computer will determine how much and how efficiently he uses it. It is essential to any decision-making process that results required for action be presented in a form that is meaningful, easily comprehended, and available when needed. The user must not be exposed to the great amounts of data generated by the computer yet he must be given sufficient information on which to base questions, make decisions, and supply information.

The man–machine dialogue developed by the authors to enable the program package to function in the interactive, conversational mode was designed to lead the user through a pipe distribution system design by requesting at various stages or steps specific input while still allowing him, at any point in the design, to change the sequence. The user may respond with one of the following allowable inputs or commands: (i) enter a number; (ii) enter YES or NO; and (iii) enter one of the following commands: CHANGE, STATUS, END.

The interactive dialogue has the following features.

(i) The correct method for entering data is illustrated before any prompts are displayed.

(ii) Only a small amount of information is displayed at any one time. Prompts posed are made as concise as possible without sacrificing clarity.

(iii) Responses required from the user are brief, thereby avoiding ambiguity.

(iv) The computer responds to all input. The user is made aware that the information input has been read and understood by the computer with a signal.

(v) Error detection and correction facilities implicit in the program are executed automatically to check each piece of input.

Procedure for Design of Optimum Pipe Distribution Networks

The entire design package comprises two distinct and separate programs, as follows.

(i) NETWORKOPTIMIZE—an interactive design of a cost-effective pipe distribution network operating under steady-state conditions. The total cost of the system represents the cost of pipeline material, expressed as a function of pipe diameter, plus the cost of installing and operating the pumps over the service life of the system expressed as function of the pump power. The program computes the optimal diameter for each link in the network such that the total capital cost (initial investment plus operating costs reduced to an equivalent capitalized present value) of the system is minimized.

(ii) ARCTIC—an interactive design of an optimal pulsed water supply system. Given a specified network topology and set of link diameters the program computes the optimum time required to raise the system temperature to the operating temperature and the optimum thickness of insulation and capacity of heat tracing tape to maintain the operating temperature. The optimum values are computed such that the total annual cost (repayment of borrowed capital plus annual operating costs) is minimized. Repeated use of ARCTIC with a range of pumping durations enables the user to determine an optimum duration of pumping by comparing the total annual costs of the optimum designs for the range of pumping duration.

Each program is designed to operate separately from the other; however, the manner in which the package is used is determined by the nature of the design problem to which it is to be applied. If a pipe distribution network is to be designed for a region in which the pipelines can economically be buried to protect them against freezing, the system may be designed using NETWORKOPTIMIZE only. The pipecost function is determined by the material and installation costs of bare pipe. The optimal set of link diameters is computed based on this cost function.

If a pipe distribution network is to operate in permafrost regions or regions where the possibility of pipe freezing exists and the diameter of each link has been fixed, then only program ARCTIC need be used, to determine the optimal thickness of insulation and capacity of heat tracing tape for each pipe as well as the optimum system warm-up time.

However, if a new network that is to operate in permafrost regions is to be designed completely (link diameters and thermal protecting devices) then it becomes necessary for ARCTIC and NETWORKOPTIMIZE to interact. The programs can interact in two ways. If the system warm-up time is not to be optimized then the optimum sizes of materials and devices for thermally protecting the pipes, and the associated costs, are functions of the pipe diameter alone. In this case ARCTIC is used first to determine an optimum insulation thickness and heat tracing tape capacity for a specified diameter of pipe irrespective of its length or position in the network. Then NETWORKOPTIMIZE is used, employing a pipe-cost function (comprising the unit capital cost of pipe material, insulation material, and heat tracing tape, and the annual cost of providing power to the tape, reduced to a capitalized present value) to determine the optimum set of diameters for the network.

A second mode of interaction is one in which a specified network topology and a pipe-cost function determined by bare pipe cost alone are used as input to NETWORKOPTIMIZE to determine an optimal set of diameters. Based on this optimal set ARCTIC is used to compute the optimum thickness of insulation and capacity of heat tracing tape for each pipe in the network. If an optimum system warm-up time is to be computed or spatial variations in environmental conditions exist the optimum sizes of insulation and heat tracing tape are no longer functions of diameter alone. Thus, the approach using NETWORKOPTIMIZE before ARCTIC is the correct one. If an optimum system warm-up time is not to be computed and spatial variations in environmental conditions do not exist the situation would be identical to that of the first form of interaction. However, the approach using NETWORKOPTIMIZE first and ARCTIC subsequently would still be applicable if the possibility of feedback from the unit thermal costs affecting the pipe-cost function is accounted for. Based on an initial set of diameters for the network and a cost function determined by bare pipe cost, NETWORK-OPTIMIZE computes an optimum set of diameters, which are used as input to ARCTIC. The optimum thickness of insulation and capacity of heat tracing tape are computed for each pipe. Associated with these optimum sizes are unit costs that affect the cost function used as input to NETWORKOPTIMIZE. Thus two or more iterations of the procedure must be made until the optimum set of diameters D^* converges.

Evaluation of the Design Package

Perhaps the most significant measure of a program's efficiency is the total turnaround time, i.e., the time from preparation and submission of input data to the output of the final design details. Batch mode computing requires the investment of many hours if not days to refine a design to a satisfactory solution. It has been found that by using this design package, an optimal design of a water distribution network may be found in times ranging from 15 min to 1 h, again depending on the complexity of the system. This time excludes data abstraction off original maps and documents, of course. Time is saved by having computed values transmitted directly to the terminal so that decisions can be made immediately. Delays in obtaining essential output have been eliminated, so that time spent in searching through excess batch output has been reduced to the minimum.

In this paper emphasis has been placed on designing a least-cost water distribution system. Thus, it is appropriate that the economics of interactive design be considered as well. It has been found that the cost of computing a complete design of a water distribution system for an arctic community ranges from $5 to $10 depending on the network complexity and the unit computing charges of the computer system. It was also found that making changes on line, as part of an ongoing analysis, cuts computing costs significantly. Because the design is at a more advanced stage, there is no need for reprocessing data that have already been used and are no longer required, as would be the case if the program were operating in the batch mode. The savings may amount to several dollars for each trial. Thus, if four or five trials were needed to achieve an acceptable solution using batch mode the cost of computing interactively would be covered by the savings alone.

More significant than computing cost is the cost of the designer's time. Using the program in batch mode may require 1 day's effort, depending on the designer's capability. It has been observed that savings of about 85% of turnaround time can be achieved using the interactive package. The costs of that time difference may be quite significant to consultant and client alike.

In summary, it has been the authors' experience that the design time, including user transaction time, and program turnaround time are greatly reduced when the interactive approach to computing is taken. It has been found that concise, unambiguous man–machine dialogue promotes a reduction in user transaction time, minimizes the potential for error, increases user confidence, and encourages investigation of different design situations. It has also been found that transmitting only key output to the terminal enhances convergence to a final design.

Conclusions

For the design of water distribution networks, two new programs that operate in the interactive, conversational mode were developed. They appear to be more efficient, easier to use, and less costly than similar programs operating in the batch mode. The interactive programs described in this paper can be used for carrying out a series of water distribution network designs or for evaluating the effects on a previous result of changing one or more design parameters. The package has been found in two design applications to be capable of computing an optimum pipe distribution system for a locality in the Canadian arctic as well as one in southern Ontario (in both cases for any network configuration and for a wide variety of pipeline components). The integration of interactive, conversational software with efficient programs carrying out hydraulic and economic analyses has evidently resulted in a reliable, versatile, and easy-to-use package. However, the complexity of pipe networks is of course not eliminated by the programs and users are naturally required to be competent hydraulic engineers.

The package is locally available to dial-up terminals through a commercial computer network in most cities in North America. Enquiries are encouraged and may be directed to either or both author(s).

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Nomenclature

The following is an alphabetical list of the symbols used in the text with their meaning and, where applicable, the English and metric units in which each is measured.

C_H = Hazen-Williams roughness coefficient
CP = the discounted present value cost of the pipe per unit length
D = pipe diameter (ft, m)
E = discounted present value for a source of raising a unit volume of water a unit length per second continuously over the service life of the system
E_P = energy added to the liquid by a pump (ft, m)
h_L = total head loss in a pipe (ft, m)
H = actual elevation of the pressure surface at a node (ft, m)
H^{min} = required minimal service pressure at a node
H^{nat} = pump suction pressure at a source
J = total number of junctions in the network
L = pipe length (ft, m)
n = number of pipes in a loop or path
P = total number of pipes in the network
PL = number of primary loops in the network
Q = flow rate (ft^3/s, m^3/s)
R = pipe roughness
S = total number of sources in the network
TE = total number of terminal energy points in the network
ΔE = energy difference
Φ, Ψ = functions of

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Brief Reports

Developing Simulation Models

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Abstract. Simulation models are commonly resolved in time and space beyond the integrity of available field observations. Input data are fudged accordingly, and, ultimately, expensive simulation models coexist with inadequate data bases. Such manipulation is necessary in sensitivity analysis, but a compromise between data collection and model extrapolation must be found. The rationale for the systematic development of a simulation model and the concomitant data collection program is presented in this paper and should be useful for managing difficult simulation studies.

The steps normally taken in the development of a simulation model are presented in this paper; often these steps are not taken systematically. In the absence of a logical system the management of simulation studies can degenerate into a diffuse discussion of processes, data collection, numerical problems, and finance to the extent that the reasonable development of the model is jeopardized. The rationale described in this paper was drawn up originally by the author as a formal foundation for supervising several catchment simulation studies in geography at McMaster University. The approach is similar to that used in industrial engineering [e.g., *Crowe et al.*, 1971].

STEP 1: PROBLEM STATEMENT

The first step involves the fundamental decision to simulate. The process is presented schematically in Figure 1, in which the connectors refer to Figures 2 and 3. In practice the decision to simulate is frequently made before the problem is properly formulated. Hence the problems to be resolved will necessarily be selected from those related to the proposed simulation study. Moreover, the client will often require considerable guidance from model builders on the nature of the processes before the problem can be clearly enunciated and settled.

It is useful at this stage to decide whether the simulation study is to have a long-term payoff any more tangible than a mere clear explanation of the dominant processes and interactions involved. Much care should be devoted to this first step; the risk that the simulation may eventually produce a trivial answer is great.

STEP 2: FIRST STAGE SIMULATION MODEL

Probably most workers intuitively set up the first stage model suggested in Figure 2 to check the reliability of their data. In this step the data may be aggregated; e.g., monthly precipitation and runoff may be aggregated into annual totals to establish average annual evaporative losses. The effect of the first stage model is to filter out high-frequency processes. The purpose is to propose a few dominant processes as a basis for elaboration into an ultimately acceptable model. The first stage model may well treat the prototype space as a single unit or entity, e.g., an entire catchment or lake as a storage tank. What is not immediately obvious is the coupling between the size of the computation time step Δt and the prototype size of the model elements. This point is covered in the third step.

If the objective function is to be used to verify the model, the function chosen should be supported by sufficient field observations or recorded data. Having sufficient data is a

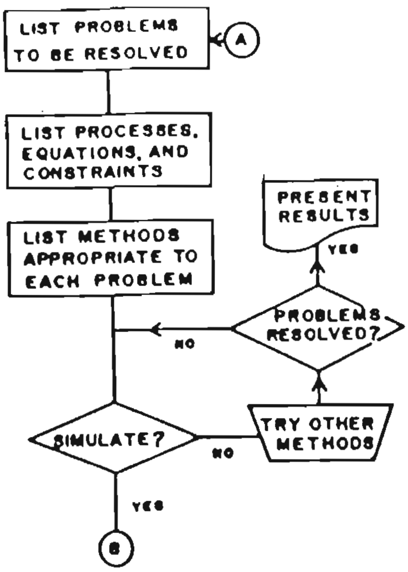

Fig. 1. Problem statement.

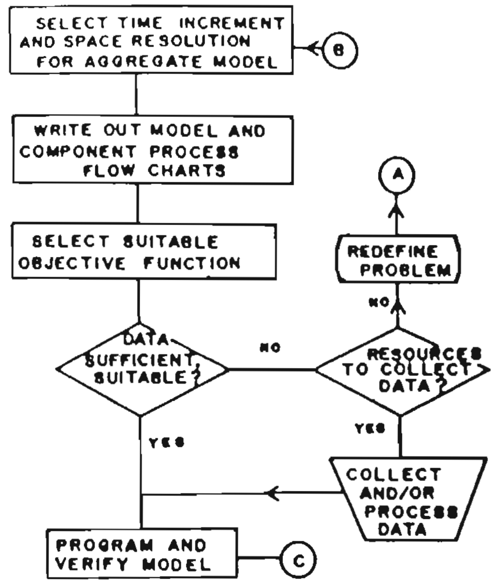

Fig. 2. Aggregate model.

stringent requirement and may be the decisive criterion in selecting the problems to be resolved.

If sufficient resources (i.e., time and money) are unavailable for adequate data collection and/or processing, the problems can often be redefined. Many different questions and objective functions that appear equally relevant to the client can usually be proposed.

For the sensitivity tests described below the model will be arbitrarily disaggregated and input data synthesized. Systematic disaggregation is facilitated by constructing the first stage model in modular form. Thus a simple change of a do loop end variable effectively disaggregates the model into as many constituent elements as may be desired.

Additionally, at the outset the programing should support variable time steps Δt in the computation not only to allow sensitivity tests on Δt but to facilitate continuously varying time steps. In hydrograph synthesis, for example, it is desirable to specify the rising limb more frequently than the recession limb.

STEP 3 AND HIGHER STEPS: HIGHER ORDER MODELS

The systematic development and disaggregation of the model is illustrated in Figure 3, and the procedure is essentially similar to that of Figure 2 except for sensitivity tests and the fact that the cost of simulation now becomes a factor.

In practice, disaggregation and sensitivity analysis proceed simultaneously. Often, careful disaggregation and appropriate selection of Δt will produce better results than sophisticated optimization of empirical factors, such as friction factors for river reaches.

During disaggregation, of course, new arithmetic must be incorporated for those processes tuned to the finer time resolution. For coupling Δt to the size of component elements, the con-

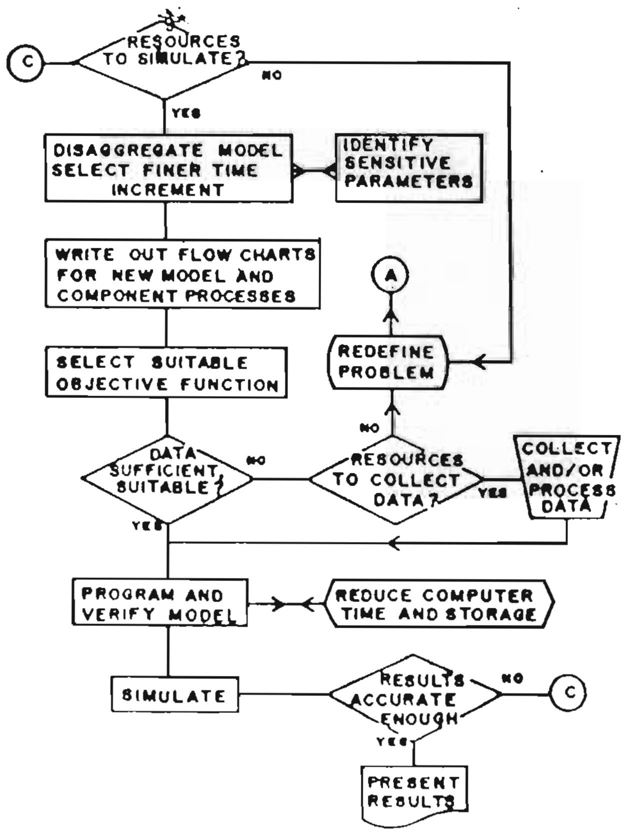

Fig. 3. Model disaggregation.

cept of an impulse response function is useful, e.g., an I.U.H. for flood hydrographs or concentration–distribution resulting from slug loads in dispersion processes. In one of our studies, for example, it was accurate enough for our purposes to represent this response function by a time vector of 20 elements; this usefully indicated the approximate magnitude of Δt. This method also provided a reasonable basis for designing a model of physical processes in a lake but was not altogether satisfactory in a model for predicting snow accumulation in urban areas. Sensitivity tests in which Δt is systematically reduced were evidently more appropriate for this model.

In a sense the flow charts apply to research method in a more general way but seem to be worthwhile for studies for which data collection is difficult (or expensive), such as our catchment studies in the high Arctic and the Virginia karst areas.

I would value any comments on the schema proposed.

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COMPUTATIONAL HYDRAULICS SYSTEMS AS DISTRIBUTED DATA MANAGEMENT

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ABSTRACT

Computational systems in Civil Engineering hydraulics and hydrology include: data acquisition, management, archiving and presentation; documentation and text processing; application programs in meteorology, hydrology, hydraulics, water quality modelling, fluid mechanics, and coastal and ocean engineering; program maintenance and support; and real-time control.

Hardware and software systems are undergoing rapid development in all these areas. Emerging techniques make it possible to supplant event modelling and its associated design storm methodology, with more rational continuous modelling using AES archives, statistical processing of output, and graphics I/O. Continuous modelling, however, leads to data management difficulties. Special software for time series management is essential.

Sophisticated data base management systems and distributed network disc operating systems permit improved cost performance. The paper recommends a network of desk-top microcomputers with a central hard disk system set up as a single user application of a multi-CPU system. The paper reviews some of these developments and discusses the inevitable evolution of microcomputer-based distributed data management systems for Civil Engineering computational hydraulics and hydrology.

INTRODUCTION

Areas of fundamental importance to Civil Engineering computational hydraulics and hydrology include microprocessor-based instrumentation; field data acquisition; data management and presentation; data archiving; data base accession; documentation and report generation; text processing; application program systems, incorporating all water disciplines; program maintenance and support; and real-time control of water systems. Application programs currently widely used in Civil Engineering computational hydraulics include processes usually subsumed under the disciplines of meteorology, hydrology, hydraulics, and municipal and environmental engineering: for example: water distribution, pump stations, urban drainage networks, water and wastewater treatment plants, water quality control, river systems, coastal and tidal engineering, and even limnology.

Hardware and software systems are undergoing rapid development in all these areas, but apart from program systems, little or no overall system building (including data acquisition and control) is evident. Certainly no performance evaluation of such general systems is being attempted.

A few large systems of program packages in the water area were recently developed in a remote batch mainframe environment. These operational or application program systems allow continuous modelling as well as event modelling. By event modelling we mean the traditional design storm philosophy. Continuous modelling requires use of Atmospheric Environment Service climatological archive magnetic tapes for input, and Water Survey of Canada streamflow tapes as well for calibration and validation purposes. The large system packages include statistical processing and graphics input and output (I/O).

Special interactive processors have subsequently been developed for data file management and user-friendly interfaces. The Computational Hydraulics Group at McMaster University has been developing these interfaces for the past ten years. The current or imminent transition to continuous modelling now foreshadows enormous problems of input and output data management. Special software is essential for I/O time series management, software that has proven much more problematical than the relatively simple engineering and scientific algorithms and routines typically embodied in the large system packages.

The ready availability of sophisticated data base management systems, and distributed multi-processor disk operating systems, appears to offer a reasonable path through the jungle. The improved cost performance of powerful personal microcomputers, with a central Winchester hard disk system, encourages a single-user application of a multi-CPU system. The system we require is a distributed network radiating from the central disk operating system functioning on a boss CPU dedicated primarily to data base management (DBM) and time series management (TSM). Each peripheral CPU executes a separable block of logical routines, downloaded from the TSM, associated with, for example, precipitation analysis, runoff modelling, drainage system network, water quality modelling, computational hydraulics, limnology, cost-benefit analysis, or statistical post-processing, as the case may be. This can now be achieved at a fraction of single mainframe costs, and probably just as fast in terms of overall turnaround time.

In this paper, we attempt to review relevant emerging hardware and emerging developments in software systems. We conclude that hydrological research or development groups such as ours will inevitably adopt distributed data management systems based on 16-bit microcomputers using UNIX-type operating sytems and FORTRAN as the high level language.

IMPLICATIONS OF THE CONTINUOUS MODELLING ERA

At the first impression, the traditional mainframe environment with its multi-user and multi-processing capabilities does not seem to be significantly different from the distributed microcomputer system which we propose. After all, the data base used by a group of engineers or scientists is maintained on a single disk system with magnetic tape backup capabilities. The single data base could be made coherent and available to every member of the research and development group with appropriate security safeguards.

Moreover, software can be written to ensure that the relevant portion of the time series output from the logically preceeding block of programs (for example, the precipitation analysis), is properly completed before the logically subsequent computational procedures are allowed to access that time series (for example, the rainfall runoff modelling). Thus a series of users develop a shared data base system in which the first input data set segment is processed for precipitation by one user and, when complete, the output data set is made available for that segment to the rainfall/ runoff block under the control of another user. A simple example is: precipitation data is processed for year 1979 while rainfall/runoff data is processed up to year 1978, the transport network is processed up to 1977, the sewage treatment plant processing up to year 1976, and the dispersion of the resulting pollutants in the receiving waters up to year 1975: all of this processing occurs concurrently.

Yet this kind of system has generally not evolved in the mainframe environment. Users have tended to generate their own applications data bases with severely restricted access and independant storage procedures. Thousands of hours of special software has been coded to convert output from one block of routines to a form suitable for input to the subsequent block. Indeed research in a big machine environment has typically been fractured to the point where a single group of associated researchers has not supported an identifiable data base at all.

In the past, modelling has been restricted to discrete events and thus data management has presented few problems. However, with the improved affordability of computing, continuous processing is much more attractive. Of course, there is a price to be paid: simple changes to the formatting of time series extending (for example) over twenty years at a one minute time step, and associated storage costs, could be quite problematical. For this reason the present writers have adapted the time series management software from the Hydrological Simulation Package-FORTRAN (HSP-F) distributed by the US Environmental Protection Agency, to an 8-bit machine.

TYPICAL USER-GROUP ENVIRONMENT

The activities involved in a typical study by our Group include the following:

1. Set-up microcomputer-based field instrumentation, typically sampling on a 60-second integration period,

2. Acquire a data base of rainfall, runoff and water quality parameters for the system to be modelled,

3. Collect environmental data as required by the models,

4. Develop and apply the continuous model,

5. Verify, sensitivity analysis, calibrate and validate the continuous model,

6. Insert the continuous output time series from the models into the data base system,

7. Conduct a statistical analysis of the continuous output time series,

8. Develop ARIMA models of the continuous input and output time series,

9. Develop control programs for diversion and control structures in the drainage or transport systems being modelled,

10. Elaborate the original continuous models to incorporate the new control programs for the diversion and control structures,

11. Run the continuous models with the control structures appropriately modelled for the full time series record,

12. Compare the output for various control structure programs and choose the "best",

13. Summarize the output,

14. Prepare documentation,

15. Install the 'best' real-time control system in the field,

16. Set up a continuous program of data acquisition, modelling and software maintenance to ensure model and prototype performance.

It is clear from this list that the data base management system (DBMS) is of central concern. Indeed a group of (say) five or six researchers ought to give serious consideration to the designation of one of their members as Time Series Administrator (TSA). The TSA is responsible for the acquisition and archiving of all field data, application programs, remote data bases, text processing files, man-machine dialogue, pre-processors, statistical and other post processors, and so on.

In the Computational Hydraulics group at McMaster University the microprocessor field instrumentation was developed by Hector Haro, and installed by Dale Henry and Sam Adile. The software for acquiring the field data was developed by Mark Robinson and is known as FASTPLOT. The data is stored on hard disks with magnetic tape backup on a PDP1123 under RSX11M. The operational modules processing the data are blocks within HSP-F and the Stormwater Management Model (SWMM3). The package of programs which pre-process the data base for SWMM3 is known as DATANAL, also written by Mark Robinson. These files are stored in a form compatible with the FASTSWMM routines. The evolving Computational Hydraulics Group data base, called CHGTSS in this paper, is stored in random access memory (RAM) with weekly archiving on sequential media.

EVOLVING HARDWARE

Memory chips have doubled in capacity without price increase on an approximately annual basis. Microcomputer manufacturers pay the same price today for a 64K-byte RAM as they did for a 4K-byte RAM in 1975. Sometime in 1984 a 256K-byte chip should cost about the same.

The Zilog Z80 and the MOS-Technology 6502 have achieved wide acceptance in personal desktop 8-bit computers. Both of these chips are 64K RAM, the Z80 appearing in Radio Shack and most CP/M based microcomputers, while the 6502 apppears in the Apple, Commodore, and Atari microcomputers. The newer 16-bit microprocessors have extended memory space up to between 1 and 16 megabytes. The INTEL 8088 chips has up to one megabyte of memory, the Motorola 68000 a 16 megabyte memory space. The 8088 was first on the scene and has been widely distributed amongst Civil Engineers in the IBM-PC. 16-bit microcomputers should satisfy most Computational Hydraulics and Hydrology needs for the next three to five years, by which time bigger program packages incorporating subsystems in eco-modelling would have become available. In addition, in the meantime, some inter-disciplinary computer programs (eg. chemical processes) where the word size required is necessarily much greater because of the high number of iteration cycles, may find application in our area. When the distribution of sophisticated interactive man/machine software, such as the LISA system distributed by APPLE, becomes widespread, it will be essential to move up to 16-bit or 32-bit microcomputers. Our experience indicates a minimum dedicated central core memore space of 500K-bytes per user for our models.

Video displays have not come down in price as rapidly as chips. Displays should be in colour, at least 80 characters per line to be compatible with software developed in the past five years. To be compatible with earlier software, displays should preferably be at least 132 characters per line. The best software today requires a high resolution color graphics display, i.e a RGB (Red, green, blue) monitor. Separate input is required for each of the primary colours. We suggest high resolution bit-mapped colour graphics of about 600 x 300 pixels.

A standard alpha/numeric typewriter keyboard is the minimum requirement for the input device. Alternate input devices such as graphics tablets, joy sticks and the mouse are becoming increasingly important. Such devices allow fast cursor control; in the near future these will be highly utilized for direct model calibration and validation. For real-time control or other embedded computer applications , the Hewlett-Packard instrumentation bus (HPIB) is a general purpose interface designed specifically for instrumentation systems requiring limited distance communications. HPIB is compatible with most systems and no understanding of internal hardware is necessary.

In the past twelve months, printers with plotting capability have been the fastest growing peripheral market in the microcomputer industry. New dot matrix printers operate at 120 to 160 characters per second and have significantly improved print quality due to high dot density, for example 13 x 17 per character font. Unfortunately buffers are still quite small, typically 2K-bytes.

For mass storage, the transition from cassettes to floppy disks drives is now quite standard. In addition, floppy disks are sufficiently inexpensive and store sufficient data to offer an attractive backup system for hard disk secondary memory. Although double sided, double density 80 track 5 1/4 inch floppy disks store up to 0.7 megabytes, 8 inch floppies are to be preferred.

Recent improvements in hard disks have kept pace with those in floppies. Their sizes have been reduced and the amount of storage offered is now typically in the range of 5 to 30 megabytes. It is the storage capacity which can be expected to increase rapidly in the near future. For a group such as ours, a rotating mass storage of approximately 60 megabytes is indicated, with file serving software allowing routine backup to double density 8 inch floppies, for transportability.

Other hardware to consider includes built-in modems, power supplies and RAM cartridges. But the overriding consideration is the user community with its available software.

EVOLVING SOFTWARE

Operating Systems for Personal Computers

CP/M was the first disk based 8-bit operating system to be available on a wide variety of CPU's. CP/M is the module that gets the command from the processor and reconciles the differences at the hardware level. Unfortunately, CP/M is not particularly easy to learn, requiring the use of a large manual. Of course, new operating systems are available that get around many of CP/M's problems. For example, CP+ is much easier to use. CP/M operates on 8080, Z80 or 8085 8-bit micro computers in a single user/single task environment. CP/M does not provide any user protection and is best suited for experts.

Advanced Operating Systems

CP/M 86 is designed for the 8086 16-bit microprocessor. It is otherwise similar to CP/M. MP/M on the other hand runs on the same 8-bit microprocessor as does CP/M but allows multi-user operation. MS-DOS is the operating system supplied with the IBM-PC. It is therefore widely used in Civil Engineering offices. Another MP/M type operating system is OASIS which also utilizes BASIC, but the length of the variable is over 200 characters. The drawback is the limited distribution and number of application programs. OASIS is based on UNIX which is mini-computer oriented. UNIX requires a 16-bit microprocessor with at least a 128K of RAM. It is so user friendly that systems programmers oppose it because they fear obsolescence! Several variants of UNIX are available. For example, CROMIX, written for Cromemco microcomputers has superior data integrity, and MICRO-SHELL offers UNIX like features on CP/M machines.

High Level Programming Languages.

The smaller the local CPU, the more likely it is that the programs written in the high level language will be dependant upon the host language, which in turn is more machine dependant. Local compilers and interpreters interface with the hardware, consequently transferring application programs from one small system to another becomes increasingly difficult as the system becomes smaller.

The languages of widest interest in Computational Hydraulics include FORTRAN, BASIC, PASCAL, PL/1, and FORTH. PASCAL and FORTH are more of a programming environment than a high level programming language. PASCAL is multi-purpose, rather complex and expensive, but interactive, provides a dynamic memory and generates relatively bug-free code. The math library in PASCAL is not as good as that of FORTRAN. FORTH on the other hand has an unconventional syntax and seems to use the programmer as a compiler. BASIC is widely used in microcomputers and millions of lines of code have now been generated, but not in Computational Hydraulics. There are too many versions to regard BASIC software as being easily transportable.

FORTRAN remains the best language for scientific and engineering applications because of its superior mathematical qualities. Computation of complicated mathematics is handled efficiently with good library routines and overlay features. However, character handling is still inadequate and absence of recursion is a handicap. The language is not interactive and is hard to debug. It is evident that FORTRAN will remain a vital programming tool in the future, but in the 8-bit personal microcomputer field, it is important to recognize that compilation may be a lengthy and tedious procedure. As popular microcomputers support more and more mainframe features of FORTRAN, the language should quickly emerge from its relative obscurity in the 8-bit marketplace.

DATA BASE MANAGEMENT SYSTEMS

A DBMS provides data independence for all users. Knowledge of the data organization and access is built into the DBMS application logic. Thus the DBMS is a set of procedures as well as data structures that isolate the application from mundane details of storage retrieval, creation, modification and security of the data base. From the point of view of Computational Hydraulics, a DBMS is attractive because the user does not refer to physical storage locations but deals instead with the conceptual data structure. The data base is built and maintained by the data base administrator (DBA). Each of the individual users have an entirely different logical view of some portion of the data base. A data base may be conveniently categorized into three general types: relational, hierarchical, and network. If data is structured like a tree, it is said to be hierarchical. If the data structure is more general than this, and can be represented by records and links, such that any given record may have any number of immediate superiors, as well as any number of immediate dependants, the structure is said to be a network. The relational approach is the most important for Computational Hydraulics. By 'relational' we mean the equivalent of a table. Each row or tuple represents the attributes of the data.

As soon as several users require access to operational data which is being changed, and which is sufficiently large that personal copies of the data for each user would represent a significant load on the computer system, it is desirable to provide a centralized control of the data bases. In this way, redundancy can be reduced, inconsistency can be avoided, data can be shared, standards can be enforced, security restrictions can be applied, integrity can be maintained, and conflicting requirements can be balanced. All of these DBMS attributes are significant to an enterprise concerned with Computational Hydraulics and Hydrology systems, especially enterprises with a relatively high turnover of its members.

DISTRIBUTED TASK PROCESSING

Where the user group requires a variety of computer hardware and/or operating systems software, it becomes desirable to develop a distributed processing system. Many hardware manufacturers offer hardware and software systems that may be configured into a multi-processor system. This means that there is at least a minimum level of compatability so that the I/O bus adaptors to the interconnect are not required to handle complex incompatabilities.

Of course, operational modules must be modified to execute in this new environment. For example, pre- and post-processing portions may run on one CPU while the central, externally supported, batch environment processor may run on a larger CPU configured specially to handle this kind of number crunching. Preprocessing usually includes generation, manipulation, and display of input data. The result of this phase is a formatted permanent file to be input to the processing phase. The post processing system usually manipulates the data base prepared by both the pre- and processing CPU. All of the files referred to by the pre- and post- processor systems must be separate from the scratch files, usually of considerable length, generated in the processing phase. In a typical engineering application, a solution is sought to a large number of simultaneous equations, developed as an approximation to a system of partial differential equations. Typically, fifty to seventy percent of the computational effort may be devoted to the solution of these simultaneous equations. For this reason, the operational modules in the processing phase should be modified to run on a CPU offering the best cost performance for this kind of computational effort. Large hydraulic engineering application programs that enjoy a wide distribution or utilization, are the first to be adapted to a distributed processing system, kbecause these systems offer reduced computational costs as well as maximum utilization of manpower. For example, interactive pre- and post-processing is carried out on dedicated CPU's.

