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ASSOCIATES LIMITED

HYDROLOGY
WATER RESOURCES
REMOTE SENSING

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WATERLOO, ONTARIO
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HYDROMETRIC NETWORK DESIGN

A state of the art report
prepared for

Environment Canada

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PREFACE

This report was prepared by Professor S.I. Solomon as a follow-up to a seminar on the state-of-the-art in hydrometric network design presented on April 12, 1983 for the staff of the Hydrology Division, Water Resources Branch, Inland Waters Directorate, at Place Vincent Massey Building, Hull, Quebec.

The report reviews and discusses the principles and techniques of hydrometric network design based on Canadian experience and information from international literature.

Figures 6.2(a), 6.2(b) and 6.3 mentioned in Professor Solomon's report have been omitted because of difficulties in reproduction.

W.Q. Chin
Chief
Hydrology Division

CONTENTS

1. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

1.1 Summary

The design of hydrometric networks has been a significant preoccupation of hydrologic organizations during the last 15 years. A significant number of hydrometric network studies has been carried out during this period. Numerous scientific papers have also been published on that subject. However the practical impact of these studies and papers has been minimal. The main reasons for this have been the lack of recognition in some of the studies of the practical mechanisms governing hydrometric network development, unrealistic economic and statistical-hydrologic assumptions used as the basis of some studies and papers, lack of mechanism to pass on to the data users the cost of data collection, and inertial resistance by some governmental organizations in adopting new concepts in designing the hydrometric network.

At this point in time, the studies carried out so far indicate that hydrometric networks could be separated into two main sub-networks: the inventory or general assessment section and the planning, design, and operation section.

The first section can be designed on the basis of accuracy goals or budget constraints. In such network the objective is to obtain hydrologic information at a minimum cost, or maximum accuracy for a given budget. Techniques to achieve these objectives are available and their application should be of main concern to Environment Canada.

The second section could be designed in principle using economic criteria. However, for each station under consideration the calculations involved are too complicated and costly and the results of dubious practical value. Furthermore, many of the stations included in such network are required for technological purposes and therefore not subject to design. Such stations should be installed and operated at the user's cost as this will tend to reduce if not eliminate the installation and operation of unnecessary stations. One of the users is in many cases Environment Canada itself (e.g. monitoring stations). However, this should not detract from the application of the user's charges. Separation of the budget for inventory purposes from that of monitoring and similar stations could also enhance the efficiency of the network as a whole.

Introduction of the advanced measurement techniques and data collection systems including telecommunication via satellites and the use of remotely sensed data as inputs into models for information transfer could provide further elements for increasing the efficiency of the hydrometric network.

1.2 Conclusions and Recommendations

The hydrometric network can be subdivided into two main sections. The first is the inventory network which should be the concern of the Federal Government. The second is the planning, design, and operation network, which is related to the use of water resources and should be the concern of governmental and private organizations having jurisdiction over or owning or using the water resources of the area. As the Federal Government has also jurisdiction over some water resources, it is also directly interested to a certain extent in the second network.

The design of the inventory network, which should be able to provide information for assessing water resources anywhere in the country could be based either on a target accuracy of this information, or on budgetary constraints. In the first case the objective is obtaining the target accuracy at least cost. In the second case the objective is to obtain the maximum accuracy for the given budget. In all cases of design of the inventory network the technique of information transfer plays a very important role. Whatever the technique (model) of information transfer used, it is important to introduce in it as much as possible of the available auxiliary data, primarily data on terrain characteristics. The use in such technique of precipitation as an independent variable is not recommended, since such variable is affected, particularly in Canada, by large errors. The use of drainage area as an independent variable in such technique, when flow is the dependent variable may lead to spurious relations.

Relationships between error of estimate and density of stations using the square grid technique and other techniques, indicate that at present this technique provides the smallest error of estimate for a given density. The relationship appears to hold in various climates and areas of the world, and could be used to plan the inventory network.

The time sampling error has not been yet investigated in sufficient depth in studies based on the square grid technique. This has been investigated in the case of information transfer techniques based on multiple regressions with conventional inputs of the independent data. There is a need to extend the analysis of the time sampling error in the case when the square grid technique is used for estimating the independent variables, or for joint mapping of precipitations, evaporation and runoff.

Remotely sensed data can provide additional inputs into models of information transfer, or provide updating techniques for such inputs and should be used whenever practical for both these purposes.

The cost-benefit analysis of hydrometric stations of the planning, design, and operation network has been based on the assumption that the maximum benefit is obtained by operating a station to the point at which marginal cost is equal to maximum benefit. This is based on the assumption that the budget for operation is not constrained. This assumption is not realistic in most practical cases.

The Bayesian approach to the design of the planning-design-operation stations, while justified from a purely statistical decision analysis viewpoint, can not be applied in practice because it requires uneconomically lengthy computations. When applied with a number of simplifications required to overcome computational complexity it leads to meaningless results. The Bayesian approach can not be therefore recommended for practical applications.

A large number of planning-design-operation stations are required for purely technological purposes. Monitoring and water allocation stations are required also by the Federal Government in the framework of its jurisdictional responsibilities.

Planning-design-operation stations should be installed and operated at the request of the government and other organizations that have jurisdiction over and/or use the respective water resources and should be paid for by these organizations. This will reduce to a minimum the overgauging tendency. The Federal Government should separate the budget of the inventory network from the budget of the operational networks.

New technology of network operation, in particular new techniques of telecommunication and flow measurement should be introduced as required to increase the overall efficiency of the network. Care should be exercised to prevent that the introduction of new technology reduces the network economic efficiency instead of increasing it.

2. INTRODUCTION

2.1 Background

During the last 20 years or so the design of hydrometric networks has been a major preoccupation of a large number of international, governmental and private organizations, and of hydrologists all over the world. Starting with the Quebec IASH Symposium on Design of Hydrological Networks (IASH, 1965) and ending with the recent WMO Workshop on cost/benefit assessment of hydrologic stations (WMO, 1982) several international meetings, studies