In a tightly coupled computer system, the CPU's share memories and peripherals such as printers and disks. In a loosely coupled system, the CPU's usually have their own memory and peripherals and communicate with each other over a communications network. A tightly coupled system is often called multi-processing, while a loosely coupled sysem is called distributed processing. In both cases, different CPU's perform different tasks and communicate with each other to coordinate their actions in a larger overall task.

In a single user environment, the state of the CPU, disk and other I/O devices, does not change. In a multi-task environment, the component programs may change the memory, disk files and I/O device status while any one program segment is executing on another CPU. Thus each component operational module must be written to coordinate its activities with other modules. Coordination of the segments of the program must recognize all possible conflicts. The most obvious example occurs when two programs require to change the same data record at the same instant.

A loosely coupled system may also incorporate a distributed data base in which each peripheral CPU has direct access to a component of the data base, stored in its own memory or disc system. However the directory for all data bases may reside in one of the CPU's. If access to the data base is locally controlled by the peripheral CPU's, then there is again a risk that data base management will not conform to the requirements of the Data Base Administrator.

Most of the present generation of 8-bit personal computers based on the Z80, 8080, 6800 and 6504 chips are not powerful enough to serve as nodes in such multi-processor networks. However, 8088 based personal computers and 68000 based computers such as the IBM-PC and APPLE III have the necessary power and memory capacity (for example, 256-K bytes).

Many problems in computational hydrology are equally if not more demanding of file access than of CPU time. In this case a smaller less expensive computer system than the conventional powerful mainframe (with large word sizes, and fast CPU cycles, and sophisticated instructions) with adequate file use is often just as effective a solution but at a significantly lower cost. Thus an intelligent disk storage system configured as a stand alone data base management system accessible by several CPU's (or terminals) may represent a more sophisticated solution. The intelligent, field expandable, disk controller handles all data access including indexing, searching, buffering, deblocking and storage management functions. Such a system makes more efficient use of the CPU so that the user obtains better response times and real-time applications become possible.

Simple multi-tasking can be built into any program. But it is evident that time series simulations, so common in computational hydrology, particularly lend themselves to multi-task implementation. Of course, care must be taken to identify synchronization and coordination conflicts in the multi-tasking environment. This development seems so inevitable that steps should be taken now to ensure that current engineering software generation is adapted to move into this evolving 16-bit multi-task environment. The advantages of this environment include increased reliability, increased survivability, increased distributed processing power, increased responsiveness, increased modularity, and better system expandability. The price to be paid is in the form of increased software complexity, more difficult system testing and diagnosis, more dependance on data communications, and better expertise required during software design and development. The most effective argument, however, lies in the improved benefit/cost ratio.

COMPUTATIONAL HYDRAULICS GROUP SYSTEM

The time series management system of HSP-F is being adapted by our group to form the relational DBMS, CHG-TSS. The CHG-TSS includes the time series store (TSS) management functions TS-GET, and TS-PUT. TS-GET obtains the required records of the time series from the appropriate file, aggregates or disaggregates it to the correct time interval, scales the values according to user requirements, and places the data in the required position in the internal scratch pad. TS-PUT performs the reverse set of operations. The principal library for the medium to long term storage of the time series is the TSS, which is sub-divided into many data sets containing time series, each of which is logically self-contained but physically scattered throughout the TSS. A directory keeps track of data sets and their attributes. Thus the TSS must be correctly initialized and the directory properly created, by executing the program NEWTSS. The TSS-MGR software can store many distinct time series in the TSS, thus permitting a user to easily keep track of the various time series with which he is dealing. This simplifies communication with the computer system. The primary function of module TSS-MGR is to maintain the directory using user input, such as setting up new labels in the directory, updating the label , scratching a data set label and data set contents, extending the space allocated to a data set, or displaying the contents of one or all of the labels in the directory. The TSS-MGR is used externally to prepare the time series for sequential access by various other application modules. This is common to most DBMS systems, by the way.

All of the time series included in an internal operation are viewed as being written on an internal scratch pad (INPAD) in the central memory. The operating module receives the scratch pad with some rows filled with input, and after processing is complete, returns control to the operations supervisor with another set of rows filled with output. The operating module thus may overwrite an input row with its own output. However, the total quantity of machine space available for storage of time series is also fixed, for example, by specification of a common block. This is the size or area of INPAD, specified by the number of intervals N and the number of rows M. Regardless of the amount of time series data requested in a single processing run, the core space is fixed. Obviously, the more time series data required, the more transfers required between TSS and the target INPAD.

The TSS-MGR can also be used to prepare time series data files for application modules external to HSP-F. Obviously this does not necessarily result in any improvement in overall performance but certainly eases preparation, manipulation, storage and support of the TSS. We are presently contemplating implementation of further query attributes in the TSSMGR to improve its external time series management capabilities.

CONCLUSIONS

The novelty in our approach lies in getting away from a single processing unit with a single huge memory, performing in a sequential step- by-step path. This system is replaced by a number of simple processing elements, each endowed with its own small memory. The processors themselves are arranged in a hierarchy. At the top of the hierarchy, lies the boss processor, which tells the others which jobs they should be doing concurrently, or in parallel. The boss processor may also be known as the time series store manager. Only the TSS-MGR has access to the hard disc where the time series store is resident. The peripheral CPU's download the separable operating modules from the central DBMS.

The adoption of 16-bit personal computers with their improved inexpensive computational power, inevitably leads to continuous modelling. This turns up problems of management of large time series. It follows that data base management systems will ultimately be required by every group of engineers sharing a computational hydraulics data base, as soon as the software costs become a significant part of the total computer costs. Data base software significantly reduces the cost of software development in several ways: programmers are not required to maintain their own copy of the data base and data file, an internal directory describing the relationship among the data is automatically maintained and is available to all users, and programmers are not required to write special purpose programs to fit the data.

We are generating complete Computational Hydraulics systems by integrating DBMS utility models with the regular operational or application modules required and developed by other members of our group.

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INTERACTIVE DESIGN USING MICROPROCESSORS COMMUNICATING WITH LARGE BATCH-ORIENTED PACKAGES AT REMOTE MAINFRAMES

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ABSTRACT

A variety of large, complex simulation packages are widely used for the analysis of civil engineering design problems, particularly in the area of hydraulics and hydrology. These packages usually execute on mainframes with large memory capacity and high computing speeds. The programs comprising the packages are typically designed to operate in a batch environment and require that users have a good working knowledge of mainframe Job Control Language to execute them efficiently. Thus a new engineer-user may find himself facing as many computer-oriented difficulties as he does difficulties associated with the design problem itself. Furthermore, long learning times are associated with the implementation and execution of complex packages on batch-oriented mainframes at remote sites. The problem is compounded when a multi-program computational solution is required. This paper discusses an interactive package of pre- and post-processing programs developed to facilitate input data preparation, job execution, output interpretation and job resubmission in a multi-program remote batch environment.

The pre-processors create input data files for job execution; the post-processors operate on the output from the simulation package, synthesizing, analyzing and plotting to speed up user interpretation at his terminal. The original coding i.e. the batch-oriented package is not altered. Furthermore, local text processing can be used together with disc-based skeletal outlines, to elaborate the post-processor output directly into a draft report.

A multi-processor configuration for the interactive system and the batch-oriented packages has been found to be cost efficient. The pre- and post-processors operate on a PDP 11/23

microcomputer employing the RSX-11M operating system and having 32K decimal 16-bit words of memory available to the user. All programs are written in fully-transportable ANSI FORTRAN. The batch-oriented design packages execute on a CYBER 170/730 with 140K octal 64-bit words of memory. Execution on the remote mainframe is transparent to the user.

INTRODUCTION

Current North American state-of-the-art civil engineering practice in urban drainage design studies (hydrology and hydraulics) involves heavy use of computer techniques. A large variety of simulation packages exist for simulating many of the components of an urban drainage system to varying degrees of complexity, eg. SWMM (Huber 1975), HEC-2 (U.S. Army 1979). As well, comprehensive data bases of rainfall, streamflow and water quality observations have been created by various government departments. A typical problem faced by consultants is how to economically implement these required packages efficiently and integrate them with the available data bases.

These firms are faced with certain realities:

1. They cannot afford to develop and/or maintain independent simulation models. Thus they must access specified models devised by others. Usually this means implementing a model whose input requirements are cumbersome and whose output is unsuited to the engineer's computing environment. Good documentation only partly offsets these difficulties.
2. They do not own a computer large enough to implement the model. Consequently it is necessary to use a commercial computing facility.
3. At a large batch-oriented computer centre a great deal of time can be spent simply waiting. The actual time involved in executing a job is very small compared to that required to advance the job through the input and output queues and to return the output.
4. Long delays in turnaround results in long learning times and slow model calibration prior to production runs.

Most potential users of these packages could be classed as "casual" users (Cuff, 1980) i.e. they have a good grasp of the concepts being modelled however, since they use the model infrequently they do not easily recall how to execute the model on the system. Hence, casual users tend to shy away from fully exploiting the modelling resources and the system capabilities. For example, a user may avoid interfacing two or more models, each describing in detail an individual process but requiring working knowledge of system JCL, in favour of a single, general model which deals with the processes in a more superficial way but which requires little involvement with the computer system. Kennedy (1975) identified this problem as a major roadblock to the acceptance and full implementation of a computer system.

Not all the blame for a poor partnership between the user and the machine rests with the computer centre. With very few exceptions program input and output controls are not conducive to developing a smooth working arrangement between the user and the computer. The fault often lies in the fact that the program designer is an experienced batch-user while most users range between novice and intermediate (Embley 1978). What may be an obvious technique to a professional programmer may bewilder a user unfamiliar with system JCL or program Input/Output control. Embley, et al (1978) succinctly quantify the purpose of using a computer system: to allow a user to communicate directly with a machine to accomplish a goal with the minimum expenditure of time and effort.

During the past decade significant advances have been made in computer hardware and software development. Microprocessor technology has made highly efficient microcomputers affordable to many users. Versatile, sophisticated simulation program packages have flourished. However, most computational packages currently in use in urban drainage design practice, are lacking in the man-machine interface. There is a critical need to improve the interaction between user and computer, to permit the user to compute efficiently, develop a positive attitude towards computing and to exploit fully the computer as an information tool.

INTERACTIVE DIALOGUE

Batch processing is defined as a process in which a run is submitted to a computer, through a low-speed device such as a card reader or keyboard terminal, the run executed and the output returned to the user either immediately at the terminal or printed and delivered at some later time. There is no communication between the user and the program from the time the run is submitted until receipt of output. Interactive computing is defined as a process which may be initiated by either input device, usually a terminal, but in which the program halts at predetermined points to permit the user to make certain decisions, based on the interim output and which influence the remainder of the calculation.

An extension to interactive computing is conversational computing. In this mode a pseudo-conversation is carried on between the program and the user in an effort to prompt or guide the user in making run time decisions. It is particularly helpful to casual users.

The relative merits of batch processing, and interactive time-sharing computing have been examined; differences in programs are a more important source of performance variablity than system differences (Miller 1977). For instance, a programmer-user would probably find poorly designed interactive processing less efficient than batch processing for achieving the same goal.

Whether to use batch or interactive computing depends on two issues: the most appropriate method of use for the engineer, i.e. whether or not it is useful for him to interact with on-going calculations in terms of the design function which he is attempting to carry out, and the data required to set up the various runs of the program. In most urban drainage design situations, the engineer is faced with exploring a number of alternatives for feasibility and/or cost-effectiveness. The design engineer must make subjective evaluations provided sufficient information is available. For example engineers prefer to decide themselves the best solution to a problem. Thus an interactive procedure should not be an all-encompassing decision making package in which the user plays only a passive role. Rather it is critical that the procedure be a tool which the user can employ to bridge the interface with the machine to achieve actively a full complement of information regarding all options in a multi-objective decision problem and thereby make an acceptable decision.

If a program operates in a cumbersome or inefficient manner the user could be limited to a small set of alternatives. Newstead and Wynne (1976) and Roy (1980) found that suitably designed interactive procedures assist users in making judgements necessary for the solution of multi-objective decision problems by providing more complete information. Interactive computing would appear to be preferable in most complex design situations but the package must be efficient to be viable.

Central to any interactive system is the dialogue which provides the interface between man and machine. The importance of making the numerical model work for the user has been recognized (Fitter 1979, Cuff 1980, Kennedy 1974, 1975, Treu 1975, Miller 1977, Gaines 1975, James 1978, Shneiderman 1980, Martin 1973) and cannot be overemphasized. If the method of communicating with the computer is complicated and exacting, or the dialogue ambiguous, the positive aspects of computing will be nullified. "It is important that the dialogue between the man and the machine flow naturally, making explicit the knowledge and processes for which the man and the computer share a common understanding". (Fitter (1979)). The user understands that he is to supply information for a certain calculation technique and the machine will "understand" (i.e., is so programmed) that it will be required to interpret information necessary for that technique.

Fitter (1979) observes that users often assume the computer program to have unrealistic powers of inference and a greater intelligence than it actually possesses. The dialogue must be designed to safeguard against this tendency. Treu (1975) identifies a range of languages which could be applied to interactive systems. They range from a computer-oriented artificial language, which can be difficult for the casual user to remember and implement (Kennedy (1975)) to a language very similar to the spoken word which through its ambiguity can be essentially impossible for the computer to comprehend. A flexible user-oriented limited language has evolved in our conversational procedures. The user has available a specific set of allowable commands which can be interpreted by the computer and which are therefore unambiguous, yet which have associated with them obvious meaning for users and are therefore more easily remembered.

In creating our interactive procedures thought was given to the exact nature of the interface between the man and the machine. Miller and Thomas (1977) identify four basic types of interface:

1. System guides, user has forced choice; the user is stepped through a fixed procedure and must choose data from a specific list of items. This form of interface provides quick processing and minimizes error.
2. System guides, user has free response; The user is stepped through a fixed procedure but can supply data in any form he chooses. This form of interface has the potential especially in numerical applications, for ambiguity and error.
3. User guides, user has forced choice; The user has the option at all times of instructing the system in which direction to proceed, but can only choose data from a given list of items. This type of interface would be appropriate for users who have some familiarity with the potential of the system. It could result in increased exploration of the solution space while at the same time minimizing errors.
4. User guides, user has free response; The user selects the direction of the system and supplies data in no prescribed form or order. This interface has the potential for the most error and ambiguity and should only be used in situations where all users are experienced.

Our interactive procedures have been tailored to the casual user, combining types 2 and 3. The user can select a specific calculation option or combination of options and the system will guide him, using conversational prompts, and echoing dependent parameters to supply the required data. However, the values which may be supplied for the required data are left to the user's discretion. This maximizes the potential for the user to explore alternatives while minimizing the potential for error. On the other hand experienced users can switch prompts and echoes on and off at any point.

There are three levels at which attention should be paid to user psychology - the functional level, the procedural level and the syntactical level (Robinson (1977)).

At the functional level, it must be made clear which functions are to be carried out by the designer and which functions by the computer. A brief introduction at the beginning of the procedure is important. The user is made aware that the computer will prompt when necessary. The user can devote his time and attention to interpreting the design problem. Supplemental to this, the interactive software should accord with the ongoing calculation. For example, the pre-processor's function is to create an input data deck for the analytical package. The prompts for input coincide exactly with the documentation for the program input, written and supplied by the original authors.

At the procedural level, the software is designed to provide alternatives. The alternative chosen will direct the program logic along a certain course. The designer determines the course by selecting an appropriate alternative.

At the syntactical level a number of guidelines for achieving effective conversational software can be proposed. The manner of presenting prompts for input and displaying output is the area in which the greatest scope for improvement exists. The manner in which data are formatted can have a profound effect on the user's efficiency and error rate.

Treu (1975) defines "mental work" as the concept of certain procedures, tasks or commands being more or less difficult than others. Further, he defines a concept called the "user transaction time" as being the time from the output of the last system character to the input of the last user character. The durationof each component of the user transaction time varies depending on the length and complexity of the system message, the ease of selecting an appropriate command, the length of that command (number of terminal keys depressed) and the user's reading and typing skills. One objective in designing an interactive system should be to keep the user transaction time to a minimum while maintaining a clear understanding of what is required of the user.

In designing a well-behaved interactive system we have established criteria from our own experience as well as others (Fitter 1979, Cuff 1980, Kennedy 1975, 1974, Miller 1977, Gaines 1975, James 1979, 1978, Robinson 1977, Shneiderman 1980, Martin 1973). (If these criteria appear obvious, consider the number of systems and programs in widespread use which violate them.)

1. The dialogue should be terse, coherent and unambiguous, yet should maintain an ambience which can be used to develop a cooperative attitude. The behaviour of the machine should be consistent with the dialogue. The dialogue should be structured to flow smoothly from one concept to the next. The user should know the end result that he is working towards, i.e. all pieces of input he supplies follow in a logical sequence. A prompt

for input should carry a tone indicative of the form of response required. The prompts should always be presented in the same manner. The signal that input is required (eg. the use of three dots (...)) should appear in the same position, say, forty spaces from the left hand edge of the paper, so that the user becomes accustomed to the manner in which input is requested and entered. This minimizes the chance of error and develops a reasonable tempo for the session.

2. Prompts should be concise to minimize the chances of ambiguous interpretation by the user. Each prompt should be prefaced by a number which allows the user to refer easily to a pocket manual for aid. In particular, experience has shown that easily readable, portable, concise, complete documentation for a program is often much more effective than a large output file of HELP commands. Using a manual containing examples in conjunction with an informative diagnostic presented at his terminal can resolve a problem quickly. This is especially useful when numerical quantities are to be entered. The manual should outline values the parameter in question would take in a variety of circumstances. (The pre-processor can only check an outer range beyond which input is clearly absurd.)
3. The user should not be required to respond to more than one idea at a time. This will avoid confusion and frustration and will minimize the possibility of error.
4. The input translation routine should accept data in a free-format mode. Users, particularly if they are not familiar with a computer language, say FORTRAN, may become confused if they are suddenly faced with having to enter data in a specific format, for example G10.3. Also, the translation routine should accept abbreviations for commands, instructions and data as well as in full. This accommodates users with a range of experience. For example, for numerical input zeroes for data elements at the end of an input string may be entered by leaving the remainder of the string blank. The interactive system would automatically default these data items to zero.
5. The computer should always respond to the user. Some indication should be given that the user response has been accepted or rejected. This may be a specially designed echo, message or another prompt. All input requests for the particular stage of the program operating should be made in proximity to one another; during lengthy calculations users are periodically informed "calculation proceeding".
6. The user must be able to observe and control the procedure and be able to abort the current state of the system and "reset" the procedure to either the initial state or a new user-specified local state.

7. Data entered should be validated by checking syntax and comparing the values provided with "normal" values. Care should be taken not necessarily to reject data outside the "normal" range; it is surprising how much data are not "normal"; rather flash a diagnostic (which does not antagonize the user but rather invites him to reconsider his data).
8. Error messages should be designed to convey information in a concise manner yet not antagonize the user. Carefully worded error messages will encourage the user to try again without giving the impression that errors will be tolerated continually.
9. Results relayed to the terminal should be ordered and easily read, such as graphics or clear printer-plots. They must convey only essential data yet be clearly labelled to indicate their significance. It must be remembered that further input will likely depend on these results. If the user cannot decipher then he cannot supply additional data and the design sequence breaks down.

Dialogue designed around these basic criteria will be efficient, user-friendly and reliable.

Control of User Errors

Users are human and make mistakes. It is essential that the interactive system detect as many errors as possible before the engineering design can be affected, and inform the user politely. Martin (1973) provides an in-depth examination of the control of user errors. Design criteria for error control (Martin 1973, Robinson 1977) include:

1. The dialogue should be structured to detect immediately as many errors (syntactical and numerical) as possible.
2. The dialogue must provide the user with an unambiguous description of the error and why it occurred.
3. The interactive system must provide a facility for immediate correction of errors.

Error messages should provide information and encourage the user to avoid repeating the mistake.

The Design Dialogue

Our dialogue procedures are primarily concerned with alphanumeric data entry, but the user may take control of the procedure by entering any of the following commands in place of data:

BACKSTEP	(start this step over again)
DEADSTART	(start the entire engineering design package over again)

PROMPT (alternately switch prompting off/on)
ECHO (alternately switch echo checking off/on)
END (terminate procedure)

This list does not include procedure instructions such as "SUBMIT" or "PLOT".

To accord with larger program packages supported by several U.S. Government Agencies, commands are expressed in at least the first three columns of the data line, while columns 4 through 80 are used for numeric data, keywords and data file commands. The commands are started in the first space and may be curtailed after the third letter; only three letters are necessary to identify the command.

When entering data the user shculd first leave three spaces blank as these spaces are reserved for commands. Spaces or commas are used to separate data. When all data corresponding to one prompt are entered, the line ends with a "carriage return". When prompts for entering data are printed at the terminal they are followed by a reference to "CARD n". The "CARD" descriptions correspond to program documentation.

The data can be written in any format, but at least one blank space or comma must be left between data items. If two commas occur together, for example, the procedure will assume zero as default for the expected data item and the processor inserts default input as necessary, a convenient type of shorthand for the experienced user.

A decimal is required for numbers containing fractions, but not for whole numbers. Keywords and comments can be written with the data to describe individual data items. The input translation routine will disregard alphabetic input where it expects only numeric data. This is convenient for future archiving.

INTERACTIVE CONVERSATIONAL PROCESSORS FOR LARGE SIMULATION PACKAGES

Our interactive system comprises three parts: pre-processor programs, the large externally-supported simulation package and post-processor programs. The pre-processor's functions are:

1. Solicit and accept user-directed input required by the external package, in conversational free-format mode, usually from a remote terminal.
2. Solicit and accept design-oriented input data (if any), such as a rainfall or temperature data base, from input devices or units as directed by the user for the evolving design.
3. Check the validity of the input and return values of certain key dependent parameters to the user's terminal.
4. Convert the units of the input data (usually metric) to units required by the main program package.

5. Convert the free-format user input to the formats required by the external program, for example, from our above line-oriented free-format to rigid arbitrary fixed-formats.
6. Construct a system Job Control Language file to submit the main program to the batch CPU together with this fixed-format input data.
7. Save uniquely identified input files in the user's catalogue to be subsequently modified, archived or destroyed.

The function of the external package is to simulate the requisite processes. The post-processor's function is to:

1. Return selected output to the user's terminal upon program completion and after reconverting (if necessary) to various units.
2. Calculate various correlation functions between simulation results and observed or expected data (if any), to evaluate the current solution.
3. Printer-plot the results (if requested) where applicable.
4. Reinitiate pre-processing for further exploration of the solution space once the data are modified.

By arranging these processors outside the main external package, it remains untouched and as issued. The new version of the coding is simply substituted for the old version. The processors are easily modified to incorporate any I/O changes.

The following is a typical Operation of the Interactive system. Through a log-on procedure, the computer system on which the procedure resides is accessed and the procedure is activated using a "CALL", procedure name" command. Besides simple editing, this is the only system command the user need know. It presents no difficulty for even the casual user.

Upon entering the procedure the user will be asked to provide three pieces of job-related information:

1. a 3-character identifying code for this particular user job session, "NNN", where N can be any letter or number,
2. a single identifying case number "X" to differentiate unique trials for this particular user or job session,
3. whether a new data file ("NEW") is to be created from scratch for this job session, or whether an existing, modified data file is to be reprocessed ("OLD"). All new files need new case numbers X unless the old file case number X is to be overwritten.

The pre-processor creates two files: (1) "NNNdatX" which is an echo of the free format new units input provided by the user and which may be modified, and resubmitted to the pre-processor as an OLD data file. If requested the pre-processor accesses remote design-oriented disc files and amalgamates them with the free-format input to produce a complete design data set and, (2) "NNNinpX" a data file which is specifically formatted and in the units required by the main simulation package and which will form a portion of the package job stream.

The pre-processor builds a JCL file which, if requested, submits the main program package, together with file NNNinpX and any remote disc files, generated by previous runs of this or another simulation model, to the batch CPU where the calculations are carried out.

The interactive procedure then moves into a wait mode until the results generated by the CPU are returned to the user terminal. The files are: (1) a listing of simulation results which need to be presented to the user to form part of the decision making process, file "NNNoutX" (2) machine readable code comprising pertinent results in a form suitable for input in subsequent runs of this or another simulation model, file "NNNtapX", (3) Job dayfile recording CPU times for each constituent task, for program maintenance.

The procedure then activates the post-processor. Key results are returned to the terminal in summary form and perhaps as a printer plot. The user may then request further analysis of this output including linear correlations. Based on the results the user decides if he wishes to continue to explore the solution space further. If so, the original free-format data file NNNdatX is modified using the system editor to formulate a new approach to the problem. Additionally the user decides whether to invoke a new segment of the program package in which case a NEW job session or case number may be implemented. The procedure proceeds in a cyclical, iterative fashion. The user is freed from the need to learn JCL to create disc files for subsequent runs, and can run an unlimited number of jobs from the outset. The package has been found to reduce learning times, minimize user errors and reduce total design turn-around times. The procedure evidently considerably reduces the complexity of job submission; users are able to focus on the processes modelled. No knowledge of FORTRAN formats or systems control language is required of users. As such it is especially valuable in an instructional/training environment.

APPLICATION OF MICROPROCESSORS TO REMOTE INTERACTIVE PROCESSING

Pre- and post-processor dialogue requires short bursts of CPU activity and demands fast response. On the other hand the external main package usually requires considerable

CPU resources, and so does not qualify for high priority. It is clearly better to separate these activities placing each on a CPU dedicated to that environment. Unless an active CPU is dedicated to one user, the interactive procedures function most efficiently in a multi-processor environment.

After issuing a prompt the procedure is "rolled-out" of core and the job having the next highest priority is executed. This often results in delays of up to one minute between responses, or longer where substantial CPU resources are required. Experiments in user psychology have found this seeming lack of control over the system to be detrimental to attitude and self-teaching potential (Fitter, 1979, Kennedy 1974, 1975, Brown 1975, Miller 1977, Gaines 1975, Fredrich 1972, James 1974). A self-taught approach with "hands-on" experience is far preferable to learning from manuals or documentation prepared by others. Not only must the software be user-friendly but also the computer system available must be appropriate.

Microcomputers have been found to have a number of advantages over large mainframes (Brebner 1973, Gaines 1975):

1. The inexpensive machine can be dedicated for an appreciable time-slice entirely to the user. He can take as much time as he requires to supply a response to a prompt without incurring prohibitive costs. Delays which would be suffered due to swapping on the large mainframes with a large group of users are eliminated.
2. The cost of computing on a microcomputer system is negligible.
3. File storage in the user's catalogue is essentially free and not restricted as to number and size of files, especially when the user mounts his own discs.
4. The user need only have a basic system consisting of a terminal and microprocessor CPU and disc drive.
5. Peripheral devices can be located in close proximity to the user and to one another even if the number crunching is done at a remote site. Therefore the user can have instant access, assistance from similar users and up-to-date documentation.
6. Microcomputer systems are now readily available with 32K to 256K decimal words of memory at reasonable cost.

An arrangement whereby all pre- and post-processing is carried out on a local microcomputer and all intensive number crunching is carried out on a large remote mainframe optimizes the total design turnaround time.

At McMaster University the pre- and post-processors have been implemented on a PDP 11/23 in our laboratory.

This microcomputer uses 16-bit words and has a user area of 32K decimal words of memory. The user interfaces with the machine using a VC1452 CRT terminal communicating at 9600 baud and a LA120 line printer communicating at 1200 baud. The external package executes on a remote CYBER 170/730 mainframe with 140K octal 64-bit words of memory. All software is written in fully transportable ANSI FORTRAN.

This system has been extensively used in instructional workshops on hydrology and hydraulics for consulting and municipal engineers, where the participants get "hands-on" experience with a small part of the urban drainage simulation package. The participants are expected to work from actual plans for an existing urban area, discretize the sub-areas, prepare and/or modify the input data, and apply an actual recorded rainstorm. Many jobs are run during the 3-day workshop to gain a "feel" for the model of the drainage system.

Engineering reports generally incorporate modelling results. By integrating the post-processor with a text processing system operating on the same microcomputer, output files created by the post-processor can be substituted directly into a skeletal outline of the standard sections of the report.

We have concentrated on small to medium sized consulting engineering firms in designing the interactive procedure and acquiring computer hardware. Most such firms use remote batch-oriented computing purchased from a commercial computer centre. It has been our experience that this mode of computing is not as efficient as spending $25000 to $35000 on hardware, and interactive conversational software for communicating with a large simulation program at a remote mainframe. The firm can thereby reduce the computational effort and in all likelihood reduce its costs.

CONCLUSIONS

Design time, including user transaction time and program turnaround time, are greatly reduced when the interactive approach to computing is taken. Concise, unambiguous man-machine dialogue promotes a reduction in user transaction time, minimizes the potential for error, increases user confidence and encourages investigation of different design alternatives. Transmitting only key output to the terminal enhances convergence to a final design.

Conversational interactive pre- and post-processing procedures greatly facilitate preparation of input data, job submission, output retrieval and interpretation and subsequent investigation of possible solutions to multi-objective problems. The original coding of the external packages remains untouched thereby facilitating incorporation of new versions and/or updates whenever available.

The best operating environment at this time is one in which the pre- and post-processing is carried out on a local microprocessor, devoted exclusively to a small user group. The large external simulation package executes on a remote high-speed mainframe with large available core memory. Thus, total design turnaround time is minimized, cost to the user decreased and the quality of the final design improved.

At McMaster University, a PDP 11/23 microcomputer hosts the pre- and post-processors. The microprocessor communicates at speeds up to 9600 baud with a CYBER 170/730 mainframe, on which the simulation package resides. The post-processor is easily interfaced with a text processing system to reduce the time involved in producing a draft report.

The hardware and software described in this paper is appropriate for general use and in particular for small to medium sized consulting firms actively involved in computational urban drainage design. The methodology will increase design capabilities and improve efficiency at a relatively low cost. The general concepts of the interactive system described in this paper can be extended to other disciplines.

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COMPUTER APPLICATIONS D'ORDINATEUR

HYDRO-MUNICIPAL

TORONTO, ONTARIO
MAY 18-19, 1978

EDITOR'S NOTE

The Specialty Conference on Computer Applications in Hydrotechnical and Municipal Engineering was organised and sponsored by the

Computer Applications Division
Hydrotechnical Division
Municipal Division

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and

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The Conference was held in the Downtown Holiday Inn, Toronto, on May 18th - 19th, 1978.

The Proceedings contain the papers presented at the Conference.

NOTE DE LA REDACTION

Le Colloque sur les Applications des Ordinateurs en Hydrotechnique et Génie Municipal a été mis sur pied par

la Division des Applications des Ordinateurs
la Division Hydrotechnique
et la Division de Génie Municipal
de la Societe Canadienne de Génie Civil sous l'egide de

l'Université de Toronto

et

l'Université McMaster, Hamilton, Ontario.

Le colloque s'est tenu dans le 'Downtown Holiday Inn', Toronto, les 18 et 19 mai, 1978.

Ce volume réunit les textes des communications presentées au colloque.

FAST SWMM

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Abstract: A new package called SWESWMM has been developed and used by the author at two locations in Sweden. The basic program is a transformation of the CANSWMM package distributed by the Ontario Ministry of the Environment, and is easily maintained by simply updating the basic SWMM modules in accordance with the latest modifications and corrections issued by the original authors. These basic modules have in no way been modified by the present author. Program SWESWMM is written in ANSI fully transportable FORTRAN.

The basic processes included in the package are:

a) solicit and accept user-directed basic SWMM input in conversational free-format mode, usually from a remote terminal
b) solicit and accept design-oriented input data (if any) from input devices or units as directed during execution by the user
c) check the validity of the input and return values of certain dependent parameters to the user terminal
d) convert from metric units to the British Units required by SWMM, if necessary
e) submit the blocks of SWMM in the order required
f) return selected output from the SWMM output file to the user terminal, and reconvert to metric data if required
g) re-submit the SWMM blocks as directed and return selected output, and repeat as required
h) save and list input/output files as directed by the user

Steps (e)-(h) are written in simple systems control statements and should be reasonably easily adapted to different systems.

Steps (a)-(d) are handled by program SWESWMM. The design procedure has been carefully designed to minimize user errors and reduce total design turn-around times. The paper describes the urban drainage design and SWESWMM design criteria. The SWESWMM package evidently considerably reduces the complexity of SWMM job submission; users are able to focus on the simple hydrologic, hydraulic and water quality processes modelled. No knowledge of FORTRAN formats, or systems control language is required of users. SWESWMM accepts both CANSWMM and SWMM jobs without distinction.

INTRODUCTION

Urban drainage problems arise from both the quantity and the quality of runoff. So far as quantity is concerned urban runoff models are essentially similar to the general hydrological model in the flowchart below:

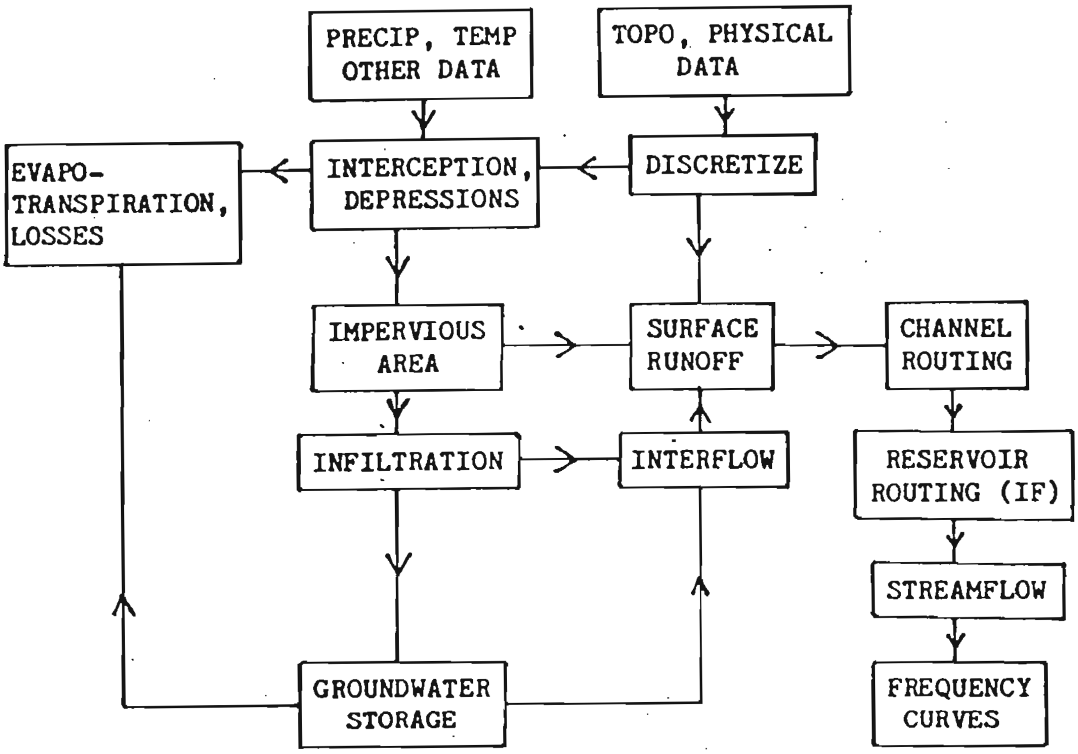

FIGURE 1

In practice there is little difference between flood control management and urban drainage management - we have to get any excess runoff away with minimum damage and cost.

The textbook approach to runoff control requires determination of a flood frequency curve for the "as is" condition and a second flood frequency curve for conditions after completion of a particular runoff control system. These curves can be converted into damage frequency curves by assuming a relation between peak flow and flood damage. The area under the curves then becomes average annual damage in the "as is" condition and average annual damage after improvements have been made. The difference between these two damage figures represents the benefits of the flood control project and can be compared with the costs for an economic evaluation. In many urban situations the damage is little more than nuisance and a decision is made rather arbitrarily to limit the probability of this nuisance to some acceptable level.

In dealing with pollution from urban storm runoff, the mangitude and frequency of the pollutant loads should also be known. A determination should be made of an acceptable level of pollution frequency given some information on the magnitude of the polluting load.

Consequently, it is necessary in urban hydrology to define the probability of peak flows, and, where storage is being considered, the volume characteristics of the streamflow as well. Unfortunately, a model designed to simulate only a few large discrete storm events cannot as readily provide the probability information which is needed.

It is usual for example to define flow frequency by selecting the maximum rain hyetograph each year and converting this to a flow hydrograph with a discrete event simulation model. But the most intense rainfall in a year does not necessarily produce the maximum peak or the maximum runoff volume [1]. A short high intensity rainfall may produce a very large peak flow but with a low runoff volume such that it will be severely reduced by available storage. On the other hand, modest rainfalls extending over many hours or even days, may produce a large volume of runoff which could fill a storage reservoir. The exact effect will of course depend on the amount of storage and its outlet capacities.

At present there is no suitable continuous simulation model available for urban drainage design and it appears to be advisable to use a tried-and-tested model such as SWMM in a flexible manner. The development of SWESWMM is an attempt to provide rapid and easy design use of the SWMM package. SWESWMM could also be extended to include other proven hydrologic and hydraulic packages.

THE GENERAL NATURE OF URBAN DRAINAGE DESIGN

Before we go any further, we must honestly face up to the reality of urban drainage design: it is an improvisation, a provisional lash-up.

We can design no drainage scheme that works properly, nor can we even ever achieve a truly satisfactory performance. It is not possible to create a storage facility that has no unfortunate or undesirable effects. The requirements for good drainage design are generally irreconcilable, and all designs are in a sense failures. Compromise implies a degree of failure, for example, durability vis-a-vis low first-cost.

The urban drainage designer has limits set upon his choice:

1. The drainage facility being designed must correctly embody the essential principle or the fundamental use of the facility - a detention pond, for example, will accumulate water. We cannot bluff our way around it.
2. The drainage components must be geometrically related as required in extent and position to each other and to the overall system.
3. The components must be strong enough to resist the required hydrostatic and erosive forces, and big enough to convey the flows.
4. Access must be provided.
5. Cost must be acceptable.
6. Appearance must be acceptable.
7. Side effects must be acceptable.

Drainage design always involves making trial assumptions based on experience; like research, design is a matter of trial and error. Design relies heavily on memory; a totally inexperienced person cannot design a good drainage scheme. Furthermore the purpose of the facility is not unique.

Purpose could be defined: "What someone has provisionally decided that the facility may be reasonably expected to do for the present". It is not an innate property of the facility. For example, the purpose of a dam is any purpose imputed to it by any man; to the contractor the purpose is to make money, to the "designer" it may be to win a competition, to the paddler it is flotation to move to some other point, but to me it may be to keep ducks.

The influence of economy in drainage is universal; the characteristics that lead people to call a design "functional" are invariably derived from the requirements of economy and not of use.

The principle ways of keeping down costs are:

a) to use as little material as possible,
b) to use the cheapest materials available,
c) to use the most easily worked materials,
d) to use mass-production techniques,
e) to use as little skill as possible,
f) to use as few operations as possible,
g) to use the cheapest energy available,
h) to design and construct it as quickly as possible.

Computer models are clever devices, and their builders sometimes seem to claim that they will show us the correct and cheapest answer. But there is no optimal design. Even if we were able to deliberately pre-determine mathematically the solution that would actually turn out to be the most economical in long-term practice, the client would probably still want something cheaper or more striking.

The general design procedure in urban flood control may be thought of as a series of nested loops as shown Figure 2 [2].

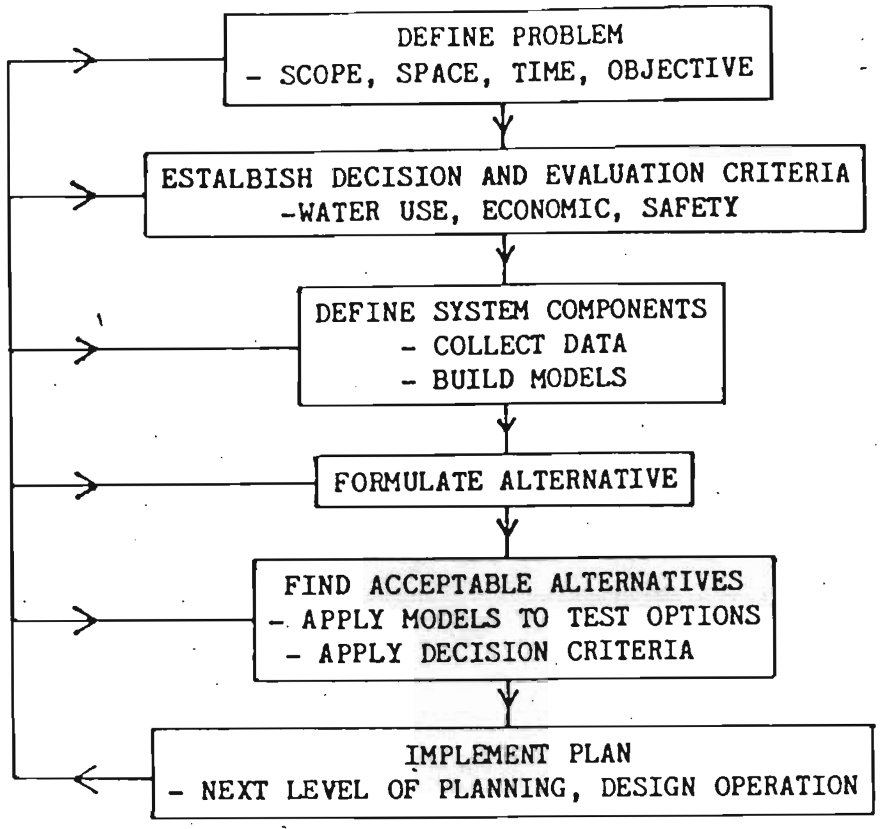

FIGURE 2

CURRENT URBAN DRAINAGE DESIGN PRACTICE

Traditional storm drainage design philosophy was to collect runoff and carry it away, as fast as possible, out of the boundaries of the watershed, by connecting all impervious areas, such as roofs and driveways, to a network of gutters and conduits with considerably higher flow velocities and density than the natrual drainage system [3]. Storm water was considered "clean" and there was no concern about pollution from separate storm sewers. Negative consequences of this were dramatic increase of peak flows at the outlet of the urbanized watershed, increased incidence of local flooding, depletion of ground water, considerable increase in the cost of new storm sewerage systems and relief sewers, and environmental drainage. The design of storage is now generally agreed to be possible only through the synthesis of hydrographs (storage in an urban system is not necessarily concentrated in a reservoir but may be distributed over different elements of the watershed, such as roads, parking lots, roofs and elements of the sewer network.)

In fact, it has recently become common to consider two extreme drainage systems in one urban area. One is a minor system consisting of closed and open conduits and the other is a major system, which is the route followed by flood or runoff waters when the minor system is inadequate. Traditional urban drainage studies considered only the minor system. Nowadays both extremes should be analyzed in a drainage project. Typcial minor systems are usually intended to have sufficient capacity to collect and transport runoff from a storm that might be expected to occur only once in a 2 to 10 year period. Return periods from 25 to 100 years might be considered for the major system.

Detention ponds which are part of the minor system should be designed with consideration of possible effects of major floods. It is desirable that the same methodology for the hydrologic simulation be applied to both systems. The elements in the simulation of urban runoff are as follows:

During precipitation on a pervious area, water is continuously abstracted by infiltration, and initially by depression storage. Other abstractions considered in rural hydrology, such as interception and evaporation are usually negligible for urban storm events. If the rainfall intensity exceeds the possible rate of infiltration, natural depressions collect some of the excess precipitation creating additional depression storage. Since the depressions have different depths and filling rates, part of the overland flow commences as soon as the infiltration capacity is satisfied.

Comparisons considering only a few storms on a couple of test areas are useful for the calibration of parameters but are not very conclusive as a test of a model's performance. Practical computerized optimization techniques for this purpose are not available at present; simpler intuitive procedures are usually sufficient for model improvement.

The cost of modelling is determined by the effort in input data collection, program input requirements and computer costs. All of these factors are also dependent on the degree of schematization used. Computer costs are not large for discrete event models when applied to basic subcatchments (less than $10.00) so that hydrologic model selection on this basis is not warranted.

OPTIMAL SIZING OF DRAINAGE COMPONENTS

Flood-control measures within urban areas may consist of storm sewers, detention storage reservoirs, channel modifications, land-use controls, levees, flood proofing, and pumping facilities. A range of alternative system configurations and component sizes can usually be identified that will accomplish a specific technical objective, such as a specified degree of protection. The need to determine the appropriate size of the components of the system has stimulated efforts to formalize the analysis of tradeoffs between facilities, performance, and costs. For example, there is a combination of best sizes for each component in a system that would maximize the system's net value or accomplish a performance standard most efficiently [4].

Most models are flow-prediction "models" for which the time variation of storm runoff can be computed if the layout, slope, length, and size of the storm or combined sewers are known or assumed. These flow-prediction models are useful from an operational viewpoint for regulating storm runoff for pollution control and other management purposes. However, they cannot be used for direct determination of sewer sizes. If these models are used for design, the sewer sizes have to be determined by trial and error. Most models do not utilize the unsteady flow equations or account for the mutual backwater effects between the sewers in the network. SWMM utilizes slightly simplified flow equations accounting for the backwater effects within each individual sewer but without considering the mutual backwater effects among the sewers.

In the past, complexity and uncertainty in sewer design has been largely handled by imposing rather conservative general design guidelines which simplify the design decision but tend to mask fundamental functional relationships and system cost considerations. It is imperative that the fundamental bases and the design methodology be carefully examined when computerizing design procedures to see if they might be beneficially modified to achieve a more sound functional or economic design. Computer models usually reduce the uncertainty by accounting for the greater complexity of the real system.

It appears that the optimizing sewer design programs may result in reduced system costs but are even more valuable in providing rapid and inexpensive cost sensitivity and alternative design capability, thus providing the engineer with valuable design and decision information. Overall cost savings of about 10% or 20% or more may result from this process [5,6].

Storm sewer systems can also be designed based on cost optimization considering risk due to uncertainties in the design parameters. Models to account for the costs of expected flood damages have been incorporated into the dynamic programming solution procedure to obtain the optimum sewer system design. Discrete differential dynamic programming is applied to make the design optimization procedure of a large storm sewer network become feasible by significantly reducing the computer time and computer storage involved. The numerical examples cited in the literature appear to demonstrate that the expected damage cost is generally not a major part of the overall expected cost of the sewer system. Therefore, precise evaluation of the exact risk level is not necessary and approximate analysis of uncertainties and risk evaluation may prove to be adequate [7]. The use of a risk-based cost optimization scheme may provide a rational yet practical approach in the design of storm sewer systems, particularly when considerable flood damages are expected.

GENERAL DRAINAGE DESIGN STRATEGY

SWMM flow simulation has proven to be accurate in a large number of applications for areas of greatly differing size, land use and sewer system configuration, and over a range of meteorological conditions. Consequently SWMM has been recommended for the simulation of flows, where required, in drainage planning and design studies, although particular advantages offered by other models in certain specific situations should not be overlooked. For example, STORM is particularly useful for the screening and comparison of alternatives and for the isolation of critical events for subsequent SWMM simulation [8].

At present, the modelling of runoff quality cannot be expected to be as precise as the simulation of runoff quantity. It is considered that some sampling of the quality of runoff and overflows will usually be required to supplement the final modelling activities in a study directed towards the design of pollution abatement facilities. Past experience with the interpretation of the results of sampling programs, and subsequent efforts for calibration, have emphasized that the collection of samples for chemical analysis should be very carefully planned in view of the high costs involved. Preliminary modelling is often useful in determining the specific sampling locations required.

Simulation of storm water quality will be useful, however, in a variety of studies, such as the assessment of the requirements for upgraded treatment plants, the need for new plants, the benefits of sewer separation or new interceptors and the control of runoff from new developments.

The costs of treatment and, to a lesser extent, storage facilities, are partly a function of the design flows. Although the modelling of quality is by no means exact, accurate flow simulation is very useful for the design of any pollution abatement facility. Even if the calibration of the parameters controlling quality is limited, quality simulation can still be of value for illustration of the severity of a problem and comparison of alternative pollution control schemes. Consequently, quantity and quality modelling should form an integral part of all studies for pollution abatement [8].

The design methodology includes three stages: data preparation stage, the planning stage and the design/analysis stage[9].

In the planning stage, various land use alternatives, drainage systems and the resultant pollutional impacts on the receiving waters are evaluated. Typically, at this stage only limited information describing the watershed is available and, consequently, a detailed runoff simulation would not be feasible or appropriate. At the same time, it is important to establish the probability of occurrence of runoff events of specific magnitudes. The SWMM model could be applied in a simplified (lumped) manner for a limited number of critical precipitation/runoff events, the frequencies and antecedent conditions of which would have been determined by some other method. SWMM can be applied as a spatially lumped model without a significant sacrifice in the accuracy of simulation[10,11]. Reduction of the number of elements in the RUNOFF and TRANSPORT blocks to a minimum results in a considerable saving in data reduction time and in computer time requirements.

Once the hydrographs and pollutographs have been simulated by SWMM, different runoff (or overflow) control alternatives may be studied. SWMM can consider seven levels of treatment and estimates and, very approximately, the costs of implementing these control alternatives. In some instances, a preliminary analysis of the impact of critical discharges to the receiving waters would also be carried out in the planning phase. At the completion of the planning stage, the user has a good indication of the nature of the runoff or overflow problem in the study area and also has a feeling for the effectiveness of various runoff/overflow control measures for a number of critical rainfall/runoff events. The information obtained during this phase of a study is at a planning level, where the relative effects and magnitudes are more important than the absolute values required for subsequent design purposes.

In the Design/Analysis stage, the design of drainage system and pollution control facilities is carried out, as well as a detailed study of probable impacts in the receiving waters. Consequently, it is desirable to produce accurate hydrographs and pollutographs for selected events using a calibrated, detailed simulation model. At this level, SWMM would be used at a high level of discretization.

Sewer surcharging would not have been considered at the planning stage, as sewers are considered as an open channel network at that stage. Critical events for pollution abatement are not usually the low frequency storms used in design. However, surcharging becomes very important when analyzing methods to reduce flooding in an existing sewer system of insufficient capacity, or when evaluating the response of a drainage system to a storm of a frequency lower than the design frequency. Under such circumstances, SWMM is not applicable and other more sophisticated models are required. For simulation of backwater effects which might occur in the sewer system, it becomes necessary to use a model incorporating dynamic wave routing.

THE BASIC SUBCATCHMENT SYSTEM IN SWMM

The urban watershed drainage system may be considered as two distinct systems for runoff modelling purposes[10]:

(a) The basic subcatchment system in which the excess rainfall is transformed to overland flow. The overland flow hydrographs at the drainage manholes form the inlet hydrographs for the transport system.

(b) The transport system in which the flows are routed through the trunk sewer network.

Each subcatchment is conceptualized as a flow plane over which overland flow occurs. Representative values of imperviousness, average ground slope, infiltration, retention storage, and land use are assigned to each subcatchement. The rainfall depth over each subcatchment is the basic input to the overland flow or RUNOFF model. The model sequentially accounts for infiltration and retention depth to determine a net excess rainfall depth on the overland flow plane. The excess rain depth is used to compute an overland flow rate for each time step. SWMM uses a very simple algorithm for this runoff computation. Subcatchments can be used to represent large portions of the entire watershed, or individual drainage areas, depending upon the desired degree of detail. The small local sewer pipes and gutters are usually ignored in large subcatchments. SWMM also provides the facility to account for simplified routing in local pipes in the RUNOFF block. Instead of considering the subcatchment overland flow hydrograph to flow directly to the inlet manhole, it can be routed initially through a "GUTTER" subroutine which is used to account for the many ditches, lateral pipes and street gutters, which are not usually included in the TRANSPORT block computations. For more details the reader is referred to the SWMM manuals[10].

OBJECTIVE EVALUATION OF SIMULATION RESULTS

Once calibrated hydrographs and pollutographs have been obtained from the detailed model, the storage and treatment design can be finalized, following an evaluation of the environmental impact of the effluent or overflow on receiving waters.

SWESWMM includes one statistical measure to evaluate the accuracy of the hydrographs computed by SWMM, when compared to the entire recorded hydrographs; the linear Correlation Coefficient (R)[3].

If the observed hydrograph is denoted O, and the computed hydrograph C, then

$$R = \frac{N\left(\sum_{i=1}^{N} O_i C_i\right) - \left(\sum_{i=1}^{N} O_i\right)\left(\sum_{i=1}^{N} C_i\right)}{\left\{\left[N \sum_{i=1}^{N} O_i^2 - \left(\sum_{i=1}^{N} O_i\right)^2\right]\left[N \sum_{i=1}^{N} C_i^2 - \left(\sum_{i=1}^{N} C_i\right)^2\right]\right\}^{1/2}} \qquad (1)$$

where N is the number of observations of O and C. R has the following properties:

(i) $-1 \leq R \leq +1$

(ii) The closer the value of R is to either +1 or -1, the better is the agreement between the two variables for the assumed linear relationship.

DESIGN CRITERIA FOR INTERACTIVE DIALOGUE

More significant than computing cost is the cost of the designer's time. We have found that using one program in batch mode may require one day's effort, depending on the designer's capability, whereas savings of about 85% of turnaround time can be achieved using an interactive package[12]. The costs of that time difference may be quite significant to consultant and client alike, especially from remote small offices.

It has been our experience that the design time, including user transaction time and program turnaround time, are greatly reduced when the interactive approach to computing is taken[12]. It has been found that concise, unambiguous man-machine dialogue promotes a reduction in user transaction time, minimizes the potential for error, increases user confidence and encourages investigation of different design situations. It was also found that transmitting only key output to the terminal enhances convergence to a final design.

The importance of making the numerical model work for the user cannot be stressed too greatly. If the method of communicating with the computer is complicated and exacting, or the dialogue ambiguous, the positive aspects of computing will be nullified. There are three levels at which attention should be paid to user psychology - the functional level, the procedural level and the syntactical level[13].

At the functional level, it must be made clear which functions are to be carried out by the designer and which functions by the computer. A brief introduction at the beginning of the program is important. In SWESWMM the user is informed that he must supply data whenever three dots (...) appear. He is made aware that the computer will do all the prompting. The user can devote his time to the design problem.

At the procedural level, the software must be designed so that the computer prompts input by displaying alternatives. The alternative chosen will direct the program logic along a certain course. The designer determines the course by selecting the best alternative.

The syntactical level is the one at which most of the guidelines for achieving effective interactive software can be proposed. The manner of presenting prompts for input and displaying output is the area in which the greatest scope for improvement exists. The way data are formatted can have a profound effect on the user's efficiency and error rate. Consideration should be given to the following[13]:

(i) The dialogue should be concise and unambiguous. The prompt for input should carry a tone indicative of the form of response required.

(ii) Prompts should be concise to minimize the chances of different interpretation by the user. Associated with each prompt there should be a number to allow the user to refer quickly and easily to a pocket manual for aid. This is especially useful when numerical quantities are to be entered. The manual should outline values the parameter would take in a variety of circumstances. The prompt should always be presented in the same manner. The signal that input is required (in this study three dots (...) were used) should appear in the same position, for example, forty spaces from the left hand edge of the paper, so that the user becomes accustomed to the manner in which input is requested and entered. It has been found that this minimizes the chance of error.

(iii) The user should not be made to respond to more than one idea at a time. This will avoid confusion and frustration and will again minimize the possibility of error.

(iv) The program's input routine should be designed to accept data in a free format mode. Users, particularly if not familiar with FORTRAN, may become confused if they are suddenly faced with having to enter data in a specific format, for example F10.3.

(v) The computer should always respond to the user. When a response is sent by the user some indication should be given him that the response has been accepted or rejected. This may be accomplished through a specially designed message or by having another prompt appear. It follows therefore that all input requests for the particular stage of the program operating should be made in proximity to one another.

(vi) Error messages should be designed so that they convey information in a concise manner yet do not antagonize the user. Carefully worded error messages will encourage the user to try again without giving him the impression that errors will be tolerated continually.

(vii) Results relayed to the terminal should be presented in an ordered yet easily readable form. They must convey only essential data yet they must be clearly labelled to indicate their significance. It must be remembered that further input will likely depend on these results. If the user cannot decipher them he cannot supply additional data and the design sequence breaks down.

Control of User Errors

Users are human and because of this they will make mistakes. It is essential that the interactive system detect these errors, as far as possible, before they can affect the design, and inform the user politely. Several dialogue design criteria should be kept in mind [13]:

(i) The dialogue must be structured so that as many errors as possible may be detected immediately.

(ii) The dialogue must provide the user with an unambiguous description of what the error is and why it occurred.

(iii) The system must provide a facility for immediate correction of errors.

Error messages should provide information and encourage the user to avoid repeating the mistake. However, the system should be designed so that the user is not encouraged to make mistakes. The interactive system should output a diagnostic at the terminal whenever a user input error is detected.

The Design Dialogue

SWESWMM follows the regular prompt/read sequence with all prompts numbered for hand reference to a pocket guide. The input format is similar to that of HYMO and the HEC programs, except for the following allowable commands [14]:

STOP
BACKSTEP (Start this step again)
CHANGE (Change an alterable variable)

Thus commands are expressed in at least the first 3 columns of the data card, and columns 4 through 80 are used for numeric data and keywords.

The data can be written in any format, but at least one blank space or comma must be left between data items. A decimal is required for numbers containing fractions, but not for whole numbers. Keywords can be written with the data to describe individual data items.

However, the input may also follow the formats required for regular batch job submission. In other words, if the job is to be set up for remote job entry, the input data files may be prepared in accordance with the rigid SWMM formats.

The relative merits of batch processing, and interactive time sharing may be considered here. Batch processing is defined as a process in which a run is submitted to the computer either through a high-speed device such as a card-reader or through a low speed device such as a keyboard terminal, and the job is then run and the output returned to the user either immediately or at some later time, with no communication between the user and the program from the time of submission to the receipt of results. Interactive computing is defined as a process which may be initiated in either of the ways described above for batch processing, but in which the program will halt at certain predetermined points to permit the user to make certain decisions before the run is continued. One further term should be noted, i.e. conversational computing. This is similar to interactive computing, but specific prompting questions are written into the program so that a pseudo-conversation can be carried on between the program and the user in an effort to prompt or guide the user in the making of the run time decisions. The decision as to whether batch or interactive computing should be used depends on two issues: the most appropriate method of use for the engineer, i.e. whether or not it is important for him to be able to interact or converse with the program in terms of the design function which he is attempting to carry out, and the data which is required to set up the various runs of the system.

It is of course possible to access SWMM in an interactive mode via keyboard terminals using existing control software, i.e. without use of SWESWMM. This could permit an engineer to set up a major run in a batch mode and then, having completed a first run, modify certain input parameters via a keyboard terminal, retain the rest of the data unchanged, and initiate another run. Many users of SWMM use this method[8]. The main object of this paper was to discuss the total urban design problem and the design criteria for SWMM, so that the reader may himself decide whether to use a program such as SWESWMM.

CONCLUSIONS

Most users are faced with a number of realities[11]:

(a) They cannot afford to develop and maintain an independent urban runoff model.

(b) They do not own a computer large enough to run a good urban runoff model.

(c) They have good telephone access to such computers and can afford a portable terminal.

(d) They are discouraged by the long learning-time required to become familiar with good urban runoff models.

(e) They would like to reduce the total design turnaround times associated with batch computing.

I have tried to develop SWESWMM to cope with this environment. The main principles were that:

(a) It should communicate directly with a standard SWMM.

(b) The SWMM must be easily updated locally on receipt of updates from the original authors.

(c) SWMM should run without long delays because of excessive memory requirements.

(d) SWESWMM should operate optionally in SI units but talk to SWESWMM in the required FPS units.

(e) SWESWMM should function on portable slow-speed terminals in conversational manner.

(f) It should be easy to include other models such as STORM.

We have thus not inserted any code into SWMM except that required to make isolated statements transportable (about half-a-dozen statements).

The mode of operation may be depicted:

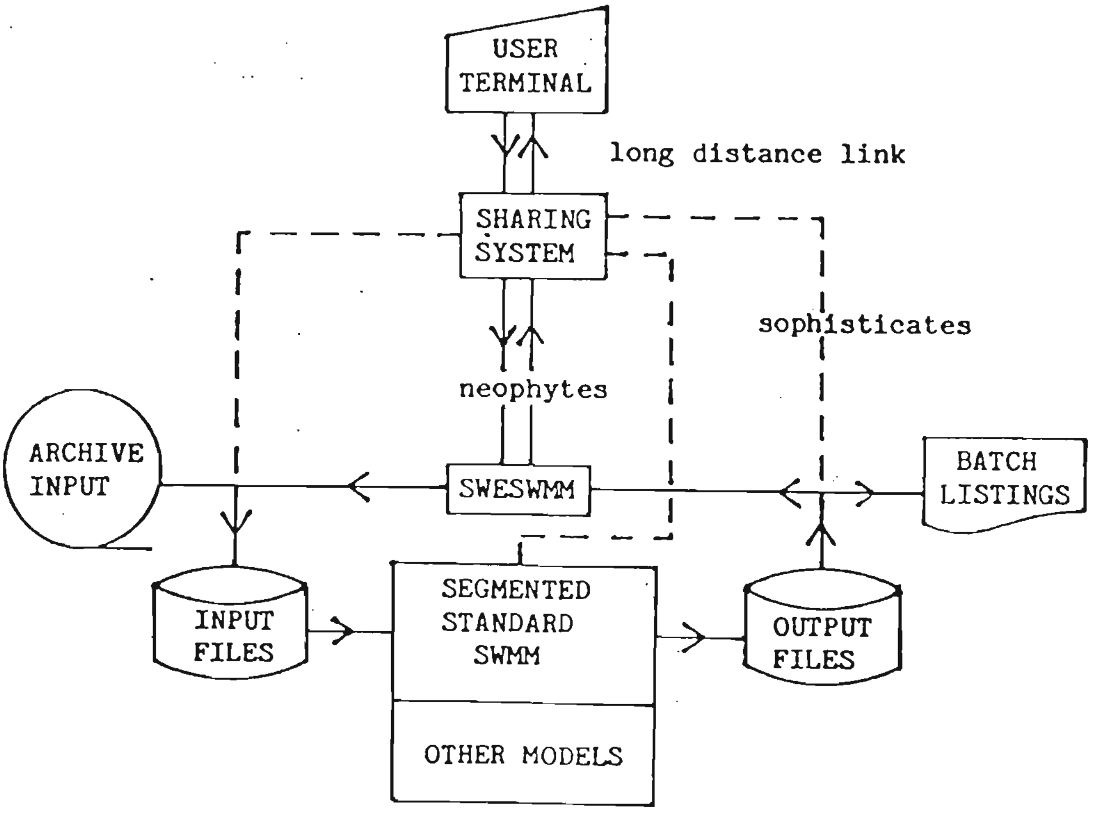

FIGURE 3

Thus the operations of SWESWMM are:

1. CONVERSATIONAL INPUT

2. SI/FPS CONVERSION

3. ARCHIVE INPUT

4. CONNECT INPUT FILES

5 RUN SWMM (& OTHER MODELS)

6. ACCESS OUTPUT

7. FPS/SI CONVERSION

8. PRINT SUMMARY OUTPUT

9. FULL REGULAR BATCH LISTINGS

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PROCEEDINGS OF THE 4th ONTARIO UNIVERSITIES COMPUTING CONFERENCE

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DEVELOPMENT AND APPLICATIONS OF LARGE INTERACTIVE CAI (SIMULATION) PROGRAM PACKAGES IN ENVIRONMENTAL SCIENCE

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A B S T R A C T

An advantage of realistic computer models of complex engineering systems is that local and easily accessible prototype systems can be simulated. Site visits to local prototypes become more meaningful if students are to simulate control of the self-same systems. Experience of field visits to such water resources systems indicate the impact of the simulation package on the curriculum.

In order to allow reasonable mastery of the response of more realistic models additional time and effort may be required of the student. To a limited degree, this can be accommodated by a rearrangement of the lecture material in the established curricula. However, there may be compromises and mastery levels of "fundamentals" may be threatened. A set of guiding principles for the development and use of simulation packages is suggested.

A cursory cost comparison between this "simulate-and-site visit" approach and the traditional approach (simplistic laboratory model and hand computation) is offered. An overview of design considerations, programmable features and classroom requirements are also discussed.

INTRODUCTION

A large CAI simulation package has been used for the past two years in the undergraduate Civil Engineering Water Resources programme. Our experience indicates that teaching and learning are significantly influenced, and that such CAI packages could prove more cost-effective than traditional alternatives. We present the general developmental background in this part of our paper.

A General Method

From the outset our aim has been to develop software suitable for administering any simulation problem. The subroutines may be categorized as follows:

a) Administrative Software -
"Bombproof" input software (e.g. format checks)
General output software (e.g. paging)
Hardware-specific software (e.g. special graphics)
General CAI software (e.g. private records)
Driver and control subroutines.

b) Simulation Software -
Design-mode input (including echo-checking)
Quiz-mode input (using random number generators)
Simulation algorithms

c) Courseware -
Branching logic (e.g. error-correcting)
Input/Output dialogue.

Our experience has been that the programming for group (a) has been the most difficult, whereas few problems have arisen in group (b). Programming for group (c) has proved a never-ending labour.

The method may be adapted for any suitable course in the hard or soft sciences, i.e. any simulation algorithm may be easily inserted in the package. Appropriate course-ware would have to be developed (of course); our methods have stressed flexibility here.

Transportability

It was clear from the outset that CAI (simulation) packages have an economic advantage over the traditional laboratory method as soon as the software is distributed among several users. We have accordingly tried to avoid system dependent forms wherever possible, e.g. using variables to specify the input/output devices.

The package was developed on a CDC6400 computer using INTERCOM 4.1 under the SCOPE 3.4 operating system, but is written mostly in ANSI FORTRAN. An early version of the package was transferred to a GE computer and required only a few minor modifications before running interactively.

Moreover the package has been adapted to five different terminals, two of which have graphics capabilities. Most graphics software, and certain optional administrative software, are system dependent, but we are prepared to assist any potential users in adapting the package to their systems.

General Package Features

We list a number of the program design requirements in the concluding part of this paper, but four features warrant particular mention at this point:

a) Students are able to access the entire gamut of course-related exercises from the beginning of the course. To further enhance this self-pacing property, the exercises are arranged in a hierarchy of complexity such that additional instruction may be obtained on component processes. The student may transfer to any exercise or complexity level at any stage.

b) Two user modes are available for each exercise, denoted "diagnostic" for quiz mode and "prognostic" for design mode. In the quiz mode the input variables are selected internally pseudo-randomly, and then varied in predetermined steps. The student is then required to diagnose the causative parameter from the plotted output. In the design mode the student enters his own input parameters in order to acquire a grasp of the system sensitivity, or to achieve a desired (design) output.

c) If during an exercise the student requires help he may access a file cataloguing such information. At first this file may seem to be open ended, but in practice students tend to experience very similar difficulties. The file is (to be) built up from the private record of individual exercises, and stored in tree structure form. The didactic value of this subroutine should not be underestimated, since one of the most important skills in engineering is learning to formulate correct questions. Frequently the file stores better, or more relevant, questions than a student may currently be capable of formulating.

d) An important requirement from the start was that the package should operate efficiently on portable terminals. We consider that the student should be able to participate in an exercise whenever and wherever he chooses, and that the quiet of his own room at night, if he has access to a telephone, is probably most suitable. For this reason all software can be accessed from several terminals simultaneously.

PEDAGOGICAL ASPECTS

In this part of our paper we present the specific application of our software package (MACCOM) to instructional procedures in Water Resources Engineering.

Societal Responsibility

Civil Engineering is predominantly occupied with modification of the natural environment to suit the requirements of man (in contrast to medicine). Water Resources Engineering is one of many fields within the ambit of the Professional Civil Engineer; it embraces the planning, design, construction and operation procedures connected with the conservation, control and utilization of water to satisfy the social, esthetic, economic and physical needs of people.

One such need is reduction of flood risk, and MACCOM currently includes subroutines in this area. Our responsibility here is clear. Floods are amplified by most forms of river basin development. Floods are costly, and cause deaths. Civil Engineers carry this can in both the environmental and judicial arenas. Society requires responsible education of these professionals; MACCOM is one attempt to meet this within the constraints of a turgid undergraduate curriculum.

Educational Background

McMaster University Civil Engineering undergraduates do not receive instruction related to the movement or mechanics of the flow of water until their third year. Then, in the first term, a course on the fundamentals of fluid flow is required of all engineering undergraduates, and in term two a wide-ranging course on hydraulics (including an elementary introduction to certain hydrologic processes) is required of all Civil Engineers.

The fourth year programme is entirely elective, but most Civil Engineers select the Water Resources option, which is analytic in approach, in their first term. (In the second term a design-oriented course in Water Resources Engineering is now also being offered.)

MACCOM currently forms part of this fourth year first term course, and forms the basis of the assignments, or tutorial component.

Traditional Teaching Method

The treatment of fundamentals of fluid flow is, traditionally, thoroughly rigorous, and solutions to problems involving even simple geometry require considerable simplifying assumptions. Furthermore, computational effort is traditionally of the hand (slide-rule) character, and consequently the laboratory or tutorial exercises are necessarily restricted to extremely idealised geometry, such as rectangular open channels, prismatic storage tanks, and conduits of regular cross-section. The approach may be summarised:

a) analytic formulation of the process in mathematical symbolism implies the (only) sound grasp of the fundamental mechanism of the process,

b) long-hand calculation implies the (only) sound familiarity with that analytic formulation,

c) experience with a laboratory model of the idealised computational concept is (only) beneficial, not the least because it demonstrates certain shortcomings of the conceptual models.

Modern Problems

There are arguments that such narrow and introspective pedagogy has led to large scale environmental problems, and impeded interdisciplinary communication. Certainly many environmental problems are directly attributable to narrowly conceived Water Resources developments*, including dams and irrigation. Simulations of aquatic systems demonstrate complex interactions, simple mathematics, and the need for wide perspectives and meaningful interdisciplinary collaboration. For example, some difficulty is being experienced with "swimmer's itch" in recreational dams in North America. The shistosomes are normally parasitic on ducks and snails, and evidently invade man accidentally. Snail ecology is important, and its assessment requires a study of plant life much affected by nutrients carried into the body of water concerned. Birds are attracted to the area by fish, and man by both. It is a challenging problem, and, like shistosomiasis (the world's number one disease) spread by water conservation works. At this time it appears that "sound" engineering offers the best hope of amelioration.

Problem-Solving Environment

Relevant simulation algorithms are being developed and improved at most institutions of higher education in the country, and many faculty possess well documented and tried-and-tested programs. Instead of, or perhaps after, using (say) prismatic tanks we could turn to any number of programs to duplicate the computation on realistic

* The root cause in reality may be greed, rather than ignorance, but we are concerned with the latter here.

topography, perhaps for a real dam. Learning may change - the equations are no longer analytic - but real world understanding may well improve.

For example one subprogram in MACCOM simulates the flood effects of sequential development of the Spencer Creek catchment in Southern Ontario. The model uses standard engineering algorithms, and seems to produce acceptable output, for simplified phases of development. Not to proffer this as part of the instructional package is tantamount to not allowing the cook a taste.

The Outdoor Laboratory

An advantage of realistic computer models of complex engineering systems is that local and easily accessible prototypes become more meaningful if students are to simulate control of the self-same systems; the learning process is considerably eased if field applications are made immediately obvious.

MACCOM in its present form relates particularly to the catchment surrounding McMaster University.* Students are introduced to the catchment early in the course, during an overflight using light aircraft, and all elements are pointed out to the students over the aircraft radio. We have not found a single student who has thereafter lost touch with the catchment features, at least for the duration of the course.

This is only one innovation directly attributable to MACCOM. Another is the use of photographs of the control devices during simulation, or of video tapes of the operation of these devices (one graphics terminal is video-compatible). Perhaps the most significant spin-off is the esprit-de-corps that always seems to follow a series of meaningful site visits.

Simulate-and-Site Visit Method

In the classroom no attempt was made to eliminate any of the traditional course content, including the long-hand computation problems. However, difficulty did arise in covering the extra ground in the time available, and certain compromises had to be made:

a) we expect less skill than heretofore in manipulating mathematical expressions;

b) we expect less memorizing of these "public" mathematics;

c) we expect less repetitive drill at hand calculations.

In order to reduce the impact of these losses, we attempt wherever possible to improve familiarity with a handbook of hydrologic principles. But on the other hand we do expect improvements in understanding of:

a) wide ranging water resources problems;

b) construction, operation and planning of water resources projects;

c) social, esthetic, physical and economic benefits of water resources projects;

d) advantages and limitations of interactive simulation of large systems.

In the classroom the simulation package is introduced by means of a teleprinter, TV camera, and individual TV monitors arranged on the student benches. Together with the usual blackboard talk-and-chalk, this has proved to be an effective means of overcoming student apprehension. But it is somewhat subject to the vagaries of computing, and system crashes or attentuated response lags wreck the best lecture.

MACCOM

The program is modular; the problem-solving routines, although logically the most important, may be any of a number of modules that are served by the set of input/output and driver modules. We repeat that program structure is so designed that the method can be easily applied to any interactive simulation.

Overlays

An overlay system is used to save computer memory. Three levels are required: main overlay, three primary overlays and (so far) four secondary overlays. This lowest level is open-ended and will increase in width as new exercises are added to the package, but without incurring further memory space. No additions are necessary in the higher levels.

* Any other catchment can be used by substituting relevant data.

Conversation

Input and output are handled by three types of routines:

a) input routine (MACINP);

b) 5 parallel output routines (MCOUTi);

c) plotting routine (PRPLOT) with associated subroutines for special terminals.

Input (MACINP)

Input consists of two types of data: words (letters) in response to questions, and numbers-supplying numerical data. MACINP contains a dictionary with keywords and their associated symbols. The dictionary is divided into logically associated keyword-symbol groups. A keyword is recognized by its symbol, which is unique within the group. At present the first letter of the keyword is the symbol, but this can be elaborated should it prove difficult to define keywords with unique first letters, e.g. in large groups. MACINP is entered with indicators pointing to the beginning and end of the group to be searched against the entered characters.

In the event of a valid entry, an error indicator is set to zero and the sequential number of the keyword within the group is used for branching in the calling program.

If a match is not found, a list of keywords within the searched group is printed and the user is given a second opportunity to enter a correct response. If the entry is again incorrect a warning message is displayed; after three errors the program branches to its end.

When numbers are to be entered the logic is different. The routine contains a table with all relevant symbols. Numbers can be entered in integer or floating point form, (i.e. I, F, or E FORTRAN format) and there is also a provision for a few written numbers, when a warning message is returned. Syntax errors in the input are detected; if the number of consecutive errors exceeds a stipulated limit, the program is routed to the exit.

At any point in the program the user may enter a routine that could answer a limited number of enquiries about the package, or terminate the program. There is an additional check here on the exit request, to avoid accidental termination.

Output (MCOUT)

All output activities in each overlay are handled by an output routine. The output routine calls the paging subroutine, with the number of lines to be printed as argument, and copies all output displayed on the terminal onto a working file for further permanent cataloguing.

Graphical Output (PRPLOT)

Results of the simulation are displayed in the form of line plots or printed graphs, depending on the type of terminal and its capability. In most cases, where teletype is used, hydrographs are printed out using a sequential digit for each curve. This is of course only an approximation to the true curve, but the accuracy is usally sufficient, using the full page (70 by 50 grid). The accuracy of printed plots is linearly dependent on the size of the grid.

Plots displayed on the 200 User Terminal utilize a 50 by 20 grid and are hardly satisfactory; curves on the graphic terminals (COMPUTEK 300 or COMPUTEK 400) are precise.

If a user restricts himself to the use of a teletype terminal, or similar, the graphics routine remains standard. However, additions and modifications have been made for various terminals to accommodate their limitations or features. The routine itself contains two important features:

a) to save memory, characters to be displayed are packed to the full size of the computer word, rather than stored one per word.

b) since display speed is important, the array is scanned from the top to the bottom and each line from right to left, to delete lines containing blanks only, in order to print only those parts of the line that contain characters.

These features render the graphics routine machine dependent, but may be omitted if necessary, since the standard version still exists.

Random Number Generator (RANDOM)

Random numbers are necessary for the simulation, and use is made of a library routine that generates random numbers uniformly distributed between 0 and 1.

The generator is initiated internally by a number resulting from real time as returned by a routine reading the computer clock. This virtually excludes the possibility of two identical simulation runs, and avoids predictable sessions. However, upon entry of a given code, the user may himself enter the initiating numbers. This is useful in debugging and developing the simulation program.

Terminals

The program was originally developed on a teletype type terminal. Since then it has been adapted to different types of terminal, to exploit such features as rate of transfer (200 User Terminal) or graphics capability (COMPUTEK 300 and COMPUTEK 400). Each of the mentioned terminals, although very similar, differ in other characteristics, such as size of the page or size of screen. These affect the input/output activities.

Although graphics terminals display hydrographs very accurately, the hard copy generated during the simulation on a teletype has proven invaluable; it allows students to refer back to the material for later study.

The terminal being used is specified early in the simulation run, and the display of hydrographs and paging of the output is adjusted accordingly.

Recording Routine

Each entire session is monitored and the record of each session is added to the existing permanent record file at the end of the session.

The record is preceded by the user's identification, date and time, and is followed by information about the duration of the session and amount of central processor time. These are also displayed at the terminal for the user's information. From these data the cost of the session can be derived and displayed. The record file is periodically printed, copied onto magnetic tape and erased.

The prime purpose of the record file is to monitor progress of individual students, and for student evaluation by the instructor. Each student is given a code from the group of defined codes allowing access to the package. If he wishes to further identify himself, he enters his initials as well.

In addition a blank code is available for practice sessions incognito.

The record file also collects user's comments, inquiries or recommendations, for use in future development of the package. The records are also used for statistical and accounting purposes.

CONCLUSION

If we accept the premise that Civil Engineering environmental problems need to be approached in a macroscopic way, and we agree to limit instruction on the continuing (microscopic) advances in the fundamental processes, then our approach may well offer educational and cost advantages, provided certain guidelines are adhered to.

Cost Comparison

To date the total "true" development costs of MACCOM are of the following order:

Computing	\$10,000*
Salaries	4,000

All costs have actually been hidden, however, because,

a) our salaries are met anyway

and

b) we have successfully scrounged excess computing time from various unspent computing accounts.

For effective use we believe that peripheral hardware costs to the extent of \$600. p.a., for a class of 20, are necessary. In addition we expect computing costs to total \$1,000. p.a.* At a cost of \$1.00 per connect hour and at 30 c.p.s., we expect this cost to fall to under \$50. p.a.

On the other hand the proprotionate cost of a suitable hydraulics laboratory will be commensurate (\$10,000) (including floor space and capital equipment) but would incur the proportionate salaries of laboratory technicians. Together with running and maintenance costs this could also be commensurate (say \$1,000 p.a. per course) but hard figures are difficult to derive.

We did not include travel costs in these figures, since these are negligible, and in any case are met by the students themselves. On the other hand we have

* based on \$14.50 per connect hour, and 10 characters per second.

not accounted for savings in demonstrators and tutorial-assistants (MACCOM auditing can be accomplished automatically) because we believe that instructional assistants are valuable in both methods.

We may add that these estimates are based on 10 connect hours per student group of four students, because we believe that group interaction is the most successful learning mode at this level.

An important point is that simulation costs do not rise discontinuously with increase in class size. Furthermore improvements in hardware, software and courseware indicate a decline in costs in the near future.

All in all, since other users do not necessarily face the same development costs, the simulate-and-site-visit approach appears to be competitive, especially if there is a shortage of suitable laboratory space, and suitable hardware is made available.

Some Guiding Principles

In summary, we believe that our approach holds hope for on-campus computing, and educational budgets, as well as meeting some of the shortcomings of traditional teaching methods in the environmental sciences, assuming appropriate policy shifts can be accomplished. We now list our own design requirements, for the record

1. Student must be able to control:
   (a) time and place for exercise;
   (b) scope and pace of exercise;
   (c) input variables, for design mode;
   (d) cessation of exercise.

2. "Computer" must not:
   (a) bomb out unnecessarily;
   (b) respond slower than (say) 0.5 sec.;
   (c) be vague;
   (d) accept poor input;
   (e) talk FORTRAN;
   (f) merely page-turn.

3. "Computer" must:
   (a) explain all actions;
   (b) quantify correctly;
   (c) reflect the instructor's mannerisms;
   (d) use valid models related to field trips;
   (e) decouple exercise from grades if so required;
   (f) improve the learning environment;
   (g) facilitate communication bbetween students and the professor.

Our experience has shown that all these software requirements are readily attainable but hardware provisioning depends upon considerable reshaping of computing policy. Still, though we have a long way yet to go, we feel too optimistic to turn back, or even to slow down.

PART 4: INSTRUMENTATION AND CONTROL
(7 papers)

Apart from the exceptions listed below, I wrote and produced all the papers. Except for Kitai, the co-authors were graduate students, working for Ph.D. or M.Eng. degrees, under my supervision. In some other cases research students carried out the experiments. In all cases they had no previous experience in these fields; and their program of work, the original line of research and the theoretical and experimental procedures to be followed, were laid down by me.

EXCEPTIONS:

Paper IC7: The rain sensor was developed under my supervision and the general ideas and performance specifications for the data logger and decoder were due to me. The circuitry for the latter were entirely due to Haro. The overall data capture system was my own conception. (A similar situation occurs in IC2 where the circuitry was due to external consultants). Paper IC7 bears equal responsibility between Haro and the writer.

Paper IC4: This paper was written by Pegram and benefits from his experience while a Ph.D. student at the University of Lancaster. The multivariate analysis originated in his M.Eng. studies under my direction, but the credit belongs primarily to Pegram. Inclusion here is appropriate since the transfer function models in the microcomputer controllers, (the current thrust of the writer's research), are conceptually similar so Paper IC4 nicely closes the circle.

Precipitation Instrumentation Package for Improved Spatial and Temporal Sampling of Rainfall

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***Abstract*—A precipitation instrumentation package for sensing rainfall intensity with fine time and space resolution is described. The package comprises a drop-counter precipitation sensor, a microcomputer-based data-acquisition system, and an intelligent data decoder. A comparison is made between the precipitation sensor and the conventional tipping bucket rain gauge, and the merits of the new system are discussed. Typical data are included.**

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I. Introduction

IN THE RECENT PAST, it has been customary for hydrometeorologists to design their field networks to minimize system cost (capital and maintenance). Standards for instrumentation have typically been established by meteorologists at the national level. Costs of individual instruments in present use are significant, being of the order of $1000 for a complete station. Running costs are high, because conventional data-collection systems require a high labor content. Thus a network of only a few rain gauges has required about one full-time technician salary equivalent.

The result of these real-cost components of running a rain-gauge network has been that the data collected tends to be insufficient. Typically civil engineers, in turn, use methodology

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for hydrology that is based on simplified assumptions, such as uniform spatial distribution of rain over significantly large catchment areas. Dynamic tracking of storms, storm models based on kinematics of cells within the rainstorm, and similar improvements, have had to await the advent of better spatial and temporal sampling of rainfall. We have shown [1] that these observations and models of storms lead to considerable improvements in estimates of runoff and pollutant washoff, a matter of great importance in municipal engineering and flood management.

Most rainfall-intensity monitoring is, at present, based on tipping-bucket rain gauges (TBRG) [2], [3]. A conventional TBRG is large, cumbersome, and expensive. The associated chart recorders frequently give rise to errors, particularly when the recorder strip charts are translated manually into time series to be analysed. Accordingly, we have attempted to produce an automated, low-cost, high-resolution, reliable, and intelligent system for rainfall data measurement, acquisition, and presentation.

The precipitation instrumentation package is specifically designed to improve the understanding of rain, its occurrence, and its effects on the environment. The package consists basically of three major parts: a precipitation sensor, a microcomputer-based data-acquisition system (DAS), and an intelligent data decoder.

The precipitation sensor collects the rainfall and converts it into water drops of almost constant size [4], [5]. The drops are detected when they close an electric circuit. The data-acquisition system senses the drops and counts them over a programmable time interval, storing the counts temporarily in a single-chip microcomputer. The acquisition system processes the time series and stores it on standard audio cassette magnetic tape. The cassettes are removed and transported to the central site to be read by the data decoder. The time interval between cassette removals can be many months. The data decoder reads the information from the tape, verifies, and communicates the rainfall time series to a computer for data processing.

II. Precipitation Sensor

The precipitation sensor is contained in two mass-produced P.V.C. cylinders, denoted *1* and *2* in Fig. 1. These are standard plumbing fittings, used in household waste-water drains. Cylinder *1* contains the sensor, while cylinder *2* is used as the base, housing the instrument. Both cylinders are easily assembled; screws *3* are used to clamp them together.

Inside cylinder *1* there is a plastic funnel *4* whose function is to collect the rain. The rubber stopper *5* is pierced by a capillary glass tube *6* that converts the water into drops. The lower part of the funnel *7* supports the sensor *8* which consists of two electrically conducting points. The points are two gold-plated mass-produced pins.

The sensor points are connected to an amplifier *9* which boosts the electrical signal produced when the drop is in contact with the sensor points. Connector *10* is mounted inside cylinder *1* and provides access to the electrical signal from the amplifier. Cylinder *1* may be separated mechanically as well as electrically from cylinder *2*.

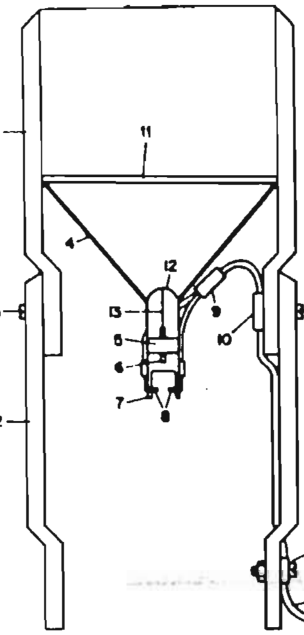

Fig. 1. Section through drop-counter precipitation sensor (DC-PS) center. Water drops are detected when they close an electric circuit via test points *8*.

At the top of the funnel, inside cylinder *1*, there is a removable coarse screen *11* whose function is to trap leaves, etc. Under this screen, at the point where the funnel shape changes, there is another dome-shaped fine screen *12* to prevent dust from entering the glass tube.

From screen *12* there is a fine wire *13* that passes through the glass tube. This is used to reduce the surface tension to be overcome by the water in order to pass through the tube.

A clamp *14* secures the two-conductor cable *15* to cylinder *2*. The cable feeds the electrical signal to the data acquisition system.

Limitations of the sensor are as follows:

For very high rainfall intensities, the flow changes from drops to a continuous jet. Tests show that, for the geometry presented here, continuous jet flow commences only well above 150 mm of rain/h, which is a very rare event. In Ontario, such a rain intensity would only last 10 min or longer, about once every 100 years on the average. Of course, a smaller catching area would handle higher rainfall intensities.

The two meshes have to be partially wetted before the water can pass through the funnel.

When the sensor is operated from the dry state, some water must be collected before water will pass through the glass tube. Also, once the sensor is wet, some water must build up at the top of the tube to overcome the surface tension and then pass through the tube. The amount of water required from the dry state is approximately 0.05 mm of rain, and the amount required to overcome the surface tension is approximately 0.03 mm of rain. These amounts are considered to be negligible in most measurements. Technical details of the precipitation sensor are listed in Table I.

III. Data-Acquisition System

The DAS was originally designed to collect and store information from the precipitation sensor, and therefore it contains only one channel at this stage. It is intended that other

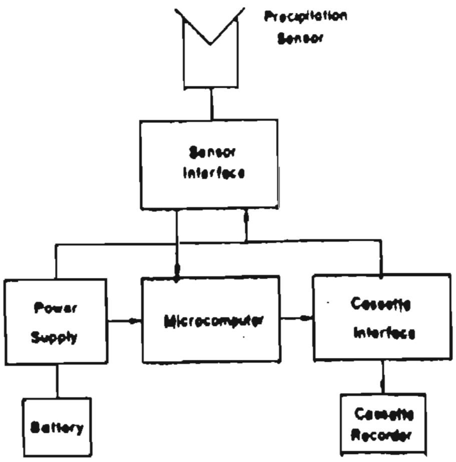

Fig. 2. Block diagram of the Data Acquisition System (DAS).

TABLE I
DROP-COUNTER PRECIPITATION SENSOR

Collection area	[illegible]107 sq. mm.
Drop size:	
Mean	0.00390 mm. of rain.
Standard deviation	0.00015 mm. of rain.
Maximum rate	150 mm. of rain/hr.
Water retention from dry state:	
Mean	0.020 mm. of rain.
Dimensions:	
Height	365 mm.
Diameter	115 mm.

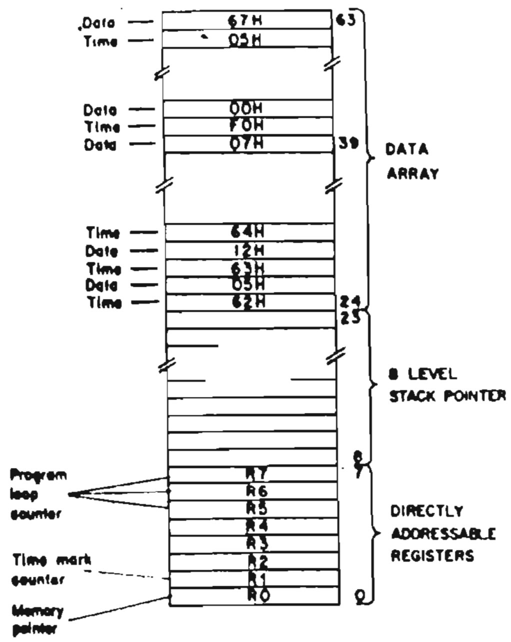

Fig. 3. Data memory map of the 8748 microcomputer. (H refers to an 8-bit number in hexadecimal code.)

measurements, such as temperature, drop conductivity, and the like will be included in later developments. A block diagram of the DAS is shown in Fig. 2. Its operation is as follows.

The microcomputer counts the number of drops during a time period known as the rain integration time. At the end of this time, the count is stored in the microcomputer RAM together with a time mark. Each data word occupies one byte (8 bits) so that the largest number that can be stored in one memory location is 255. However the precipitation sensor is capable of detecting over 10 drops/s which is more than 600 drops for a rain integration time of 1 min. To cope with this number, we would require a word length of 10 bits, so an alternative was chosen: we divide the total number of drops by two to reduce the range. Events of over 120 mm of rain/h are very unlikely in the area of Ontario where the systems are operating at present. The instrument resolution with division is 0.0078 mm of rain per count. This is satisfactory for the present data processing which is designed to take data from a TBRG where the resolution is lower. The microcomputer performs the division by 2 using software; it may therefore be modified as required with simple program changes.

The 8748 microcomputer has 64 bytes of RAM. Of these, 40 are available to store up to 20 pairs of data (data sets) and the remaining space is used for the program execution. Fig. 3 shows the data memory map. The first eight locations are directly addressable by several instructions, and some of them are used for the time mark counter, the memory pointer, and the loop counter to determine the integration time. The next 16 locations contain the program counter stack for up to eight nested subroutines. The data collected and the time marks reside in the last 40 memory locations.

Every time this memory space is filled, the microcomputer turns on the cassette recorder and dumps the block of information onto magnetic tape. In order to save space in memory and on tape, the acquisition system represents the acquisition time as an incrementally coded time mark. The microcomputer stores data in RAM memory only when rainfall is detected or when the time mark counter equals 240 decimal. The integration time is software generated and is therefore easily modified to meet unusual requirements. In the present systems it is 1 min.

The cassette interface contains the circuits needed to control the recorder power supply and to condition the signal to be recorded. The recorder is a mass-produced unit for audio purposes that has been slightly modified. In order to record the digital signal onto the tape, frequency shift keying (FSK) is used because of the limited frequency response of the recorder. Standard audio cassette tapes are used.

The microcomputer generates the FSK signal using software, and delivers it through one of its output pins to be recorded onto tape. Every character recorded comprises eight bits of information, one start bit, one stop bit, and one parity bit. The control bits are used in the data-recovery stage. The Kansas City standard [6] for recording digital information on audio magnetic tape is used.

Fig. 4 shows a circuit diagram of the DAS. Transistor $Q2$, placed inside the precipitation sensor, amplifies the current signal generated when a drop closes the electric circuit at the test points. In the DAS, the current signal is converted into a voltage by resistor $R1$. Capacitor $C1$ reduces electrical interference. Vr is a voltage reference of approximately 2 V derived by means of $R7$ and $R8$. Voltage comparator $A1$ transforms the voltage signal into square pulses. $R2$, $R3$, $C2$, and $D1$ form

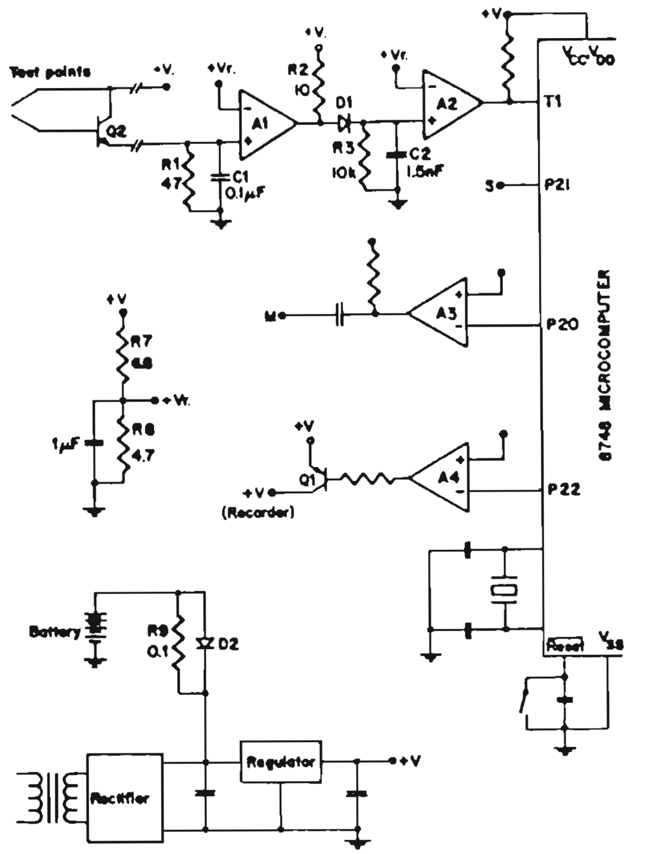

Fig. 4. Circuit diagram of the DAS. Test points and transistor $Q2$ are placed at the precipitation sensor. Resistor values are in kilohms.

a retriggerable one-shot that produces a pulse of approximately 10-ms duration. The output of $A2$ is fed into the microcomputer where the pulses are counted.

The modulated signal M is sent to the cassette recorder via output port 2, bit 0 ($P20$), and through voltage comparator $A3$ where it is boosted. Capacitor $C3$ is a high-pass filter to remove dc and low-frequency components from the signal. The microcomputer turns the cassette recorder power supply on and off via $P22$. Voltage comparator $A4$ amplifies the output signal from the microcomputer to drive the transistor $Q1$ as an ON/OFF switch. The collector of $Q1$ is connected to the cassette recorder power supply line.

The power supply includes a rectifier, filters, and a voltage regulator; it also contains a self-charging battery backup. When the primary power source is on, the battery charges through $R9$, $D2$ being reverse polarized. In the event of primary power source failure, the battery provides the required current through $D2$.

The circuit is not intended for battery-only operation because its power consumption would not allow this; however, the microcomputer can be replaced by its recently introduced CMOS version, if desired.

The electronic components are assembled on a 5 by 7.5 cm single-side printed circuit board. The circuit board, the transformer, the battery, and the cassette recorder (that has been removed from its plastic case) are housed together in a 20-cm (width), 9-cm (height), and 15-cm (depth) metal case.

The unattended time period of the system depends basically

TABLE II
DATA ACQUISITION SYSTEM

Microcomputer	Intel 8748
Storage media:	
Magnetic cassette tape	Audio.
Modulation	Frequency Shift Keying (FSK).
Carriers	1.8 kHz. for logical "0", 2.7 kHz. for logical "1".
Baud rate	300 bits per second.
Format	50 bytes per block, 11 bits per byte.
Data block	10 bytes of sychronism, 20 bytes of data, 20 bytes of timing.
Measured error rate	Less than one error in 10E6 bits stored.
Integration time	Programmable.
Period of unattended operation	7 days of rain. (tape C-60 one side).
Operating temperature	+10 to +50 C.
Power requirements:	
Steady state	75 mA. @ 5 V. D. C.
Peak	225 mA. @ 5 V. D. C. for 3 1/2 seconds when data is transferred to tape.
Battery back-up	17 hrs. Six "C" size rechargable.
Dimensions:	
Height	9 cm.
Width	20 cm.
Depth	15 cm.

on the tape capacity. The DAS stores approximately 17 blocks of information on 1 min of tape. The number of data sets recorded depends on the duration of rain and not on the amount of rain. When the integration time is programmed to 1 min, the system can be left unattended for more than 7 days of continuous rain or more than 4 years of no rain.

The microcomputer is fully dedicated to data acquisition and storage. However these tasks use only a small fraction of time, so the remaining time may be used to perform some data processing.

The program in the microcomputer is organized into four major subroutines: one for data acquisition, one for the integration time delay, one for the parity bit generation, and one for the FSK signal modulation. The complete program uses just over 270 memory locations, leaving space for data processing programs. Details of the DAS are given in Table II.

IV. DATA DECODER

A microcomputer-based data decoder retrieves the information from the tape and transfers it to the central computer. A block diagram is shown in Fig. 5.

The played-back signal is a distorted sinusoidal FSK signal. A second-order bandpass filter at the input of the decoder attenuates noise outside the band of interest. The filter also

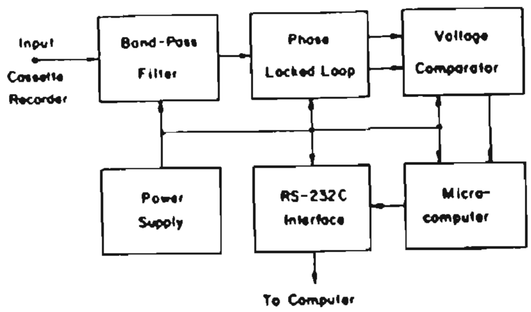

Fig. 5. Block diagram of the Data Decoder (DD). The microcomputer receives and verifies the recorded information and transmits it to the central computer for further processing.

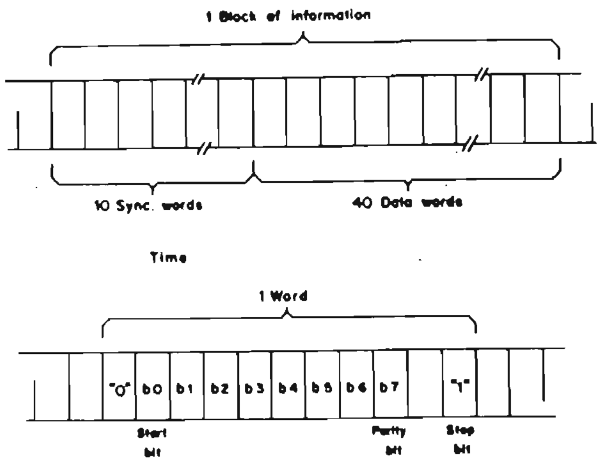

Fig. 6. Timing diagram of tape information and word structure.

amplifies both signals, but the high-frequency carrier is amplified slightly more than the low-frequency carrier, to equalize the recorder frequency response.

After the input signal has been filtered and amplified, a phase locked loop together with a voltage comparator perform the signal demodulation. The digital signal at the output of the comparator is fed into the microcomputer through a testable input pin using conditional jump instructions.

The microcomputer receives the digital signal and converts it into parallel format, checks the validity of the data using the parity bit, and informs the user of any error detected. The information retrieved is converted from straight binary into ASCII and transmitted asynchronously to the central computer for data processing. Data transmission is at 1200 Bd via the RS-232C interface.

Every block of information recorded on tape is preceded by a synchronization block, consisting of at least ten characters of zeros. Fig. 6 shows a time diagram of the information recorded on tape. To perform the data recovery, the microcomputer synchronizes with the start bit of each character and generates the required time delay to sample the rest of the bits at the center of occurrence. The presence of the stop bit is checked for error detection. This type of synchronization ensures that small amounts of pulse-to-pulse jitter and frequency shifts have no effect on the data recovery. Tests performed show an error rate of one soft error in more than one million and one hard error in more than ten million bits stored.

When played back, the tape contains blocks of noise between the information blocks. These blocks are introduced onto the tape when the microcomputer turns the cassette recorder on and off. The tape-use efficiency is reduced to just over 55 percent because of these blocks. The efficiency may be improved if circuitry is incorporated to achieve faster start and stop of the recorder drive mechanism.

Tape installed October 5, 18:30 hrs.

098005099018100042101046102040
103021104007105007240000240000
236017237063238098239140240153
001165002149003141004128005103

FIRST NUMBER	SECOND NUMBER	TIME	DAY	DROPS	mm. OF RAIN
098	005	20:08	5	10	0.039
099	018	20:09	5	36	0.1404
100	042	20:10	5	84	0.3276
101	046	20:11	5	92	0.3588
102	040	20:12	5	80	0.312
103	021	20:13	5	42	0.1638
104	007	20:14	5	14	0.0546
105	007	20:15	5	14	0.0546
240	000	22:30	5	0	0.0
240	000	2:30	6	0	0.0
236	017	6:26	6	34	0.1326
237	063	6:27	6	126	0.4914
238	098	6:28	6	196	0.7644
239	140	6:29	6	280	1.092
240	153	6:30	6	306	1.1934
001	165	6:31	6	330	1.287
002	149	6:32	6	298	1.1622
003	141	6:33	6	282	1.0998
004	128	6:34	6	256	0.9984
005	103	6:35	6	206	0.8034

Fig. 7. One complete block of information and its interpretation. Starting time: October 5, 18:30 hours.

V. Data Processing

After the block of information has been converted into ASCII format by the microcomputer, each of the 20 data sets comprises 2 numbers of 3 decimal digits each. The first number represents the acquisition time and the second the amount of rain sensed in the previous integration time. To convert the series of numbers into meaningful information, one needs to know the time and date when the tape was installed. Fig. 7 shows one complete block of information (40 words) and its interpretation. High rainfall is seen at the start and end of this block. The intensity increases then it tails off and rain ceases for 10 h 11 min before another onset.

IC7

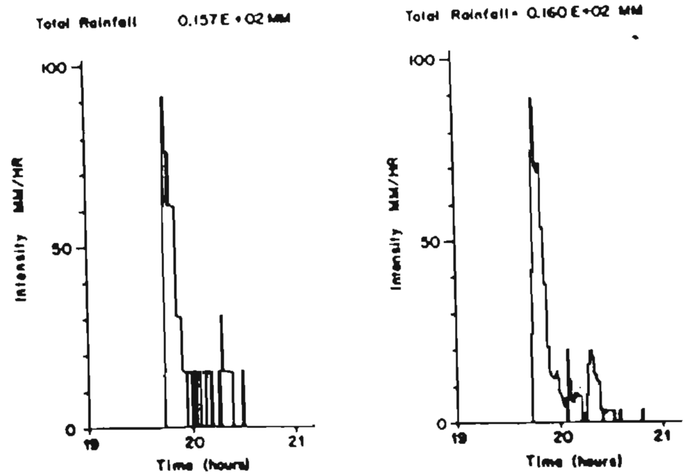

Fig. 8. Typical hyetographs of the same thunderstorm obtained with both measurements. Tipping bucket (left) and drop-counter precipitation sensor (right).

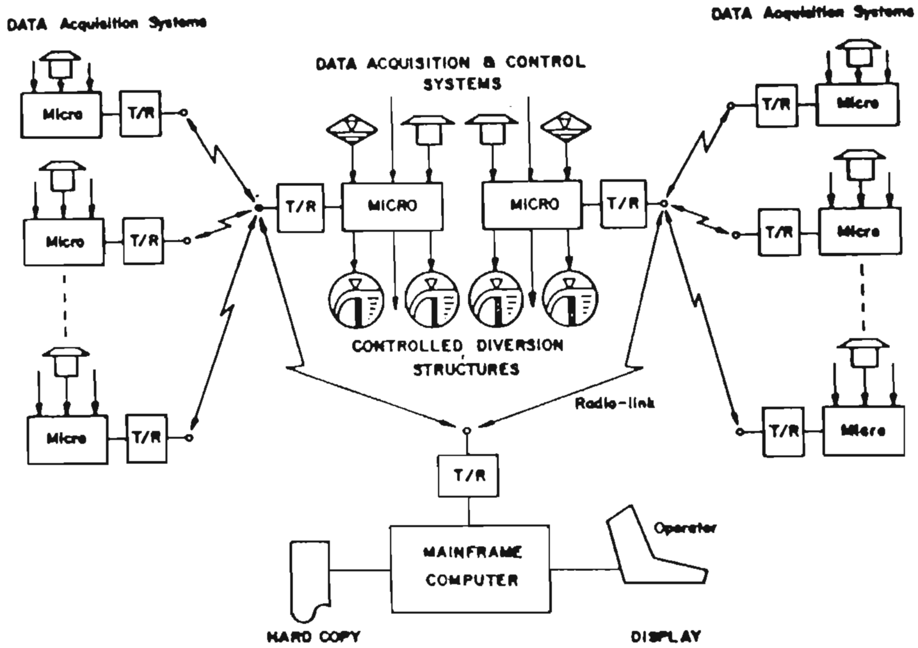

Fig. 9. Envisaged real-time microcomputer-control system.

Once the information has been received and interpreted by the computer, it is ready to be processed. At present, two different types of processing are used. The first is the derivation of a hyetograph [7] for each one of the storms recorded for each one of the monitoring sites. The second type of processing is done using computer programs for stormwater management modeling. These models are used in turn to determine average, daily, monthly, and annual amounts of stormwater runoff entering the receiving waters [8], [9]. The models are also being used to investigate a wide range of design alternatives and strategies for minimizing pollutant overflows due to stormwater from the city of Hamilton. In the field program, rainfall intensity, stormwater quantity, and quality samples are collected from various field stations located throughout the city.

Substantial improvements in stormwater management are achieved when more and better information is available. The need is to increase the number and quality of sampling sites without increasing capital and maintenance costs appreciably, by using low-cost systems that require little maintenance and data processing, and which provide the user with information of higher resolution. The component costs for the instruments described are approximately \$15 for the precipitation sensor, \$100 for the DAS, and \$70 for the data decorder. For comparison, a tipping bucket and chart recorder installation cost approximately \$1000.

At present, ten of the drop-counter precipitation sensors are in use in the field in Hamilton, as part of the hydrometeorological field program of the Computational Hydraulics Group at McMaster University. Also nine sensors are being used in different precipitation sampling sites across Ontario while others are to be used in Ottawa, Halifax, and the Northwest territories. Some of the sensors used in the Hamilton area operate at the same sampling sites as the tipping-bucket rain gauges in order to compare the performances of these instruments. Standard rainfall totalizers are also located at each station to provide a comparable total volume of rainfall for individual storm events. The data from the TBRG were collected and stored with a modified version of the DAS. Fig. 8 shows typical hyetographs of the same thunderstorm obtained with both instruments. The shapes are seen to be similar, and at high rainfall rates, the measured intensities are nearly the same. For all rainfall rates, the drop-counter precipitation sensor exhibits superior resolution. Rainfall starting times just about coincide in this example, where the initial rainfall rate is high.

The system is to be extended in various directions, including the addition of a radio communication link between the sampling sites and the central computer for real-time operation and the extension of acquisition channels. The new channels will be used to collect more information, using additional transducers for measurement of water depth in pipe discharges, water and ambient temperature, water conductivity and pH. It is envisaged that the real-time microcomputer control system would include the following, as depicted in Fig. 9:

1) remote monitoring and telemetering with microcomputer-based stations which include a cassette recorder in each station,
2) radio communication link,
3) microcomputer-controlled diversion structures for runoff control,
4) central minicomputer with display, operator control console and magnetic tape archive.

Conclusions

It has been shown that precipitation instrumentation can be constructed by using low-cost mass-produced components. The only components that needed to be fabricated are the drop sensor and the printed circuit boards for the electronic components. The low cost of the instrument and the lengthy period over which it may be left to operate unattended open the way to the deployment of many more rainfall gauges over a catchment area than has hitherto been economically possible. Improved spatial resolution permits new theoretical development on the spatial distribution of rainfall, and more realistic models of thunderstorm dynamics. This in turns allows total rainfall to be estimated much more accurately for stormwater control and for anti-pollution measures. A second and significant advantage is that failure of any one or two gauges does not affect the measurement system as a whole to any particularly significant extent.

Rainfall instrumentation systems often fall victim to vandals who destroy the comparatively expensive tipping buckets all too frequently, no matter how remotely and unobtrusively they may be situated. The small size of the drop-counter system, compared with the tipping tucket, also helps to make it inconspicuous.

Comparison measurements using the drop-counter gauge and the TBRG's show that the drop counter is more sensitive, leading to a superior signaling of the beginning and end of rain, and with comparable accuracy. Finally, the versatility of the microcomputer opens the way to a broader range of measures of atmospheric phenomena that share the same electronics and storage/communications.

Acknowledgment

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IC6

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Power spectral analysis of a forcemain failure caused by waterhammer

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Power spectral analysis of a forcemain failure caused by waterhammer

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The 14 in. (0.36 m) forcemain followed an overall convex pipeline profile for 4000 ft (1200 m) and would be subject to negative surge problems associated with pump shutdown. The forcemain couplings failed three times near the pumphouse and the failure was thought to be related to waterhammer effects. The sequence of breakages is reviewed. A series of pressure recordings were then made over 14 days on the repaired forcemain, leading up to and including the final failure. These pressure recordings were digitized and subjected to power spectral analysis. The power spectra pointed out several significant events that were not evident from the pressure record alone.

These included the fact that the original breakdown in the forcemain occurred several days prior to its ultimate failure and discovery on the surface. It was also determined that the break in the system was due to the apparent merging of the primary waterhammer wave with an existing but gradually changing lower frequency wave. This second wave was associated with rigid column motion and gradually increased its frequency. The resultant wave superposition collected sufficient energy at one point to cause the ultimate failure of the evidently already damaged forcemain system.

Power spectral analysis proved useful as a method for analyzing waterhammer effects in a forcemain complicated by column separation, leakage, and vapour pocket collapse, and may be a useful way of monitoring the performance of longer pipelines.

La conduite sous pression de 14 po (0.36 m) longe un modèle de pipeline complètement convexe pour une longueur de 4000 pieds (1200 m) et est sujette à des à-coups associés à l'arrêt de la pompe. Les joints de conduites sous pression ont fait défaut par trois fois près de la station de pompage, le problème étant lié à l'action des coups de bélier. L'enchaînement des ruptures est examiné. La pression a été enregistrée à plusieurs reprises sur la conduite réparée pendant une période de 14 jours, menant à et y compris la rupture finale. Ces enregistrements de pression ont été chiffrés et analysés au moyen de spectre de puissance. Ce dernier a signalé plusieurs cas importants qui n'étaient pas évidents uniquement avec le relevé de pression.

L'analyse a démontré que la première perturbation de la conduite sous pression s'est produite plusieurs jours avant la dernière et sa découverte à la surface. On a constaté également que la panne du système semblait découler de la combinaison de la vague du premier coup de bélier avec la vague existante à basse fréquence bien que cette dernière changeait graduellement. Cette deuxième vague était associée au mouvement de la colonne rigide, augmentant peu à peu sa fréquence. La superposition des vagues a capté suffisamment d'énergie à un moment donné pour entraîner la dernière perturbation du système de conduite sous pression qui était évidemment déjà endommagé.

L'analyse spectrale a fait preuve de succès pour examiner l'action des coups de bélier dans une conduite sous pression, cette dernière étant compliquée par la séparation des colonnes d'eau, les fuites, l'éclatement de petites bulles, et pourrait être un moyen utile de surveiller le fonctionnement de plus longs pipelines.

[Traduit par la revue]

Can. J. Civ. Eng., 7, 36–44 (1980)

Introduction

In this paper power spectral analysis is applied to a series of pressure recordings in a forcemain to interpret the failure of the pipeline system. The pressure surges or waves in the forcemain were caused by the stoppage of a pump. The pressure recordings were taken over a period of 14 days and include failure of the pipe system; they provide a unique data set for study of the waterhammer problem.

The forcemain was constructed about 2 years ago to pump raw sewage from a wet well near the old Ancaster Road to the main gravity sewer, on

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Lowden Avenue, in the Township of Ancaster, Ontario. The line was originally constructed entirely of 14 in. (0.36 m) diameter class 100 asbestos cement pipe. It is 3963 ft (1208 m) long and rises a total of 133 ft (40.5 m).

There was some question at the time of construction as to whether proper pressure testing was carried out. The pump was first started in the fall of 1975. Shortly thereafter a leak occurred about 150 ft (46 m) downstream from the pump. The pipe was excavated, and the leak found to be the result of a broken coupling. The pipe was repaired, the trench refilled, and the pump restarted.

The line broke again at the next coupling down the line. These couplings were located adjacent to a stream. At this time it was believed that the breaks were the result of settlement of the pipeline resulting from the poor soil condition in this area.

After the third break about 150 lineal ft (46 m) of line in the area of the breaks was replaced with ductile iron pipe. The entire line was then pressure tested and found to meet required standards. It was then thought that the breakage problem was caused by waterhammer effects coupled with pipe settlement. It was likely that vapour pockets were forming in the line, and protective pressure vessels were installed in the forcemain system. These vessels were not in use during the next tests.

In early September pressure transducers were installed at two points. One transducer was placed in the line near the pump and the other at the location of the three breaks (denoted P and B respectively). The transducer was set up to record the pressure fluctuations that occurred for a few minutes after pump shutdown. In the period from September 9 to September 14, 1976 a total of 18 pressure wave recordings were made at this location. On September 14 the pipeline again failed, while the pressure recorder was operating. This provided a complete record of waterhammer pressure waves, during the deterioration of the pipe and including the failure: a unique data set for the study of waterhammer-related failure of such pipelines.

The latest break proved difficult to locate. It was eventually found 450 ft (137.2 m) from the pump and again proved to be a broken coupling. A camera was used to assist in locating the break. The camera showed deposits of gravel and dirt in the pipe to a depth of approximately 4 in. (10.2 cm) immediately downstream of the break.

Forcemain Details

The pipeline profile is shown in Fig. 1. The forcemain rises rapidly near the pumphouse. This convex profile, governed by the ground profile, could create a waterhammer problem associated with pump shutdown. Negative pressures could occur in most of the pipeline, if no protection were provided. The inertia of the rotating elements is small and the negative pressure would reach the vapour pressure of water; the water column would separate when large discharges are pumped.

Details of the line, which included one pump, are:

Length of forcemain (ft (m))	3963 (1208)
Diameter of forcemain (in. (m))	14.0 (0.36)
Pumping discharge (gal/min (L/min))	1000.0 (4546)
Pump head (ft (m))	157.5 (48)
Motor speed (rpm)	1750
Rotational inertia of pump and motor (estimated) (lb · ft² (kg · m²))	25.0 (10.32)
Specific speed (gal/min (L/min) units)	1254.0 (6518)
Pump efficiency	0.73

A discharge check valve is mounted at the pump in order to prevent flow reversal and drainage of the pipe into the wet well.

The steady-state hydraulic gradients for the above three operating conditions are also shown in Fig. 1.

Review of Waterhammer

Waterhammer is the name given to the phenomenon of pressure waves travelling back and forth in a pipe flowing full or partly full. These pressure waves are caused by a change of velocity in the pipe. Rapid velocity changes in a pipe are usually the result of closing or opening a valve, or of stopping or starting a pump. The resulting pressure waves travel up and down the pipe at the speed of sound in the elastic fluid and pipe.

Now, consider the pressure at a point in the pipe near the pump as the pressure wave moves up and down the pipe. The local cycle of pressure change appears as a square pressure wave, if we neglect the effect of the inertia of the rotating mass and also the finite closure time of the check valve. In practice, the amplitude of the wave is damped out because of fluid friction and imperfect elasticity in the fluid and pipe wall. Also, because of pump inertia, the actual curve appears to be sinusoidal. After a few cycles the fluid comes to rest under the static head or the normal operating head, as the case may be.

The period of the wave is a function of the pipe length since the recurrence of the wave is dependent upon its reflection at the end of the pipe. If the celerity of the pressure wave is a and the length of the pipe is L, then the period of the pressure wave is $4L/a$.

In fact the wave period is related directly to pipe

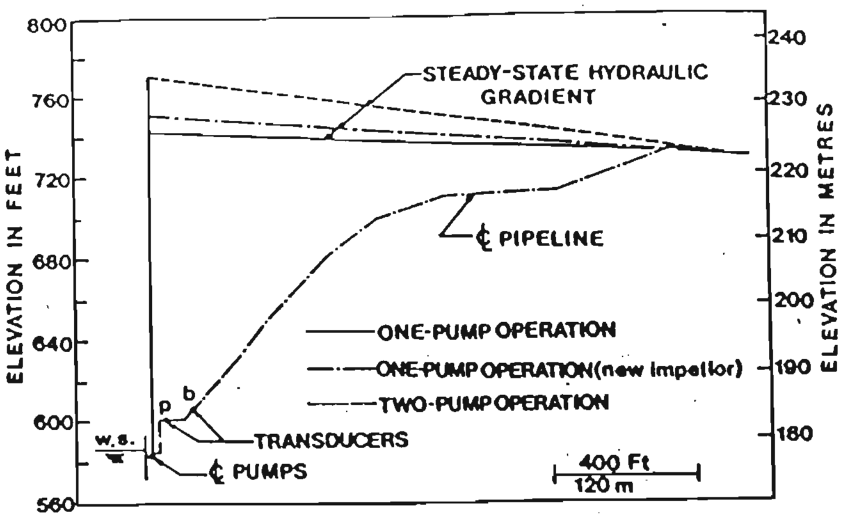

FIG. 1. Profile of Ancaster forcemain.

length, wall thickness, pipe elasticity, and the density and elasticity of the fluid (Streeter 1951; Parmakian 1955): period = $4L/a$ or in general

$$[1] \quad \text{period} = \frac{4L}{\left[\frac{\gamma}{g}\left(\frac{1}{K} + \frac{D}{Et}\right)\right]^{1/2}}$$

where γ is specific weight of fluid.

Note also that the wave frequency is the reciprocal of the period:

$$[2] \quad \text{frequency (Hz)} = 1/\text{period (s)}$$

On the other hand the amplitude of a wave is generally directly related to the momentum quantities: rate of stoppage, velocity, diameter, length, reflectivity, transmissivity, etc.

Review of Power Spectral Analysis

The power spectral density function is usually plotted as the variance of a process in terms of frequency. It can be compared to the auto-correlation function, which is the variance of a process with respect to time (Blackman and Tukey 1959; Box and Jenkins 1976).

The power spectral density function is the average power of a process expressed as a function of frequency. In wave mechanics the spectrum is plotted such that the average power of the wave process is the area under the single-sided power density spectrum between two specified frequencies.

There are several methods for calculating power spectra. One method uses the fast Fourier transform. Given a function of time, its Fourier transform is a function of frequency. This method is very suitable for calculating power spectra.

The equation solved by most computer programs in order to obtain the power spectrum is

$$[3] \quad S(\omega) = 4\int_{\tau=0}^{\infty} c(\tau)\cos\omega\tau \, d\tau$$

Note that $S(\omega)$ above is only an estimated value. The real value of $S(\omega)$ would require an infinitely long data set.

There are several library programs available to perform spectral analysis. The International Mathematical and Statistical Library subprogram FTFREQ produces a reasonably well-defined spectrum for the short records used in this paper and was consequently used for the analysis of the total record. The input required for the program consists of the discretized wave data set, the time interval between readings (0.20 s), and the number of desired lags. Also available are prewhitening and detrending routines that can be accessed with suitable input. These options were not used in this analysis. The output from the subprogram consists of the frequencies and associated power spectra for the input data set. The number of points computed in the power spectrum corresponds to the number of lags specified in the input. In this case 90 lags were specified, with approximately the first 30 being nonzero.

Application to Waterhammer

The power spectrum produced from a waterhammer wave is affected by the shape of the waveform. The pressure record for an idealized fluid in a pipe and for instantaneous stoppage is rectangular in shape. Where stoppage is affected by finite rotational momentum and valve closure times the shape tends to be saw-toothed. Viscous effects and other frictional losses cause the waves to damp out rapidly and to assume a sinusoidal shape. Spectral analysis of the dampened sinusoidal wave may thus produce a narrow spectrum, whereas the rectangular or saw-toothed patterns will produce harmonics in the spectra.

In this study the rotational momentum effects and valve closure times are of the order of 50% of the primary wave period (Kassem 1976). Inspection of the pressure records also indicates an absence of the rectangular and saw-toothed profiles. For these reasons, the deviation of the fundamental pressure waves from a sinusoidal shape has not been considered important; this aspect will be subjected to further study.

Application to the Forcemain

The profile of the forcemain studied is such that vapour pockets will occur in the line on pump stoppage. The formation of a vapour pocket in the line will cause the pressure wave to deviate from a simple idealized wave. When a waterhammer pressure wave meets a vapour pocket in a line, it is partially reflected back upstream as off an open end. The downstream column decelerates and returns as a rigid column as in the case of surge tanks and other similar devices. In the case of the forcemain under study, the downstream end of the pipe is open. Thus a volume of water approximately equal to the volume of the vapour pocket is lost during the first waterhammer cycle.

The expected frequencies of the waterhammer waves can thus be described as follows:

Firstly, a primary frequency can be expected to correspond to the theoretical waterhammer frequency. The observed frequency may be somewhat lower than the theoretical since the speed of the pressure wave is decreased in a partially formed vapour pocket. On the other hand it may be higher because the effective length of a partly empty and nearly horizontal pipe is reduced because of the horizontal water surface.

Secondly, a high frequency wave can be expected since a portion of the waterhammer pressure wave will reflect from the vapour pocket instead of travelling the entire pipe length.

Thirdly, a low frequency wave should be present to account for the rigid column motion beyond the vapour pocket.

The Data Set

The data set consists of graphical records of surge waves generated by 18 separate pump stoppages. It covers the period from September 9 to September 14, 1976. A typical recording is plotted in Fig. 2. The complete set of recordings may be published elsewhere (James and Disher 1979). The recordings were derived from the output of an XY plotter that was attached to two pressure transducers. The transducers were located at the pump (P) and in the pipe approximately 150 ft (46 m) downstream from the pump (B).

Note that the location of the transducer at a distance from the pump will produce a waterhammer pressure plot with narrower peaks than if the transducer were next to the pump. However, in this case the spacing of 150 ft (46 m) in relation to the total length of the pipeline is small enough that this effect can be neglected. The frequency response of the transducer and recorder is several thousand hertz and not a factor in the records.

The original plotting paper generally travelled at a speed of 5 mm/s while recording the pressure waves. It was only activated upon pump stoppage and continued until the waves were completely damped. The vertical scale on the plotter was set such that 1 mm represented 4.9 psi (33.784 kPa) of pressure within the pipe. The reading was calibrated against static head.

The records have now also been digitized by reading the pressure every 0.20 s (1 mm original horizontal scale).

The Results

Power spectra obtained using FTFREQ on each of the 18 original records are presented in Figs. 3–5. The following points may be noted:

(A) Initially, power spectra were produced for several pressure recordings to detect how frequencies generated by a particular pump stoppage changed during the duration of the recordings. The pressure records were analyzed in three half-record length portions: the first half, middle half, and last half. It was found that although the significant frequencies did not shift to any extent within specific records, the peak diminished from the start of the record to the end. That is, the first half of a record showed much higher power peaks than the last half.

This is to be expected since the power of the wave decreases during the life of the pressure waves because of imperfect elasticity and fluid friction. It was also found that the rate of attenuation is higher for higher frequencies than for the lower frequencies, which is also to be expected.

(B) Of greater interest is the change in the spectral estimates from one pump stoppage to another.

These spectra show that the significant frequencies at the start of the record are 0.25 and 0.03 Hz respectively. These correspond to wave periods of 4 and 33 s. The 4 s wave is readily visible on the original plots.

Calculations indicate that the primary waterhammer wave period is about 5 s. The observed period would be smaller because:

(1) The spectral estimate may not be "sharp"

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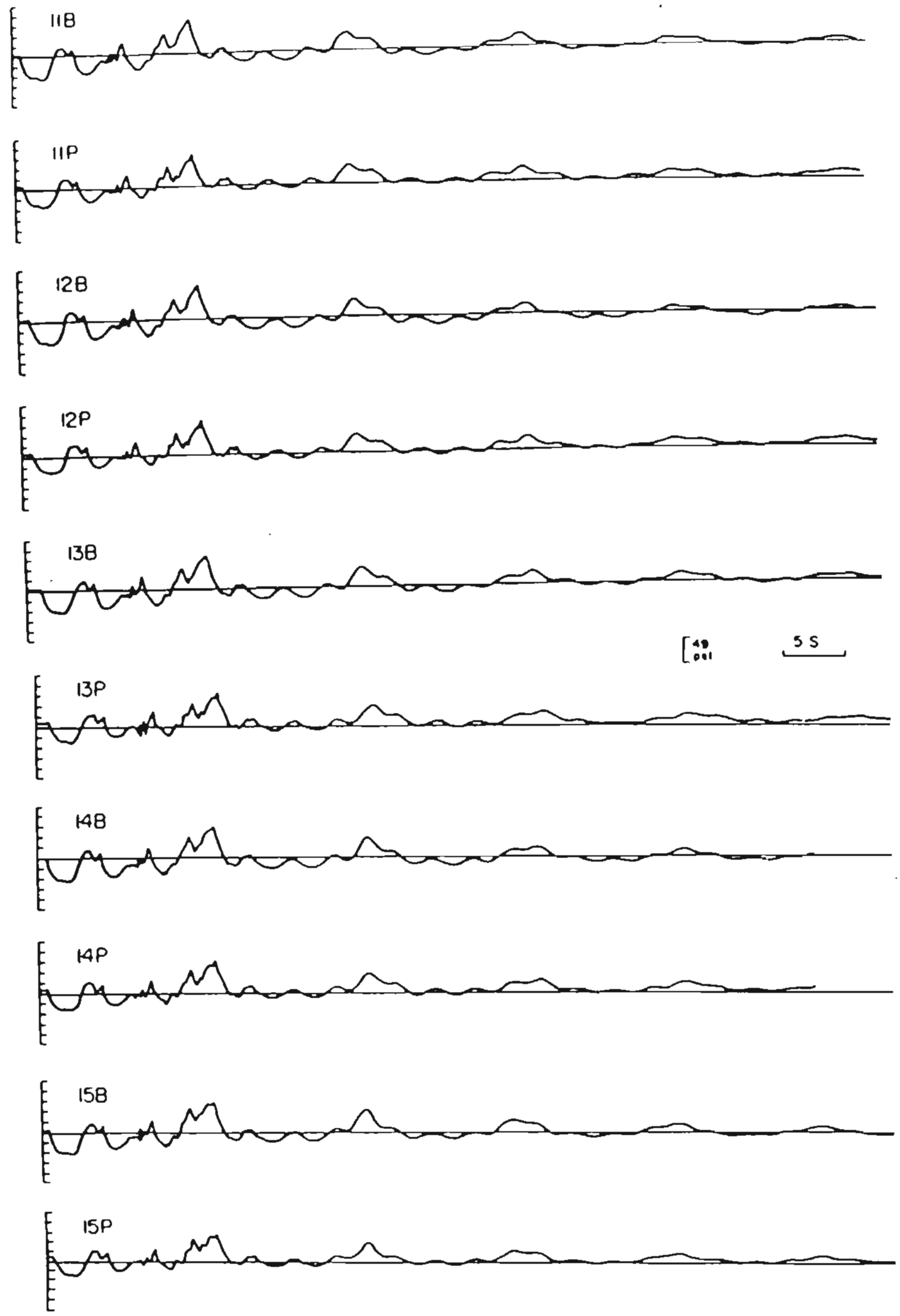

FIG. 2. Typical pressure record.

enough (i.e., the peak may actually correspond to a period in excess of 4 s).

(2) The fluid properties were assumed to be those of water whereas the actual fluid contained a large amount of impurities (sewage).

(3) The pipe characteristics may vary from those used in the calculations.

(4) (2) and (3) above could increase the celerity of the wave. The calculated celerity is between 3400 and 3500 ft/s (1036–1067 m/s). A faster celerity would give a shorter theoretical wave period (of the order of 4.4 s) and hence better agreement with the power spectral estimates.

A theoretical waterhammer wave period of 4.5 s and phase lag of 1 s would give a fairly close approximation to the actual pressure wave. The actual wave is not characterized by the rectangular waveform dictated by instantaneous stoppage. It is thus con-

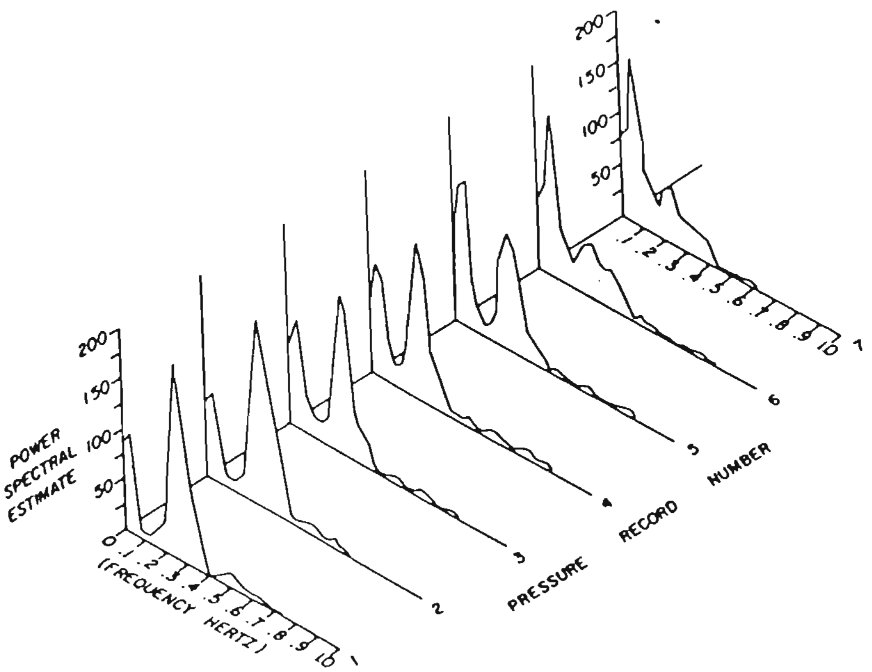

FIG. 3. Power spectra for the pressure records.

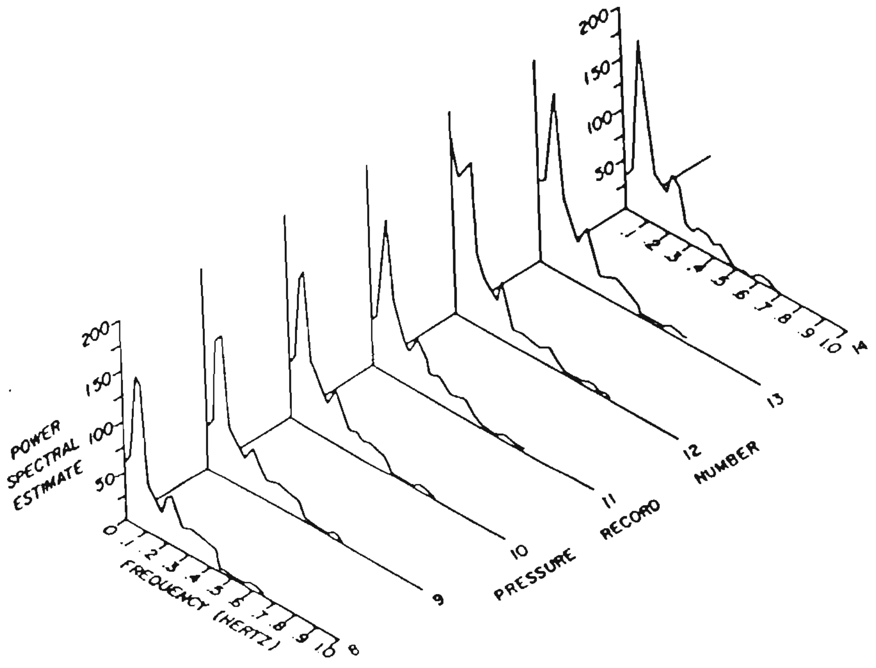

FIG. 4. Power spectra for the pressure records.

cluded that the peak in the spectral analysis occurring at approximately 0.25 Hz represents the primary waterhammer wave generated by pump stoppage.

(C) The expected high frequency peaks caused by the partial reflection of the waterhammer wave from the vapour pocket are not apparent in the power spectra. However, there are several peaks in the frequency band of 0.5–0.8 Hz. These peaks are relatively insignificant when compared with the lower frequency peaks, but tend to confirm that part of the waterhammer wave is reflected from a vapour pocket in the pipe.

(D) There is also a significant peak at 0.03 Hz in the power spectral estimates. This frequency corre-

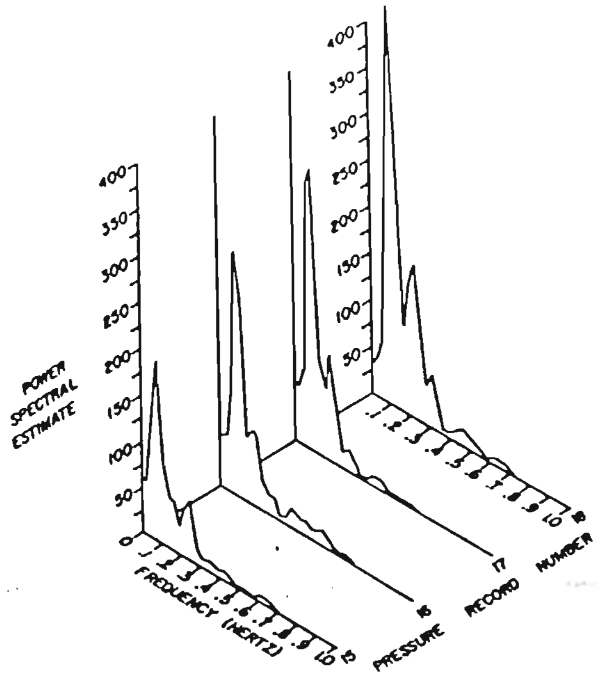

FIG. 5. Power spectra for the pressure records.

sponds to a wave period of approximately 33 s. This wave is not easily detected in the original pressure recordings. The wave period could be considerably in error since the spectral estimates are not very accurate at frequencies close to zero.

This wave corresponds to the expected low-frequency wave formed on the downstream side of a vapour pocket. It corresponds to the back and forth motion observed at the outfall long after pump stoppage.

Discussion on Failure of the Forcemain System

Upon study of the entire set of waterhammer recordings, it is evident that commencing at the sixth record, and progressing until the end of the set, there is a systematic growth of one distinct wave peak and an increasing pattern of longer period waves developing at the end of each record.

However, it is not until the 16th, 17th, and 18th records that it becomes evident that the original wave pattern has changed into a much longer period wave.

The first spectrum clearly indicates the primary waterhammer wave at 0.25 Hz and the low frequency peak at 0.3 Hz, respresenting rigid column motion.

The spectra remain relatively unchanged until the fifth spectrum where the rigid column peak becomes higher than the primary waterhammer peak. There is a considerable reduction in the magnitude of the primary waterhammer peak and a slight spreading of this peak into a higher frequency.

In the sixth spectrum the 0.25 Hz wave has completely degenerated and is spread over the frequency band of 0.2–0.4 Hz. It seems clear that a significant change in the pipeline occurred at this time. This spread of the spectrum into higher frequencies indicates that part of the wave is being reflected from a disturbance in the pipe.

The rigid column wave becomes the dominant feature in the power spectrum. The frequency of this wave increases from 0.03–0.06 Hz. It seems clear that the disturbance in the pipe is also affecting this wave.

The power spectra of records 7–13 remain similar to spectrum 6.

In spectrum 14 there is a significant increase in the peak of the rigid column wave. This peak has also shifted from 0.06–0.09 Hz. Thus the period of the wave decreases. There is also a renewed growth of the peak of the 0.25 Hz primary wave.

In spectra 15–18 the trend continues at an extremely fast rate. The peak of the rigid column wave on spectrum 18 has shifted to 0.11 Hz (9 s period) and its magnitude is 2.5 times greater than that of spectrum 14.

On the next cycle, No. 19, there was a noticeable breakdown and the system collapsed.

In summary, the spectral analysis shows that during the total period of the pressure records the following significant events occurred:

(1) The peak of the 0.25 Hz primary wave degenerated early in the record and was shifted into a higher frequency band.

(2) The 0.03 Hz (33 s period) rigid column wave became the dominant wave in the power spectrum.

(3) The 0.25 Hz primary wave peak reformed and grew in magnitude during the latter part of the record.

(4) The peak of the rigid column wave shifted from 0.03 to 0.11 Hz during the period of record.

(5) The peaks of the 0.3 Hz rigid column wave and the 0.25 Hz primary wave superimposed and grew in magnitude at an extremely fast rate during the latter part of the records.

An attempt will now be made to explain these observations in terms of probable fluid mechanics in the forcemain. The collapse of the 0.25 Hz peak and its spread into the higher frequencies indicates that there has been a significant disturbance occurring in the pipe. The waterhammer wave is no longer travelling the entire pipe length but is being interrupted at an intermediate point. The waves are being partially reflected at this discontinuity and hence the shift in the spectrum to higher frequencies occurs.

The phenomenon was detected in the sixth spectrum, although the system did not actually break down until the 19th record. These observations are both consistent with the effects of a faulty coupling (the pipeline had a history of coupling failures; furthermore, when the breakage was eventually repaired, evidence pointed to a faulty coupling as the source of the problem).

The growth and frequency shift of the low frequency wave are also consistent with the gradual increase in the magnitude of a leak or breakage at the coupling.

Any fault in the coupling would let in a volume of air during the negative surges. This air would act in the same way as a vapour pocket. With each pump stoppage the magnitude of the break would likely increase and the air volume injected would increase. Thus, increased amounts of water would be ejected from the downstream end of the pipe. This has the effect of shortening the effective length of the forcemain. A decrease in effective length of the rigid column component will decrease the period of the surge. This is also consistent with the power spectra as the low-frequency wave period decreases from 33 to 9 s.

This explanation is further reinforced by the fact that whereas the frequency of the reflected primary pressure wave (high-frequency wave) depends only on the location of the fault, the frequency of the low-frequency wave depends on both the location and the air volume injected at the break. A larger fault will cause more water to spill from the end of the pipe, thus decreasing the effective length of the pipe and the period of this wave.

The wave transmission is further complicated by the presence of solid matter in the pipe since the pumping velocity of the forcemain is not sufficient to scour this material from the pipeline. The surges will be partially transmitted and reflected at these obstacles thereby introducing more high-frequency waves.

The power spectrum indicates that the peaks of the original 0.03 Hz rigid column wave and the 0.25 Hz primary wave have merged into one 0.11 Hz wave at the time of breakage. This superposition of peaks provided a sufficient energy peak to finally fracture the coupling.

Conclusions

(1) At the start of the waterhammer pressure wave record, when the coupling is probably only slightly defective, the primary wave is the dominant feature in the wave spectrum.

(2) There is also a peak in the spectrum associated with a low-frequency rigid column wave, probably arising from vapour pocket formation or small air pockets. This wave is not distinguishable in the early records, but shows up clearly in the power spectrum. This wave has an initial period of approximately eight times that of the waterhammer wave. This wave proves to be important, contributing to the failure of the forcemain.

(3) There is a significant deterioration of the primary waterhammer wave on the sixth record (corresponding to the sixth pump stoppage). At this time the primary waterhammer wave suddenly reduced and the wave energy spread over a greater frequency band. There is also an increase in both the peak and the frequency of the rigid column wave. This wave becomes the dominant feature in the spectrum at this stage.

The explanation offered is that a crack or small break has occurred in the line; for example, a rubber gasket in a coupling may have moved. This break is not large enough to show up on the ground surface, but it does have an effect on the primary waterhammer wave. The waves show an upward frequency shift. This is probably the result of the introduction of air into the forcemain at the crack, and reflection of part of the wave.

(4) The rigid column wave showed a sharp frequency shift in the last two records before the ultimate failure of the system: the wave period decreased to approximately 9 s from the initial 33 s. The peak of the wave in the power spectrum increased to a value 4.5 times that of the initial record.

(5) The area under the power spectrum in any frequency band represents the proportionate amount of power available in that band. Evidently a large amount of energy built up in a narrow frequency band centred on 0.11 Hz (second period). This energy resulted in the ultimate failure of the forcemain.

(6) Once a small failure occurs in a forcemain subject to column separation, there is a strong possibility that a crack will inject air into the vapour pocket. With continued use, the incipient condition will deteriorate letting in more air. This alters the tuning of the low-frequency wave. The probability is high that the residual primary wave will superpose with the upwardly tuning low-frequency wave to produce high pressures of short duration. Such pressures could result in complete collapse of the rapidly deteriorating pipeline at a critical point.

(7) On-line power spectral analysis offers an interesting opportunity to monitor important pumping pipelines and to provide advance warning of potential pipeline failure.

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Notation

a = celerity of pressure wave
$c(\tau)$ = autocovariance function
D = inside diameter of pipe
E = modulus of elasticity of pipe
g = acceleration due to gravity
K = bulk modulus of fluid
L = length of pipe
$S(\omega)$ = single-sided power spectral density function
t = time, thickness of pipe wall
γ = specific weight of fluid
ω = angular frequency
τ = time lag

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Proceedings of the American Society of Civil Engineers

APPROXIMATELY LINEAR LOW-PASS WAVE FILTERS

By William James,[1] A. M. ASCE

INTRODUCTION

Spectral wave generators are being used increasingly by experimental researchers in coastal engineering, particularly for the class of wave problems generally investigated in unidirectional wave channels. For both random and regular spectra it is usually desirable to decouple effectively the fluid container from the geometrical configuration being tested (2). This is especially desirable for the frequency bands containing energy peaks, as amplitude growth at undamped frequencies may be troublesome.

Two devices popularly used for decoupling are absorbers and low-pass filters. Absorbers typically are placed at reflecting boundaries, while filters usually are placed between the paddle and the upstream recording station, and often also between the downstream recording station and the absorbers.

Accordingly, it has become important to predict the spectra transmitted and reflected into the experimental or recording area from that set up and generated at the source and those transmitted through and reflected from the decoupling devices. To compute transmitted and reflected spectra it is necessary to develop both a transmission and a reflection transfer function for the filters, as well as a reflection function for the absorbers.

Clearly for ease of computation the decoupling devices should preferably

Note.—Discussion open until July 1, 1973. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Waterways, Harbors and Coastal Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 99, No. WW1, February, 1973. Manuscript was submitted for review for possible publication on May 17, 1972.

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TABLE 1.—SUMMARY OF RESULTS FOR VARIOUS AMPLITUDES

T, in seconds (1)	α_1 (2)	α_2 or α_b (3)	β (4)	$\alpha_{f(max)}$, as a percentage (5)
0.712	0.042	0.073	0.437	2.8
	0.031	0.074	0.475	
	0.034	0.074	0.474	
0.750	0.040	0.091	0.451	0.8
	0.027	0.081	0.483	
	0.03	0.072	0.483	
	0.021	0.058	0.477	
	0.02	0.074	0.498	
	0.018	0.077	0.483	
	0.02	0.086	0.484	
	0.023	0.090	0.472	
	0.034	0.084	0.461	
0.802	0.043	0.08	0.555	1.8
	0.04	0.083	0.581	
	0.024	0.100	0.555	

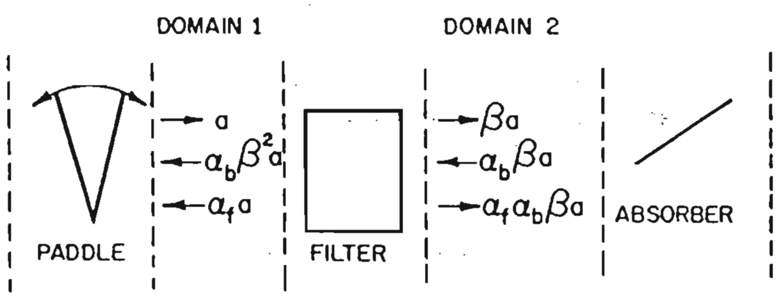

FIG. 1.—WAVE ANALYSIS

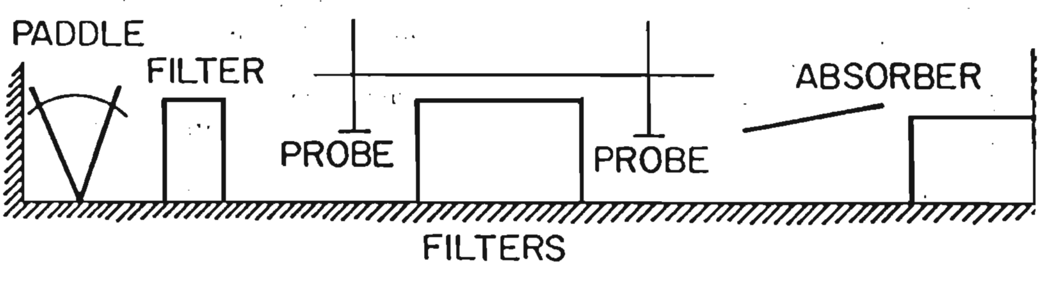

FIG. 2.—CHANNEL LAYOUT

exhibit linearity; additionally their transfer functions should not be both amplitude and frequency-dependent. Long, narrow, and shallow channels are of course low-pass devices also, and equivalent transfer functions for long propagation distances are of interest. Attenuation by wall friction is thought to be linear and amplitude-independent and consequently it follows that filters of similar characteristics to long narrow channels are of special interest.

A widely used type of filter comprises a bundle of fibers, from each strand of which small eddies and drag forces are generated by the instantaneous or-

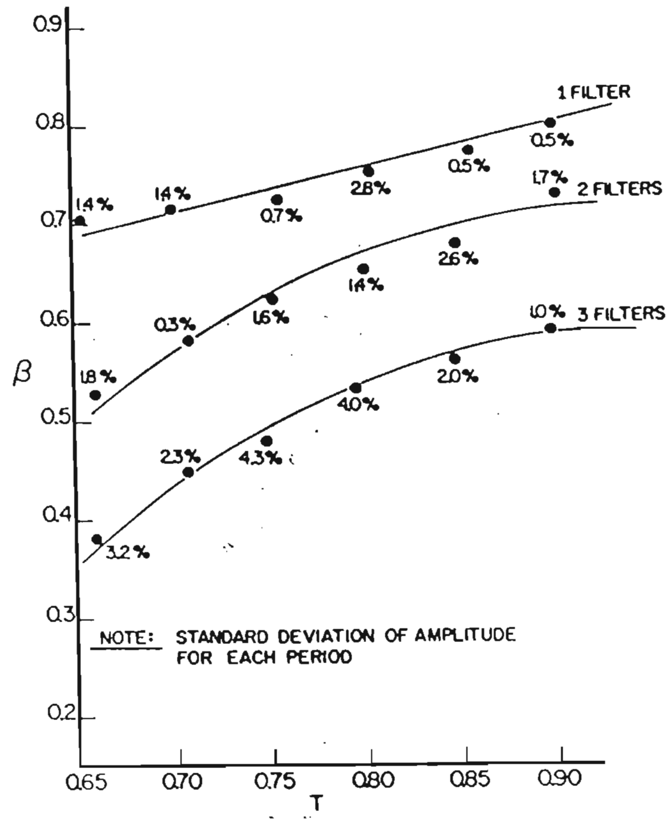

FIG. 3.—TRANSMISSIVITY MEANS AND STANDARD DEVIATIONS FOR ONE, TWO, AND THREE FILTERS

bital velocities of the wave spectrum. Such local head losses are likely to be related to the square of the instantaneous orbital velocity. However, in a laminar wall boundary layer, the head losses are probably linearly related to the local instantaneous orbital velocity.

For these reasons, several filters, each comprising a series of flat plates arranged in the orbital planes from the bed through the surface, were used in this study (1). Such filters incorporate comparatively large wall areas.

EXPERIMENTS

Equipment.

Absorbers.—The wave absorber used was similar to that used earlier by

James (3); the length [8 ft (2.44 m)] was chosen to exceed the maximum wave length tested. The slope of the beach, when fixed (rigidly) in position, therefore, did not exceed the minimum wave steepness used. The absorber was constructed from a 2-ft (0.62-m) wide, 1/4-in. (0.63-cm) thick sheet of aluminium alloy, in which evenly spaced 3/4-in. (1.90 cm) holes were drilled at 2-in. centers. The plate was stiffened with extruded angle sections along the edges, which in turn were tightly clamped to the channel walls. Results of experiments carried out on the beach are shown in Table 1; an approximately null reflection function, α_b, is obtainable (5) with this filter. (In these tests minor difficulties were experienced in generating small amplitude line spectra.)

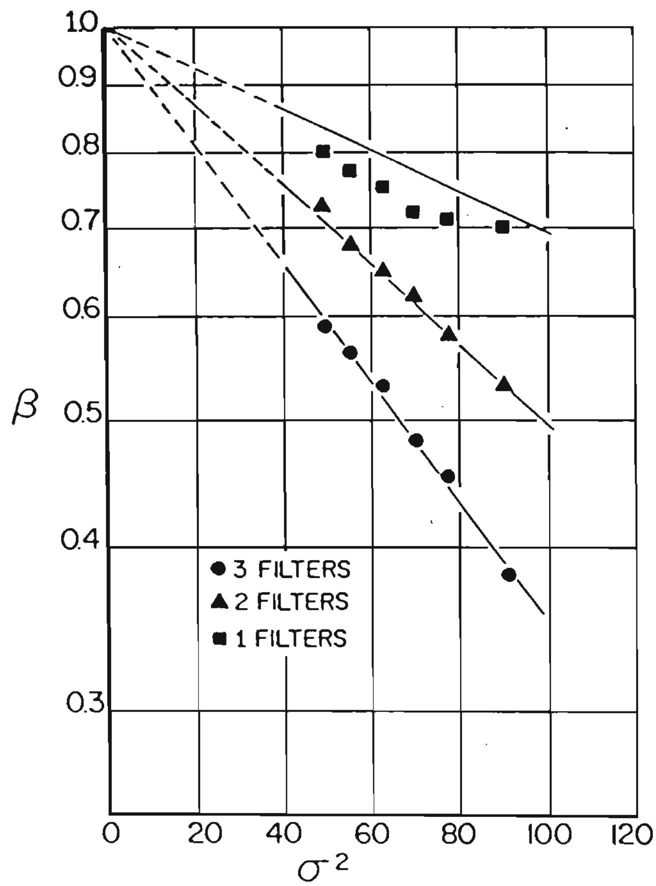

FIG. 4.—TRANSMISSIVITY COMPUTED USING EQ. 1

Probes.—Two capacitance-type probes were used in the experiments- capacitance between a horizontal plate suspended above the water and th water (ground) caused a change in the feedback circuit of the transducer am plifier. After amplification this change was relayed to a strip recorder. Th probes were mounted on triangular carriages which moved 25 ft (7.62 m) o level track. This allowed measurement of reflectivity and transmissivity i the usual way (4) (see Fig. 1). The probes were calibrated before each te series in order to counter any temperature-humidity effects, which were however, generally negligible.

Filters.—Four of the filters originally developed by the writers (3) were constructed by wrapping a continuous roll of fiberglas gauze on a rectangular frame. An adjustable support at the top of the framework was constructed to maintain tension in the fiberglas screening, which was 4 ft (1.22 m) wide and 0.015 in. (0.38 mm) thick. Each filter was 4 ft (1.22 m) long and up to three were tested in a series.

Results.—The equipment was set up in a 2-ft (0.61-m) wide channel as shown in Fig. 2. The depth of water in the channel was maintained constant throughout the experiments at 1.83 ft (0.557 m).

Monoperiodic Waves.—Experiments were first conducted on a range of monoperiodic waves of various periods and amplitudes, and the reflectivity and transmissivity of a filter bank comprising one, two, and three filters were measured. The results are presented in Figs. 3 and 4 and in Table 1. The results showed that for these filters transmissivity was sensibly independent of amplitude, but increased with period. This contrasts with Keulegan's results (5) in which the transmissivity of filters arranged across the orbital plane was found to be amplitude-dependent.

The equation plotted in Fig. 4 is

$$\beta = e^{-0.0009 L_f \sigma^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Note that measured filter reflectivity (α_f) was negligible; the filter reflection function is thus also null.

Two-Component Spectra.—A second series of tests was carried out to determine to what extent the filtering of simple two-component wave spectra is linear. The wave frequencies were limited to those whose beach reflectivity was small. Nine permutations of amplitude ratios of 1:1, 1:2:, 1:3 were used for each of six wave-frequency combinations. In each test a 9-ft (2.74-m) wave envelope was recorded in each domain in order to determine reflectivity. The wave analysis was based on an assumption of a line spectrum, and of linear (first order) waves, because incident amplitudes were kept small (typically 0.5 cm). A 12-ft (3.66-m) filter length was used in all of these tests. Selected results together with relevant monoperiodic wave results are shown in Figs. 5 to 7 inclusive. Significant standard deviations were obtained but the results indicate that the filter behavior is linear for these wave spectra, at least to the first order. The discrepancies include errors inherent in the assumptions underlying the wave analysis procedure, and errors arising from traces of waves of other frequencies.

Channel Attenuation.—Treloar and Brebner recently reported (6) tests conducted in the same laboratory. Attenuation was measured in artificially stretched flumes 6-in. (15.24-cm) and 36 in. (0.92 m) wide. The measuring equipment used was generally similar to that described previously.

The main contribution of their study lay in isolating the attenuation caused by bed friction in wide, shallow channels, but their experiments may provide some insight into the effect of low-pass plate filters with a high cut-off period.

Clearly, increasing plate spacing, or decreasing filter density, decreases the cut-off frequency, and a 6-in. (15.24-cm) channel would represent an extremely transmissive filter. Treloar's empirical formulation for attenuation therefore would approximate an asymptote for plate filter transmissivity:

$$\beta = e^{-KL_f}; \; K = \frac{k}{s}\left(\frac{T\upsilon}{\pi}\right)^{1/2}\left(\frac{1.48sk + 0.94 \sinh 2kh}{2kh + \sinh 2kh}\right) \quad \ldots\ldots\ldots (2)$$

APPLICATION TO WAVE SPECTRA

The filter transmission transfer function determined for the monoperiodic waves and a 1-in. (2.54-cm) filter density was applied to both a peaked spec-

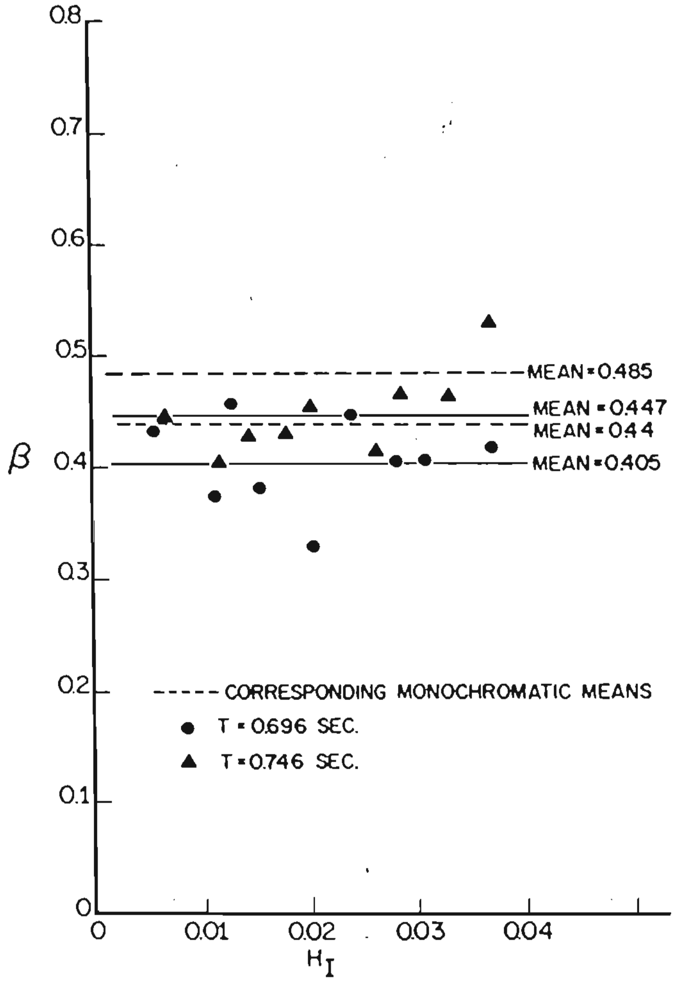

FIG. 5.—TYPICAL RESULTS FOR TWO-COMPONENT SPECTRA

trum (based on the Bretschneider spectral energy distribution) and a flat wav spectrum, assuming linearity. The results are shown in Figs. 8 and 9, fo various filter lengths.

A transfer function was also calculated using Eq. 2, in which the chann width has been replaced by S, the filter plate spacing. It was found that bette results for the 1-in. (2.54-cm) spacing were obtained by setting S in Eq. 2 t 0.6 of the actual value. This is not surprising because boundary layers c

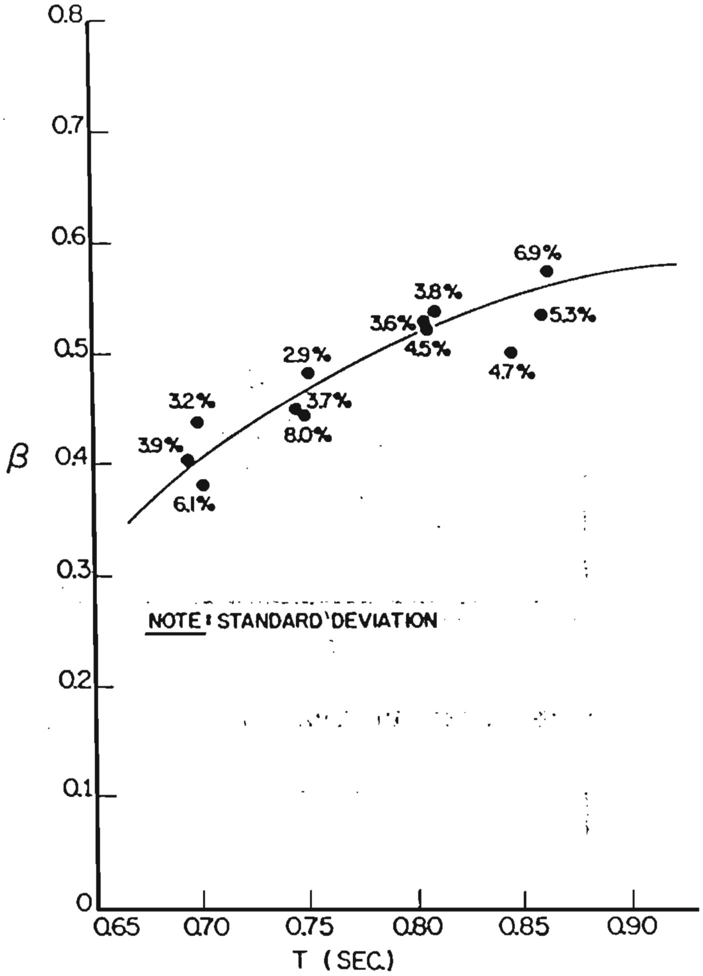

FIG. 6.—TRANSMISSIVITY MEANS AND STANDARD DEVIATIONS

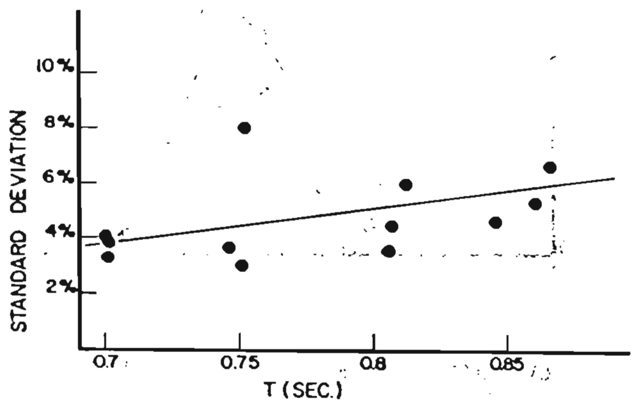

FIG. 7.—STANDARD DEVIATIONS

the filter plates are relatively thicker, and also in view of the basic differences between Eqs. 1 and 2; further experimentation is required.

The results are plotted in Fig. 10 for various filter densities. Figs. 8-10

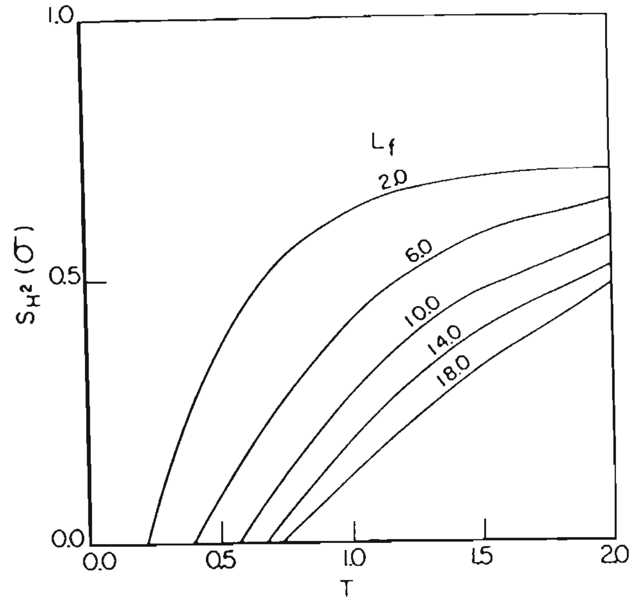

FIG. 8.—TRANSMITTED SPECTRA, 1-IN. PLATE SPACING

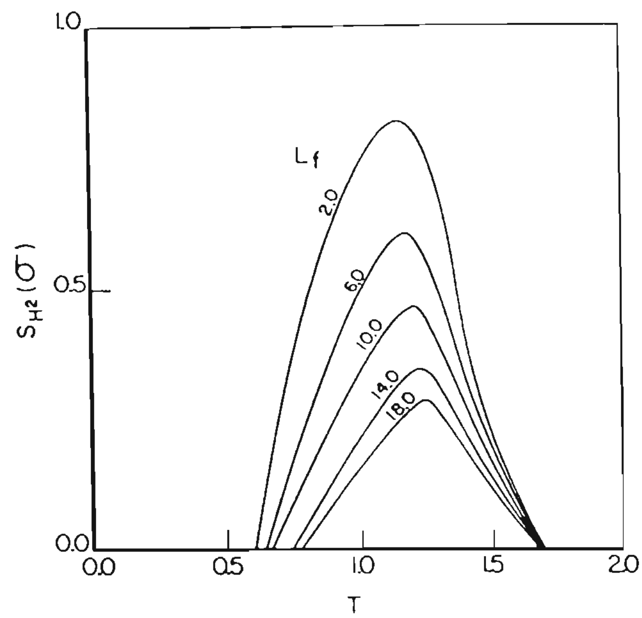

FIG. 9.—TRANSMITTED SPECTRA, 1-IN. PLATE SPACING

indicate the effect of a filter bank placed between a spectral wave generator and the measuring station in an experimental flume. A more detailed comparison does not appear warranted because of the various errors reported herein. Readers should appreciate the need to interpret these data carefully,

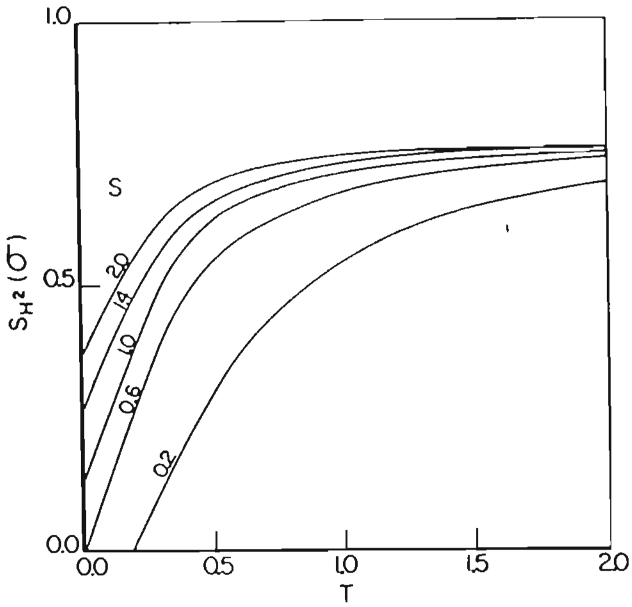

FIG. 10.—TRANSMITTED SPECTRA, 12-FT FILTER

in view of the limitations of the study and the danger of nonlinear interaction in their particular spectra.

CONCLUSIONS

The experimental program described herein was conducted in order to ascertain whether planar wave filters were linear to the first order, using sensibly low-amplitude waves. It is believed that sufficient evidence has been derived to indicate that these filters are probably more promising in this respect than filters causing energy dissipation through eddy formation and turbulence.

A transfer function was determined experimentally for a bank of filters using fiberglas screening at 1 in. (2.54 cm) spacing. That transfer function was applied to a flat incident spectrum and to a peak, Bretschneider, spectrum.

A similar calculation was carried out using an established equation for wave attenuation in long narrow channels, and the results were similarly plotted to compare with those obtained using the empirical transfer function. The equations give an estimate of transmissivity for various filter lengths and densities (plate spacing).

Such filters should prove valuable in studies involving spectral wave generators, especially where the experimental situation requires effective decoupling of the wave channel or wave tank. Furthermore, the results should allow the design of approximately linear low-pass filters, i.e., filter length and density, to achieve a required wave transmittance.

ACKNOWLEDGMENTS

The experiments were conducted by Mike Garrett under the writer's suppervision during a visiting appointment at Queen's University. Earlier tests by the writer on a prototype were financed by a grant at the University of Natal (and CSIR). The writer would like to express his gratitude for the leave arrangements at both universities during 1970 and to A. Brebner for providing generous laboratory facilities for the final experimental program.

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APPENDIX II.—NOTATION

The following symbols are used in this paper:

a = wave amplitude, in feet (meters);
H_I = incident wave height, in feet (meters);
h = depth of water, in feet (meters);
K = attenuation modulus;
$k = 2\pi/L$ = wave number;
L_f = filter length, in feet (meters);
s = spacing of parallel screens, in feet (meters);
T = wave period, in seconds;
α_b, α_f = reflectivity due to beach, filter;
α_1, α_2 = reflectivity in domain 1, domain 2;
β = transmissivity;
υ = kinematic viscosity, in square feet per second (square meters per second); and
$\sigma = 2\pi/T$, in radians per second.

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Brief Reports

Multilag Multivariate Autoregressive Model for the Generation of Operational Hydrology

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Abstract. The multivariate model previously described by N. C. Matalas is extended to the multilag case.

A model currently popular in the generation of operational hydrology is the lag 1 multivariate autoregressive process [*Matalas*, 1967; *Young and Pisano*, 1968]. In many cases this model adequately describes the combined streamflow of a region, but its extension to a multilag model may be useful since streamflow records have occasionally been found not to resemble realizations of simple Markov processes [*Fiering*, 1967]. This report accomplishes the extension.

All variables are treated as stationary and normally distributed with zero means. Recognized transformations exist to do so [*McGinnis and Sammons*, 1970].

Let $y_t = [X_{1t}, X_{2t}, \cdots, X_{nt}]'$ be a vector of random variables such that the process $\{y_t\}$ is defined by the autoregressive equation

$$y_t = \sum_{i=1}^{q} A_i y_{t-i} + B\epsilon_t \tag{1}$$

Here $\{\epsilon_t\}$ is a sequence of vectors of mutually and serially uncorrelated normally distributed random variables with zero means and unit variances, and the A_i and B are $n \times n$ matrices. Let the lag j variance-covariance matrix of $\{y_t\}$ be $M_j = E[y_t y_{t-j}'] = M_{-j}'$ (the prime indicates matrix transposition). In general $M_j \neq M_j'$, $j > 0$. Postmultiplying (1) by y_t' and taking expectations, we have that the dispersion matrix of $\{y_t\}$

$$M_0 = \sum_{i=1}^{q} A_i M_{-i} + E[B\epsilon_t y_t']$$

$$= \sum_{i=1}^{q} A_i M_i' + BB'$$

as $E[\epsilon_t y_{t-i}'] = 0$ for $i > 0$; thus

$$BB' = M_0 - \sum_{i=1}^{q} A_i M_i' \tag{2}$$

When the A_i and M_i are given, B can be found as a lower triangular matrix from (2) by the Gram-Schmidt orthogonalization process [e.g., *Franklin*, 1968].

Postmultiplying (1) by y_{t-j}' and taking expectations, we obtain

$$M_j = \sum_{i=1}^{q} A_i M_{j-i} \qquad j = 1, 2, \cdots, q \tag{3}$$

which in matrix form can be expressed as

$$[M_1 M_2 \cdots M_q] = [A_1 A_2 \cdots A_q] \cdot \begin{bmatrix} M_0 & M_1 & \cdots & M_{q-1} \\ M_1' & M_0 & \cdots & M_{q-2} \\ \vdots & \vdots & & \vdots \\ M_{q-1}' & M_{q-2}' & & M_0 \end{bmatrix}$$

Accordingly the A_i are given by the solution of the set of q matrix equations:

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$$[A_1 A_2 \cdots A_q] = [M_1 M_2 \cdots M_q] \cdot \begin{bmatrix} M_0 & M_1 & \cdots & M_{q-1} \\ M_1' & M_0 & & \\ \vdots & & & \\ M_{q-1}' & & & M_0 \end{bmatrix}^{-1} \quad (4)$$

If q and n are large, the setting up and solution of (4) present difficulties. If, however, the A_i are diagonal matrices, both aspects of the problem are eased.

Considering the physical phenomenon of streamflow generation, we note the following:

1. The groundwater recession curve of a hydrograph is usually assumed to be exponential in form, so that two different recession curves would form deterministic, mathematically independent sequences.
2. The streamflow during a particular period is a fair indicator of the state of storage in the catchment.
3. Most sequences of rainfall residuals appear to be serially uncorrelated [*Roesner and Yevjevich*, 1966; *Pegram*, 1970] although they may be highly interdependent.

When (1) is used for the generation of operational hydrology, the preceding points taken in turn tend to justify the diagonalization of the A_i, the multilag nature of the process, and the form of the error term.

Defining the matrix $S = \text{diag}\ [\sigma_{11}, \sigma_{22}, \cdots, \sigma_{nn}]$, where σ_{ii} is the standard deviation of $\{X_{it}\}$, and R_j as the lag j correlation matrix of $\{y_t\}$, we have $M_j = SR_jS$. From (3)

$$M_j = SR_jS = \sum_{i=1}^{q} A_i SR_{j-i} S$$

which if the A_i are all diagonal, becomes

$$S[R_j]S = S\left[\sum_{i=1}^{q} A_i R_{j-i}\right] S$$

so that we can rewrite (4) as

$$[A_1 A_2 \cdots A_q] = [R_1 R_2 \cdots R_q] \cdot \begin{bmatrix} R_0 & R_1 & \cdots & R_{q-1} \\ R_1' & R_0 & & \\ \vdots & & & \\ R_{q-1}' & & & R_0 \end{bmatrix}^{-1} \quad (5)$$

(Note that the operation of standardizing the $\{X_{it}\}$ to unit variance presupposes that the A_i are diagonal.)

An advantage of using (5) over the equivalent form (4) is that because $|R_{ijk}| \leq 1$, the computation is likely to be more stable.

From (5) it is evident that the equations involving the diagonal terms of the R_i form a set independent of those equations involving the off-diagonal terms. Thus the diagonal terms are linearly related to the off-diagonal terms through the A_i. The easiest parameters to estimate are the serial correlation coefficients (the diagonal terms); thus we derive the A_i by solving n sets of q equations, each having the form

$$\begin{Bmatrix} a_j(1) \\ a_j(2) \\ \vdots \\ a_j(q) \end{Bmatrix} = \begin{bmatrix} 1 & \rho_{jj}(1) & \cdots & \rho_{jj}(q-1) \\ \rho_{jj}(1) & 1 & \cdots & \\ \vdots & & & \\ \rho_{jj}(q-1) & & & 1 \end{bmatrix}^{-1} \begin{Bmatrix} \rho_{jj}(1) \\ \rho_{jj}(2) \\ \vdots \\ \rho_{jj}(q) \end{Bmatrix} \qquad j = 1, 2, \cdots, n \quad (6)$$

Here $a_j(i)$ is the jjth term of A_i, and $\rho_{jj}(i)$ is the jjth term of R_i. Of course, $\rho_{jj}(0) = 1$.

Rewriting (5) as

$$\begin{bmatrix} R_1' \\ R_2' \\ \vdots \\ R_q' \end{bmatrix} = \begin{bmatrix} R_0 & R_1 & \cdots & R_{q-1} \\ R_1' & R_0 & & \\ \vdots & & & \\ R_{q-1}' & & & R_0 \end{bmatrix} \begin{bmatrix} A_1 \\ A_2 \\ \vdots \\ A_q \end{bmatrix} \quad (7)$$

(we may do so because the A_i are diagonal), we have $qn(n-1)$ equations in the remaining unknowns (which total $qn(n-1)$ for the off-diagonal terms of $R_1, R_2, \cdots, R_q$ and $n(n-1)½$ for the off-diagonal terms of R_0). If R_0

is known, the system in (7) is nonsingular, and we provide the cross correlation for the process in the obvious way.

A plausible way of calculating the R_i is iteratively from (7); we cannot solve for the R_i explicitly since $R_i \neq R_i'$; i.e., we have $2q - 1$ matrix 'unknowns' and only q matrix equations. If, however, an explicit solution of (7) is required, we must resort to the elemental form; e.g., if $n = q = 2$, (7) is

$$\begin{bmatrix} \rho_{11}(1) & \rho_{21}(1) \\ \rho_{12}(1) & \rho_{22}(1) \\ \rho_{11}(2) & \rho_{21}(2) \\ \rho_{12}(2) & \rho_{22}(2) \end{bmatrix} = \begin{bmatrix} 1 & \rho_{12}(0) & \rho_{11}(1) & \rho_{12}(1) \\ \rho_{21}(0) & 1 & \rho_{21}(1) & \rho_{22}(1) \\ \rho_{11}(1) & \rho_{21}(1) & 1 & \rho_{12}(0) \\ \rho_{12}(1) & \rho_{22}(1) & \rho_{21}(0) & 1 \end{bmatrix} \begin{bmatrix} a_1(1) & \cdot \\ \cdot & a_2(1) \\ a_1(2) & \cdot \\ \cdot & a_2(2) \end{bmatrix}$$

The unknown off-diagonal terms form a set of equations

$$\begin{Bmatrix} \rho_{12}(1) \\ \rho_{21}(1) \\ \rho_{12}(2) \\ \rho_{21}(2) \end{Bmatrix} = \begin{bmatrix} \cdot & a_1(1) & \cdot & a_1(2) \\ a_2(1) & \cdot & a_2(2) & \cdot \\ \cdot & a_1(2) & a_1(1) & \cdot \\ a_2(2) & \cdot & \cdot & a_2(1) \end{bmatrix} \begin{Bmatrix} \rho_{12}(0) \\ \rho_{21}(0) \\ \rho_{12}(1) \\ \rho_{21}(1) \end{Bmatrix} \tag{8}$$

or since $\rho_{12}(0) = \rho_{21}(0)$ are known,

$$\begin{Bmatrix} \rho_{12}(1) \\ \rho_{21}(1) \end{Bmatrix} = \begin{bmatrix} 1 & -a_1(2) \\ -a_2(2) & 1 \end{bmatrix}^{-1} \begin{Bmatrix} a_1(1) \\ a_2(1) \end{Bmatrix} \rho_{12}(0)$$

from which we get $\rho_{12}(2)$ and $\rho_{21}(2)$ by us (8). Generalization to larger n and q is a di extension of this argument.

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A NOTE ON INEXPENSIVE TELEMETRY OF RIVER SEDIMENT CONCENTRATIONS

Une Note sur Télémétrie à Prix Bas Mesurant la Concentration de Sédiments des Rivières

W. JAMES* and G. D. FINLAYSON**

SYNOPSIS. An inexpensive method for continuous monitoring of suspended sediment concentrations in rivers has recently been developed at the University of Natal. The instrument uses newly available solid state electronic circuitry and has been thoroughly laboratory-tested. The circuitry and the operation of the instrument are described in general terms.

RÉSUMÉ. Une méthode économique a été récemment développée à l'Université de Natal pour la surveillance successive des concentrations de sédiments en suspensions des rivières. L'instrument se met en jeu par circuits électroniques solides normals, récemment procurables, et a été examiné à fond en laboratoire. Le circuit et la mode d'opération de l'instrument est décrit en termes généraux.

Les études récentes à l'Université de Natal ont constitué le premier échelon de dévelopement pour une méthode de surveillance automatique dans les rivières de Natal, et pour télémétrer les données des stations lointaines à une station de base centrale. Les exigeances fondamentales envers l'instrument étaient: une construction vigoureuse, pour pouvoir resister aux rigueurs en plein air, une opération sans interromption pendant 2 semaines, fonctionnant d'accumulateurs, et une ponctualité de première ordre.

Le système de circuit employé dans ce projet sera certainement bientôt démodé, pourtant il a été trouvé économique et évidemment facile à manier et à maintenir.

INTRODUCTION

Recent studies[1,2] at the University of Natal constitute the first stage of the development of an automatic method to monitor suspended sediment loads in Natal rivers and to telemeter the data from remote field stations to a central base station. The basic design requirements of the instrument were: sturdy construction to withstand the rigors of field use; continuous operation for two weeks on power from lead accumulators; and first order accuracy only, because of the nature of the problem.

The sensor in the feasibility study was attached to at bridge pier above the river bed. It consisted of three separate transducers:

(i) a turbidity meter comprising a light source and two photoconductive cells, to give a first order approximation to the sediment concentration[1],
(ii) a pressure cell acting as a depth gauge, for river stage,
(iii) a Savonius rotor velocity meter, tested and found to be reasonably self-cleaning in water containing grasses and loose debris.

All three sensors were attached to a submerged, quarter-inch thick steel plate, mounted and streamlined to deflect river debris, as shown in *Fig. 1*. A sealed, water-proof electrical cable linked the sensors with the field station, located on the pier beneath the bridge super-structure, well above flood level. The cost of the materials used in the sensors was negligible.

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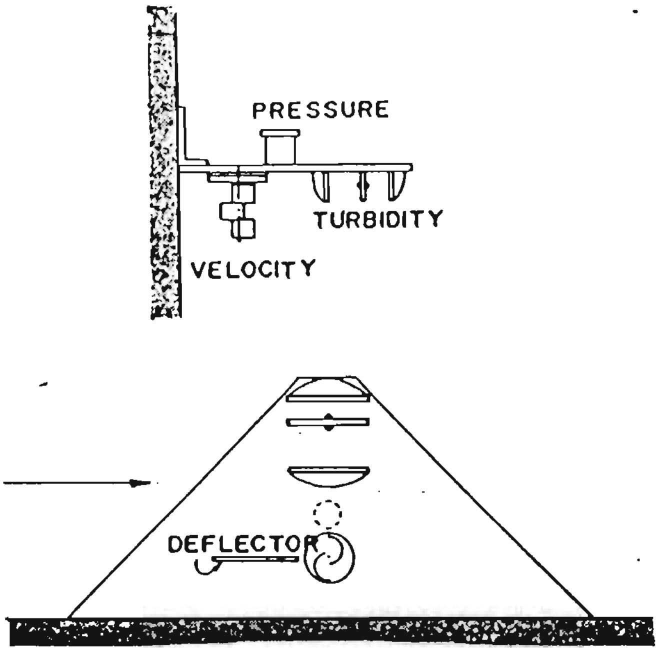

Fig. 1.

FIELD STATION

The field station converts the outputs from the three transducers to a form compatible with the rest of the circuitry (analyser circuits), multiplexes the data chanels and encodes each channel into a form suitable for radio transmission. All the information may then be transmitted over a single radio frequency as shown in the block diagram *(Fig. 2.)*.

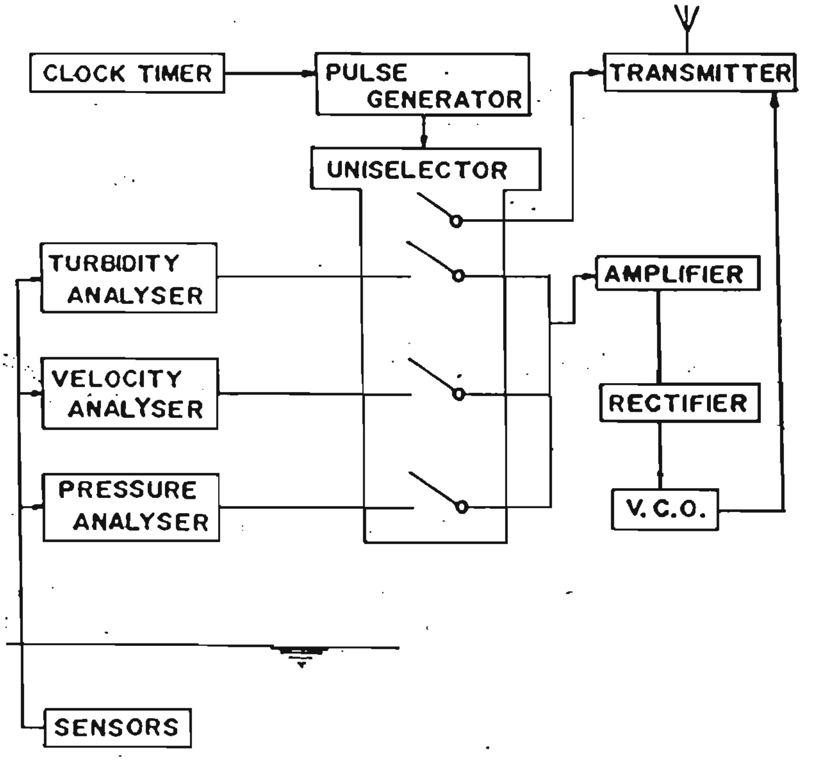

Fig. 2.

An electric clock timer steps the field station down between transmissions in order to conserve the charge in the accumulators. This up-dating time, which may be preselected by the designer, should be related to the concentration time of the catchment, and the required accuracy. For the Umgeni River catchment (1,600 square miles) an up-dating time of fifteen minutes was chosen.

When the clock activates the station, a pulse generator emits a square wave pulse every six seconds. This pulse operates a relay to drive a uniselector (rotary switch). The uniselector steps once for each pulse emitted. On the first step the uniselector switches the transmitter to STAND-BY and no further operations occur for the next five steps (thirty seconds). (This delay allows the transmitter valves sufficient time to warm up and should be omitted for a solid state transmitter). Then the uniselector switches the transmitter to TRANSMIT and the multiplexing sequence begins. The first transmitting step could be used for station identification: a small standard voltage may be read through the uniselector to the encoding circuitry. Thereafter the outputs of the turbidity, velocity and depth analyser circuits are read through the uniselector in turn. Each reading is taken over a six second period thus reducing sampling errors arising from river turbulence. The last operation of the uniselector switches off the transmitter and teps down the whole field station.

The cost of components used in the field station is given in the Appendix.

Radio link

The signals emitted from the analyser circuits and relayed through the uniselector to the encording circuitry are 1,000 hz. A.C. voltages in the 2–12 mV amplitude range. A miniature integrated-circuit audio-amplifier, (gain about 500) boosts these voltages, and the output is rectified on a full wave bridge (rectifier bridges are available as integrated circuits) to produce a D.C. voltage in the 1–6 volt range. This is fed into a voltage controlled oscillator which produces an audio tone in the 500 hertz to 10 kilohertz range with frequency dependent on input voltage (Encoder). The audio tone is then relayed over the radio link to the base station.

At the base station, a monostable multivibrator attached to the radio receiver is triggered by the waveform received and, if used in conjunction with a pen-recorder, it can record station identification and river data. Alternatively, the frequency could be digitized automatically, and fed into a computer to give a display or record of the sediment load in the river at the field station.

The cost of the VCO is included in the Appendix, but no costs of the radio equipment or base station recorders are included therein.

Conclusions

Data collection systems for river turbidity and discharge have generally been expensive. This study has shown that economical systems are possible. Many types of circuitry are available in standard integrated-circuit form and their costs are declining steadily as usage increases. The circuitry used in this project will certainly be outmoded soon, but the system was inexpensive and evidently is easy to operate and maintain.

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ACKNOWLEDGEMENTS

This research was funded by a grant administered by the S. African C.S.I.R. Thanks are due to th Department of Civil Engineering at the University of Natal for facilities, and to the Durban City Engi neer for providing transport and access to the Umgeni River at the Ellis Brown Viaduct. Radio equip ment was kindly donated by Messrs. PYE, S. Africa Ltd., and Mr. L. Eaton rendered invaluable assistan ce in the design of the station circuitry.

APPENDIX: SYSTEM COSTING

These costs applied in South Africa in December 1969. The cost of electronic components has sinc fallen, and the cost of materials, increased. No attempt was made to assess labour costs.

	Approx U.S. dollar
Active Components (transistors, diodes, unijunctions)	23
Resistors and potentiometers	6
Capacitors	6
Relays and uniselector	16
Lamp bulbs	1
Plugs and sockets	7
Plug-in circuit board	16
Electric clock	10
Batteries	271
Aerial	20
Field Station housing	92
Electronics housing	7
Multicore cable	5
Transducer shell	13
Radio transmitter and receiver donated	—
Total approx.	$500

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A NEW RECORDING TURBIDITY METER FOR RIVERS

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SYNOPSIS

An inexpensive continuously recording turbidity transducer on two different types of mounting has been developed at the University of Natal, and installed at two stations in the Umgeni River system. The geometry and components of the transducers are detailed, and a sampler used to calibrate the transducers is also described. A portable turbidity meter has also been developed.

INTRODUCTION

During a recent study [1], a continuously recording turbidity meter was developed and placed in the Umsindusi River upstream of Henley Dam, near Pietermaritzburg in Natal. Major difficulties experienced at this station arose from encrustation of fine solid matter on the glass surfaces during recession flows, from algal growths during winter flows, and from the

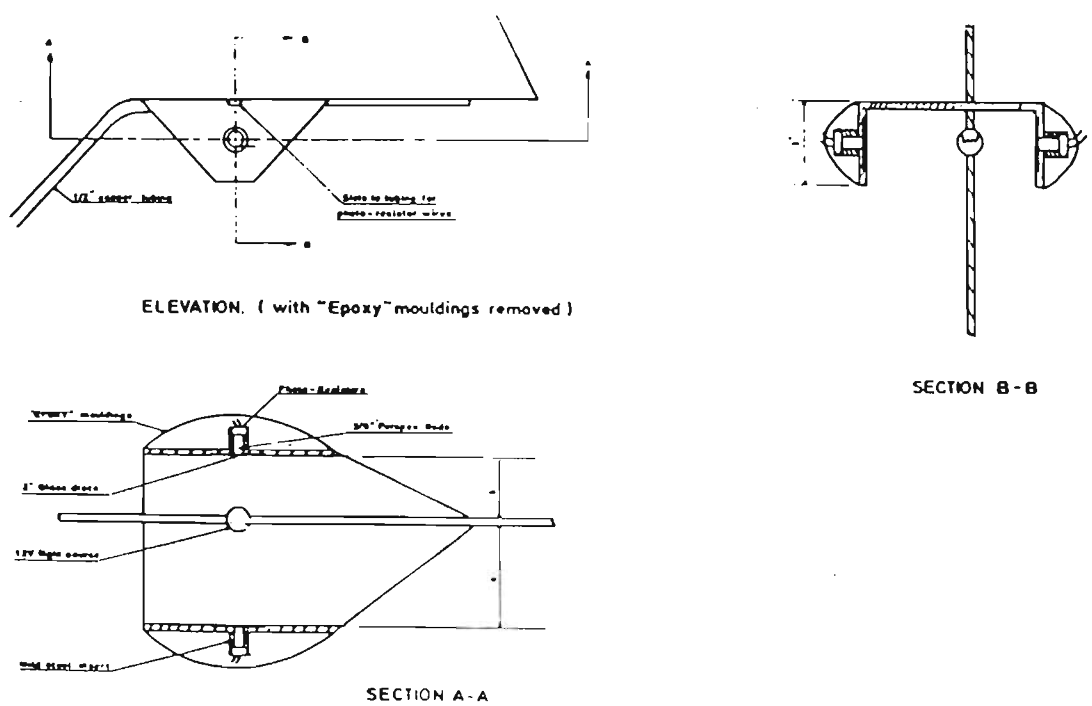

Fig. 1 — Photo-Electric Turbidity meter (Scale: 1" = 4").

snagging of grass and trees during floods. The first type of mounting described below was evolved, and the results so far indicate that these difficulties have been adequately overcome.

Subsequently a second turbidity meter was placed in the Umgeni River at the Natal Estates pumping weir near Durban in the same river system. This site proved difficult because of frequent periodical changes in bed level of the order of 4 ft., and because of vandalism. The second type of instrument described below was developed, and it is hoped to attach these instruments to several bridge piers in the catchment.

Turbidity meter

Basically the method uses a single small electric bulb transmitting light through the river water to two photoresistors spaced unequal distances from the light source. The effect of extraneous light is reduced by the top plate and by the directional selectivity of the light entering the photo-resistor, because of the two polished "perspex" rods shown in figure 1.

The meter is mounted below water level, attached either to a heavy steel framework resting directly on a rocky river bed (first type), or to a bridge pier (second type). The meter is robust;

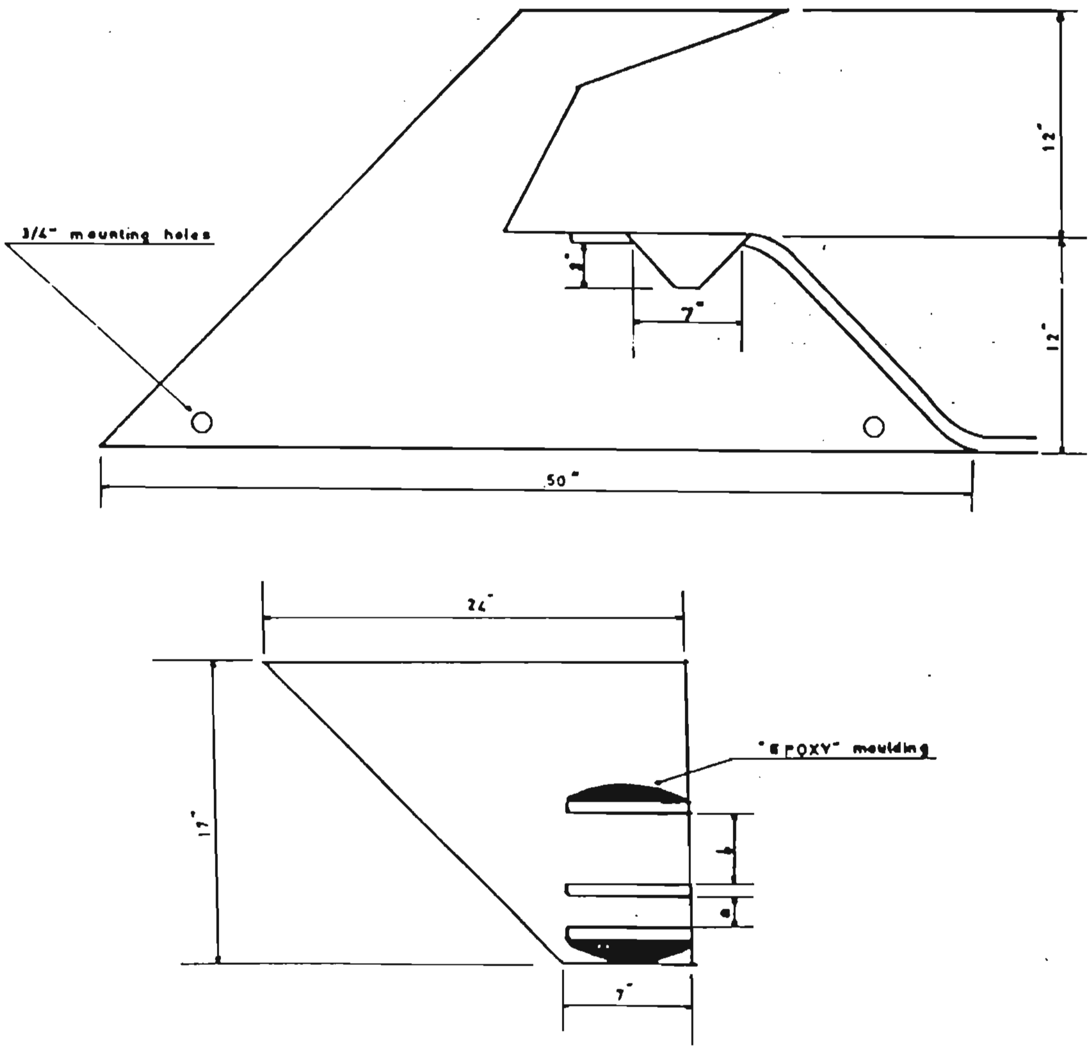

Fig. 2 — Mountings: First type—Side View
Second type—View from below
(Scale: 1" = 10")

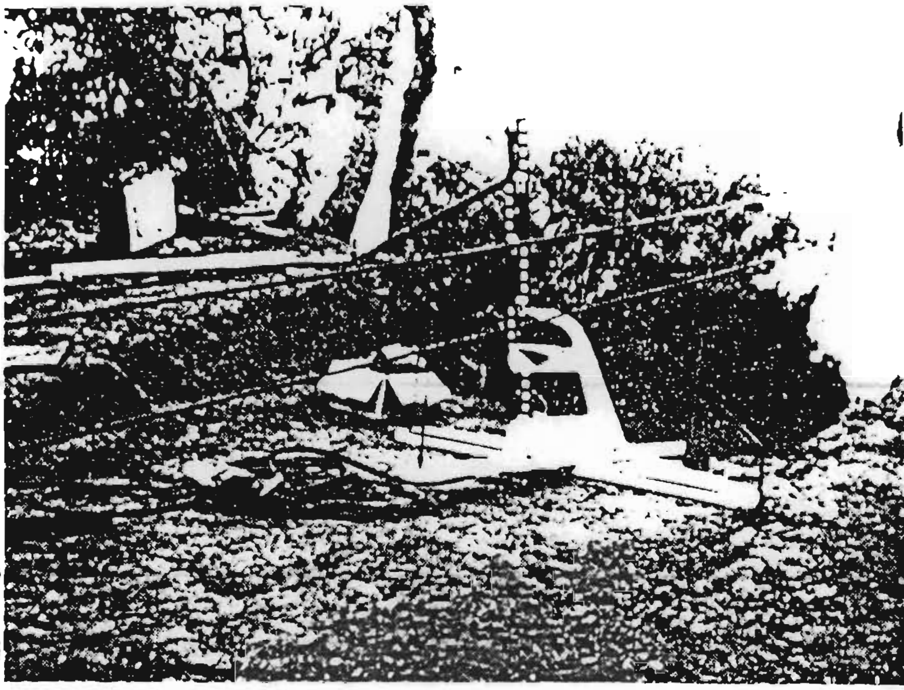

Fig. 3

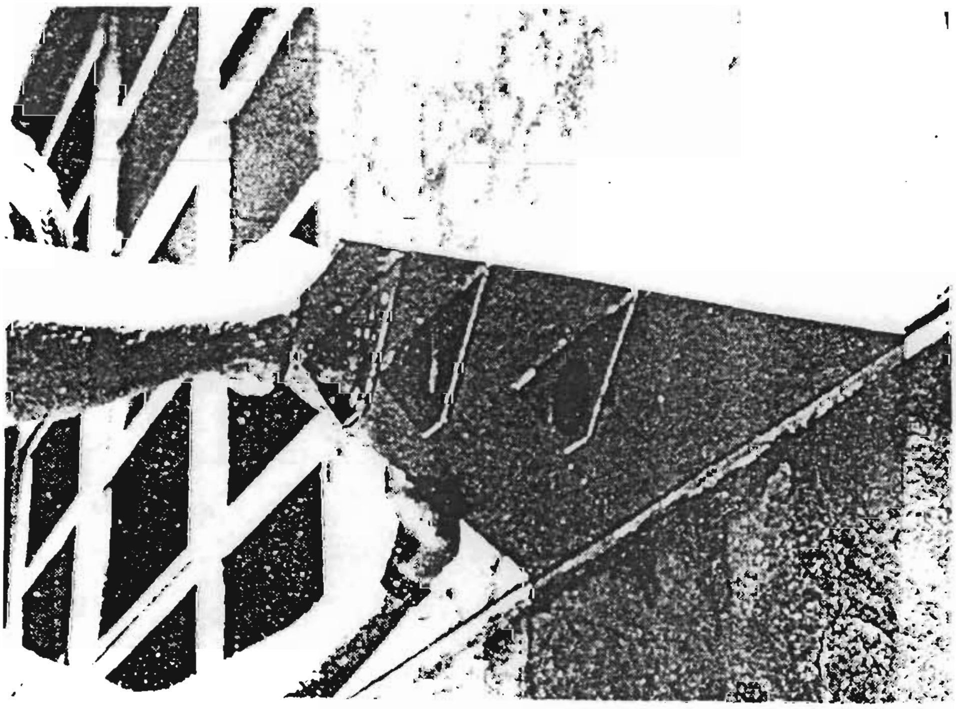

Fig. 4

electrical components are mounted in 0.375 inch mild steel plate and rolled steel channel sections, welded together and carefully shaped as shown in figure 2 to deflect flood debris. Figures 3 and 4 show the first and second type of turbidity meter respectively.

Two electrical wires from the small electric light bulb and two from each of the two photo-resistors pass through a sealed hosepipe from the meter to the river bank. Electrical apparatus housed in a steel container on the river bank supplies current to the light bulb and measures the impedances of each of the two photo-resistors. Suitable subtraction of the two resistances eliminates any effect of algal growth on the glass surfaces of the meter and indicates the decrease in light intensity over a distance equal to the difference in length of the light paths. At present, these resistances are measured by an "out-of-balance" bridge circuit and recorded on continuous chart recorders.

Sediment sampler

The sampler developed (fig. 5) to calibrate the turbidity meter consists of a tube, two inches in diameter, with water-tight flaps at each end, a catch mechanism for releasing these flaps, stabilising fins and a large tail fin to orient the sampler along a streamline. Dimensions are given in figure 6. The centre of support coincides with the submerged centre of gravity.

The sampler is positioned above the stream or river (e.g. using a cableway) and is lowered into the stream to any required depth. The sampler rapidly orients because the centre of drag is considerably downstream of the centre of support. River water passes without obstruction through the open tube. A jerk on the single cable suspending the sampler releases the flaps and the sampler with sample may be withdrawn.

Samples are being analysed gravimetrically and correlated against the turbidity recordings.

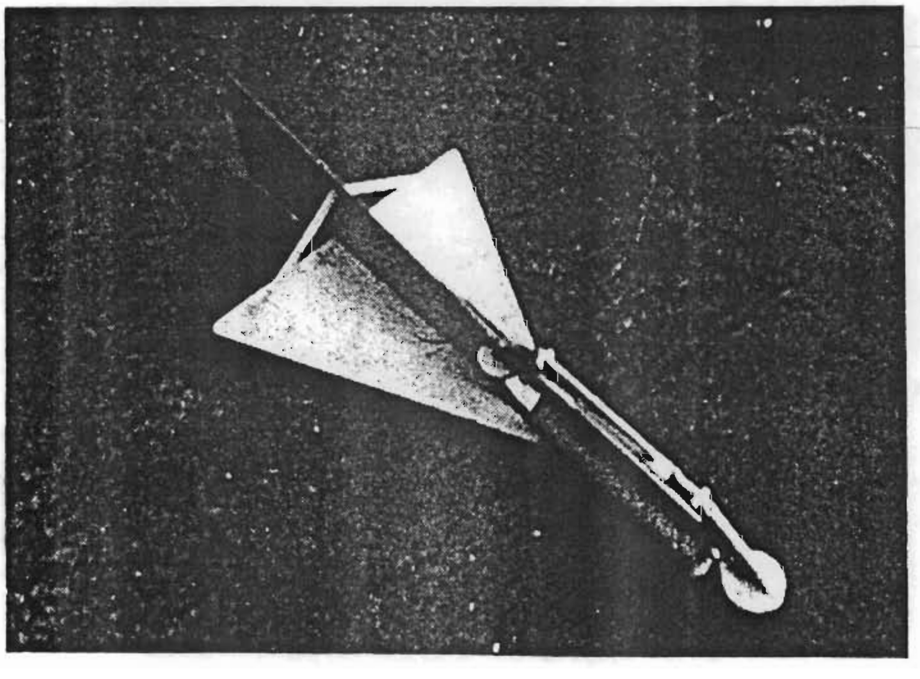

Fig. 5

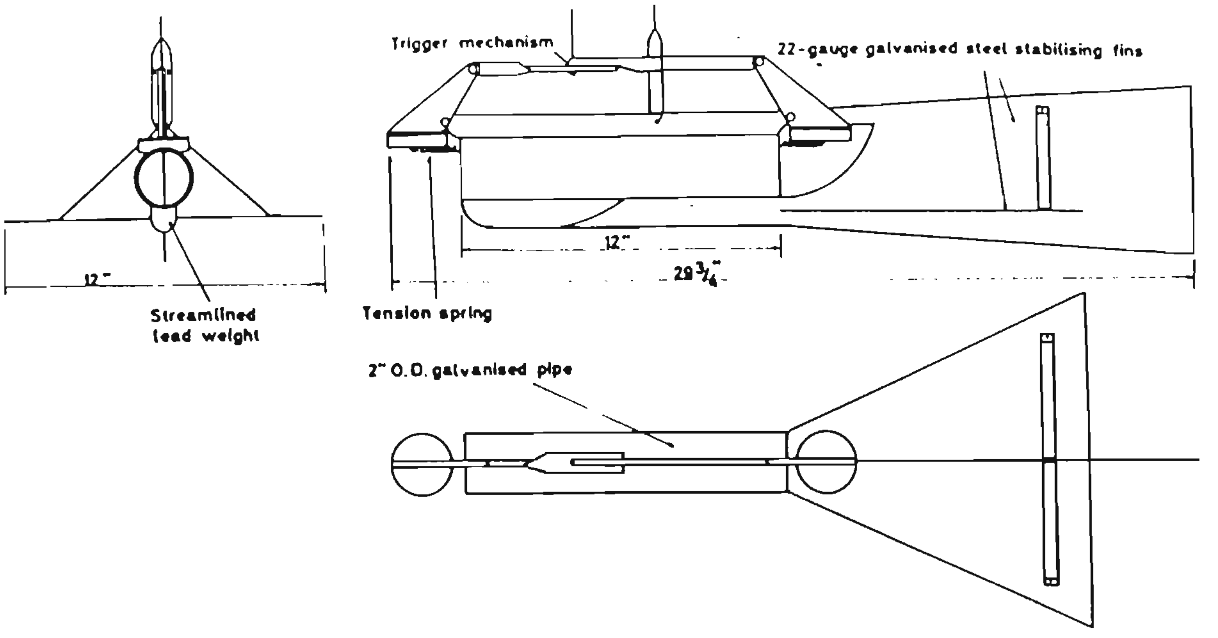

Fig. 6 — Suspended sediment Sampler (Scale: 1 inch = 4 inches).

RESULTS

Tests conducted to December 1968 show that the geometry of the meters is suitable, and that the mechanism is sufficiently robust. Insufficient results have been obtained so far to define adequately the correlation between silt loads and turbidity or between turbidity and photocell resistance. However, results indicate considerable short period fluctuations ("cloudiness" or "streakiness") in turbidity, and a portable turbidity meter is now being constructed to investigate this effect. This meter consists of an electronic section similar to the turbidity meter attached to fins similar to the sediment sampler. Such a meter will also find application in coastal engineering if towed from a boat.

ACKNOWLEDGEMENTS

Financial assistance has been provided by the Umgeni Foundation and the South African Council for Scientific and Industrial Research, and the Durban City Engineer has kindly made transport available. The Pietermaritzburg City Engineer has provided facilities at Henley Dam, and the Chief Engineer of Hulett's Mill, at Natal Estates.

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ACCURATE WAVE MEASUREMENT IN THE PRESENCE OF REFLECTIONS*

BY W. JAMES, Ph.D., M.I.C.E.**

CONTENTS

SYNOPSIS

THE PAPER DESCRIBES a few simple techniques for obtaining wave measurments accurate to the nearest 0·005 in. (0·1 mm.). These include an assessment of errors involved in using thin needles mounted on micrometers, and corrections for the errors inherent in wave tanks that are not completely decoupled. It is pointed out that both the reflections and the trochoidal nature of real waves often lead to large errors using conventional methods.

INTRODUCTION

In scale model studies of harbours and coasts, waves are usually of unique period, and measurements are often recorded at a single stationary point. Similar measurements are frequently carried out in wave flumes, e.g. in tests on breakwaters. Unfortunately, the potential errors involved are not always clearly understood. In fact, effective boundary absorption is seldom attained, and a reflected wave of amplitude 20 per cent that of the original incident wave is quite common. This could, of course, give rise to an error of the same order, since single probes merely record the envelope height at that point. (Hence such results are more meaningful if quoted to the same accuracy as boundary reflectivity, which should therefore be measured whenever possible.)

When determining the amplitudes of the incident waves in the presence of reflected waves, it is wise to use two or three probes together. Where there is relatively low boundary reflectivity, and where the wave amplitudes are small enough for the waves to be effectively sinusoidal in shape, the average of any two envelope heights E_1 and E_3, measured one-quarter of a wavelength apart, gives a reasonably good estimate of the incident amplitude. In a laboratory wave flume the incident and reflected waves are colinear (this superposition of a reflected component on the incident wave is commonly referred to as *partial clapotis*), and the quarter wavelength is measured along the direction of propagation. For non-colinear partial clapotis,

* The MS. of this paper, in its present form, as accepted for publication, was received at the Institution on 24th March 1969.

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the distance is again measured along the direction of propagation of either component, but the probes should be located on the bisector of the angle included between the advancing wavefronts. The errors incurred by this method, when the reflected component is large, are tabulated below.

Reflectivity, on the other hand, is usually measured by using either two or three stationary probes in concert, as described below. The accuracy to which the measurements of reflectivity are to be calculated should be considered before designing the experiment, since large errors are inherent in the trochoidal shape of many real waves. Other preliminary considerations include the size of the experimental facility and the measuring technique to be adopted. Long flumes and large tanks are necessary to reduce multiple reflections sufficiently, and in this connexion the author has found the National Research Council absorber (Hamill 1963)*, and an adaption of the "NEYRPIC" filter (Biesel 1948) using nylon gauze, to be very satisfactory.

Short waves are usually controlled easily, apart from the difficulties that occasionally arise when measurement accuracy is commensurate with viscous attenuation. There is, then, because of the difficulty of decoupling, an effective limit to the maximum wavelength that can be treated accurately. On the other hand, if reflectivity is to be determined to an accuracy of, say, 1 per cent, it is clear that envelope heights should be determined to an accuracy of at least 0·5 per cent. Thus, the amplitudes of the waves generated should be such that the wave heights are about 200 $\times$ recording accuracy.

Consequently, the experimental requirements set limits upon the maximum wavelength and minimum wave height, and it follows that laboratory waves capable of accurate measurement may be significantly trochoidal. Most methods currently used to analyse wave measurements are based upon an assumption of sinusoidal wave shape and, by an unfortunate twist of fate, this gives rise to serious errors in some cases (particularly the loop and node method described below).

In this paper, an attempt is made to describe (*a*) a fairly accurate method of measuring envelope heights, (*b*) an accurate method of measuring the incident wave amplitude when reflections are present, and (*c*) the errors inherent in the resolution of real partial clapotis into the original and reflected component waves.

MEASUREMENT

Micrometer Arrangement

As described above, to accord with the requirement of a sinusoidal wave shape, wave heights and hence measurement errors should be as small as possible. Usually, the main source of error in wave measurement is surface tension on the probes, both for electronic and manual recording.

In a recent study carried out by the author, probes were fashioned out of fine stainless steel sewing needles, nominal diameter about

* References are given on p. 501.

0·015 in. (0·3 mm.), and coated with a thin layer of non-wetting paint to limit the column of water draining down off the probe as it emerged from the crest. These probes were attached, through interchangeable alloy tube extensions, to micrometer heads capable of being read to 0·0001 in. More recently two probes have been arranged on finely threaded rods such that a 3 in. (76·2 mm.) diameter dial guage measures the wave height directly to the nearest 0·0005 in. (0·01 mm.). The probe should be connected to a cathode ray oscilloscope.

Stray electrical currents in the water were used to give an instantaneous signal on the oscilloscope whenever probe contact was made or broken. Provided that surface disturbances arising from menisci inversion at the walls, or other similar effects, are negligible, and that there is no seiche action in the flume and that a miniature motor is attached to the micrometer head, this system is extremely straightforward and simple.

Errors

Using the micrometers at a fixed location, readings of both crest height and trough height are noted, and the difference gives the envelope at that point. By far the biggest source of error is the sum of:—

(*a*) The depth to which the crest is actually depressed before a signal is obtained, and

(*b*) The height to which the surface of the centre of the trough is raised before contact is broken.

The total correction can be reduced to approximately 0·007 in. (0·1 mm.). All envelope heights should be increased by an equivalent amount.

INCIDENT WAVE AMPLITUDES

Where only an incident wave and a partially reflected wave constitute the partial clapotis, the envelope heights vary simply between loops (maxima) and nodes (minima). By laboriously scanning the envelope, the loop and nodal height can be measured as described above. Then, even for fairly steep waves, the semi-amplitude of the incident wave is given approximately by:—

$$a = \frac{E_L + E_N}{4}$$

where a = semi-amplitude
E_L = loop envelope height
E_N = nodal envelope height

If the waves are approximately sinusoidal, then it is unnecessary to scan for loops and nodes, and the amplitude of the incident wave can be obtained from the average of two envelope heights E_1 and E_3 measured one-quarter wavelength apart, as described earlier. It can be shown that:—

$$\frac{E_1 \cdot E_3}{2a} = \sqrt{(1-\alpha^2)^2 - 16\alpha^2(K^2 - K^4)}$$

where $K = \cos kx$

$\alpha = \dfrac{\text{reflected wave amplitude}}{\text{incident wave amplitude}}$

k = wave number

x = horizontal co-ordinate,

and the first order wave profile is given by:—

$\eta = a \cos (kx - \sigma t)$

where η = vertical co-ordinate based on still water depth,

σ = circular frequency

t = time.

Table I was calculated using these equations, and indicates the error incurred by using this method for various locations of the two probes on the partial clapotis (where "measured amplitude" $= \dfrac{E_1 + E_3}{4}$).

TABLE I. RATIO OF MEASURED TO ACTUAL INCIDENT AMPLITUDE

α	cos kx					
	0·0	0·2	0·4	0·6	0·8	1·0
0·05	1·000	1·000	1·000	1·001	1·001	1·000
0·10	1·000	1·000	1·002	1·004	1·004	1·000
0·20	1·000	1·003	1·010	1·018	1·018	1·000
0·30	1·000	1·007	1·024	1·040	1·040	1·000
0·40	1·000	1·014	1·045	1·071	1·071	1·000
0·60	1·000	1·037	1·106	1·157	1·157	1·000
0·80	1·000	1·086	1·197	1·268	1·268	1·000
1·00	1·000	1·179	1·316	1·400	1·400	1·000

Note that the locations cos $kx = 0$, 1·0 coincide with the loop and node method described above. Either that method or the three-point method described below may be used when the errors listed in the table are unacceptable. (The 3-point method obviates scanning, and has the advantage that reflectivity can also be derived therefrom). However, the loop and node method is preferable when the waves are steep and reflectivity is high, because the errors can be estimated, and the observations corrected accordingly.

REFLECTIVITY

The same two methods, i.e. the loop and node method already mentioned, and the three-point method, are suitable for calculating the reflected wave amplitude.

In the loop and node method, the measured reflectivity is

$$\alpha = \frac{E_L - E_N}{E_L + E_N}$$

This is the calculation that can be misleading if both the incident and reflected waves are very steep (implying almost complete clapotis), since the errors could approach 20 per cent. But these errors are calculable, and can be estimated by computing the actual superimposed envelope of the two trochoidal waves for given reflectivities, and plotting the results. Reflectivities measured in the laboratory according to the above equation can then be converted to actual reflectivity.

In the three-point method (Brebner and White 1965), three envelope heights E_1, E_2, and E_3 are measured at any points one-eighth wavelength apart. Then:—

$$a^2 = \tfrac{1}{4}\{E_1^2 + E_3^2 + \sqrt{4E_2^2(E_1^2 - E_2^2 + E_3^2) - (E_1^2 - E_3^2)^2}$$

and

$$\alpha^2 = \frac{E_1^2 + E_3^2}{8a^2} - 1$$

Clearly, this method gives results that depend upon the actual location of the probes in the envelope, if the waves are significantly trochoidal. According to calculations performed by the author, a practical limit would be: wave steepness 0·01, and reflectivity 0·5, beyond which the method becomes inaccurate.

CONCLUSIONS

In measurements of incident and reflected wave heights, care should be taken to ascertain whether the experimental waves are likely to be significantly trochoidal, or approximately sinusoidal.

If the latter, then wave amplitudes can be measured reasonably accurately by averaging two measurements taken one-quarter wavelength apart, for small reflectivity. If the reflectivity is also required to be measured, then the three-point method should be used.

But when the waves are trochoidal, e.g. reflectivity above 0·5 and an incident wave steepness above 0·01, the loop and node method should be used in conjunction with suitable curves. This method is tedious, unfortunately, despite the fact that the calculations are extremely simple.

Thin stainless steel needles attached to dial gauges and connected to an oscilloscope allow a measurement accuracy of about 0·005 in. (0·1 mm.), provided that the experimental arrangement is sufficiently well designed. Using such equipment, and the above methods, extremely accurate results can be obtained.

ACKNOWLEDGMENTS

The author acknowledges the kind help and encouragement received from Professor Jack Allen of Aberdeen, and financial aid from the C.S.I.R. in South Africa.

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F:Publsta

APPENDIX

COMPLETE LIST OF PUBLICATIONS

The numbering system used is different from that used in the body of the dissertation.

P. PAPERS AND ARTICLES IN SCIENTIFIC AND TECHNICAL JOURNALS

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MUNICIPAL WASTEWATER CHARACTERIZATION

Application of denitrification batch tests

by

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(MSc.)

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Abstract

The biological treatment of wastewater has evolved significantly from simple single sludge systems practicing organic carbon removal to ones which now include either nitrification/denitrification (N/DN) and / or phosphorus (P) removal. The inclusion of more biological processes have increased the complexity of current wastewater systems which has subsequently led to the development of more complex mathematical models. The operation of plants can be assessed and improved by the use of mathematical modelling tools which require accurate input data. Thus, knowledge of the wastewater characteristics is an important step towards the optimum modelling, design and operation of present and future plants. However, for these tools to be effective, the input data needs to be accurate which is dependent on the current methods used to determine them.

Wastewater is a complex substrate consisting of compounds of differing biodegradability. Biokinetically, these compounds have been divided into readily biodegradable (RBCOD), slowly biodegradable (SBCOD) and unbiodegradable substrate groups. Compounds with intermediate biodegradability i.e. compounds which fall between the RBCOD and SBCOD groups, have been termed readily hydrolyzable organic substrates (RHCOD). The organic matter is discussed in terms of chemical oxygen demand (COD). The readily biodegradable and readily hydrolyzable COD fractions of wastewater can be determined by respirometric tests such as the oxygen utilization rate (OUR) and nitrate-N utilization rate (NUR) tests.

The principal aim of this project was to investigate the NUR test as a tool for wastewater characterization and to study denitrification kinetics in batch reactors. In addition, an experimental readily biodegradable substrate, acetate, was used to determine the reliability of the NUR tests. Acetate was also used to ascertain utilization profiles and rates of a typical readily biodegradable substrate during denitrification. Biodegradable COD characterizations with enhanced biological phosphorus removal (EBPR) sludges were also investigated to determine the impact of anoxic phosphorus removal on NUR tests. The results obtained from the numerous NUR tests added to the undestanding of the NUR test.

Samples from 22 wastewater treatment plants were tested, most of which were located in France. Four South African plants were also tested. Data obtained from the NUR tests were used to calculate the RBCOD and RHCOD fractions. The SBCOD, however, could not be determined directly from the 6 h NUR batch tests. The readily biodegradable COD (RBCOD) fractions ranged between 7 and 25 % of the total COD concentration of raw wastewater, with majority of those results falling within the 10-20 % (of the total COD) range. The results also showed that the initial rapid rate associated with readily biodegradable COD utilization was sometimes followed by a short intermediate phase (*i.e.* short duration, 2 to 3 h). The intermediate fraction was found to range between 5 and 29 % of the total COD

concentration and was classed as a readily hydrolyzable COD component of raw wastewater since the magnitude of the RHCOD fraction was too small to be classed as slowly biodegradable COD which comprises approximately 30 to 60 % of the total COD found in raw wastewaters. The variability of the RHCOD fractions suggests that this fraction is either very variable or that the NUR test does adequately or accurately characterize it. Another possibility is that the RHCOD (or second biodegradable fraction) calculated from the NUR test is a component of the RBCOD of the influent wastewater. In this case, the bacteria may have used some of the RBCOD directly for energy and accumulated or stored the rest as part of a survival mechanism which allows them to be more competitive under dynamic operating conditions. Once the readily biodegradable COD becomes limiting, the bacteria will use the accumulated or stored compounds. This hypothesis is substantiated by tests done with acetate as substrate.

An intermediate phase was also observed when acetate was the sole substrate. Thus, it was possible with the 3-phase profiles to calculate a second biodegradable fraction. Results suggest that a significant part of the added acetate (as COD) was stored and the second phase is in fact an 'apparent or residual' phase brought about by the consumption of the stored or accumulated acetate products. This is suggested in two ways : (1) the calculation of the yield coefficient is lower and closer to the 0.5 mg/l values, cited in the literature, when the COD calculated from phases 1 and 2 are considered, and (2) the acetate mass balances were found to be approximately 100 % when phases 1 and 2 were used to calculate the amount of acetate utilized under anoxic conditions.

The results obtained with sodium acetate as a readily biodegradable substrate were used to formulate several conclusions on acetate utilization during denitrification. Firstly, from acetate mass balances it was found that acetate may be used exclusively for denitrification (100 % acetate was accounted for). In this case, the sludge contains a significant proportion of denitrifiers and little or no polyphosphate accumulating organisms. This observation was made only when non-EBPR (enhanced biological phosphorus removal) sludges were used. Secondly, acetate mass balances which were found to be < 100 % suggest that acetate could be used for denitrification and the production of storage products like polyhydroxyalkanoates (PHA's). These sludges probably contained a higher proportion of polyphosphate accumulating organisms which competed for the available acetate in the bulk liquid. This observation was made for both EBPR and non-EBPR sludges. Thirdly, acetate could be used for denitrification by denitrifiers and for polyhydroxyalkanoate synthesis by denitrifying polyphosphate accumulating organisms. The stored PHA's in the denitrifying polyphosphate accumulating organisms are subsequently utilized during denitrification. This secondary utilization is manifested in the second denitrification phase and is supported by the observation of phosphorus uptake. These results showed that wastewaters high in volatile fatty acids (VFA's) were also subject to denitrifying polyphosphate accumulating organism activity even though the sludge was sampled from non enhanced biological phosphorus removal systems (non EBPR).

Several of the NOx profiles revealed either 2 or 3 rates due to the control of the substrate to biomass ratio (S/X : ≤ 0.1 mgO_2 / mgO_2). Majority of the samples (i.e. 85%) tested produced initial maximum specific denitrification rates (k_1) between 3 and 6 mgN/gVSS.h. The intermediate denitrification rate (k_2) was found to vary between 2 and 3 mgN/gVSS.h. Denitrification rates (k_3) obtained from utilization of influent and endogenous slowly biodegradable COD (SBCOD) varied between 1.0 and 1.5 mgN/gVSS.h. This latter rate is significantly higher than the endogenous denitrification rates cited in the literature. One of the reasons for these higher rates could be be linked to the the reuse of stored or accumulated products by the microorganisms.

In addition, a comparative study on RBCOD determination of wastewaters with enhanced biological phosphorus removal and non-EBPR sludges. It was found that the RBCOD values derived by NUR tests with EBPR sludge were consistently lower (4 to 5 %) than those with non-EBPR sludge. Thus, the NUR tests with EBPR sludge resulted in a 4 to 5 % underestimation of the RBCOD fraction of raw wastewaters. This loss in RBCOD to polyphosphate accumulating organisms appears to be linked to the influent raw wastewater acetate concentration.

These tests showed that the RBCOD fraction could be adequately characterized using the NUR method. The accuracy of the tests appears to be compromised when enhanced biological phosphorus removal sludges are used in the NUR tests. Moreover, it was found that non-EBPR sludges can also consume some of the acetate that is present in the system for the production and replenishment of storage compounds. Fortunately, for the wastewaters tested, the acetate component of the RBCOD fraction was small and therefore, did not significantly affect the results. Mechanisms such as substrate accumulation and storage may also impact on substrate removal and hence, the determination of the readily biodegradable COD concentration of municipal wastewaters. Thus, while the results showed that the NUR is a useful characterization tool for wastewaters, it will continue to be a more tedious characterization tool than the oxygen utilization rate test, until a suitable nitrate/nitrite electrode is developed to automate the test.

Dedication

'Success is something in your heart,

its an inner strength thats born

from the ability to carry on,

to finish everything you start,

to push towards your ultimate goal.'

..........Charlotte

This dissertation is dedicated to my family and friends for their unwavering support, understanding, and belief.

Preface

I hereby declare that this dissertation is my own work, unless stated to the contrary in the text, and that it has not been submitted for a degree to any other University or Institution.

Valerie Naidoo

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Abbreviations

ATP	adenosine triphosphate
BNR	biological nutrient removal
BEPR	biologically enriched phosphorus removal
Bio-P	biological phosphorus removing sludge
COD	chemical oxygen demand (mgO_2/l)
CV	coefficient of variation
DO	dissolved oxygen
DPAO	denitrifying poly-accumulating organism
FAD	flavin adenine dinucleotide
$FADH_2$	flavin adenine dinucleotide (reduced)
GAM	glycogen accumulating metabolism
g	gaseous
h	hour
K	Kelvin
k_1	first rate (mgN/gVSS.h)
k_2	second rate (mgN/gVSS.h)
k_3	third rate (mgN/gVSS.h)
min	minute
mV	millivolts
µm	micrometer
N	Nitrogen
N/ND	Nitrification/denitrification
N/A	not applicable
n.d.	not determined
n.o.	not observable
NOx	nitrates and nitrites as N (mg/l)
NUR	Nitrate utilization rate
OUR	Oxygen utilization rate
P	Phosphorus (mg/l)
PAO	poly-accumulating organism
PAM	poly-accumulating metabolism
p.e.	population equivalents
PHB	polyhydroxybutyrate
PHA	polyhydroxyalkanoate
RBCOD	readily biodegradable COD (mg/l)
RHCOD	readily hydrolyzable COD (mg/l)
rw	raw wastewater
S	substrate
Ss	truly soluble fraction

SBCOD	slowly biodegradable COD
S-ce	COD of supernatant after centrifugation
S-$f_{0.45}$	COD of supernatant after 0.45 µm filtration
S-co	COD fraction after coagulation
S-p	particulate COD (after filtration)
S-t	total COD
S-uns.	COD of supernatant after 2h settling
T	temperature in degrees Celsius
t	time
TSS	total suspended solids
V	volume
Vt	total volume
Vww	volume of raw wastewater
Vx	volume of sludge
VFA	volatile fatty acid
VSS	volatile suspended solids
WTP	wastewater treatment plant
Xt	biomass
Xf	filtered fraction of biomass
Y_{HD}	yield coefficient (anoxic)
Y_H	yield coefficient (aerobic)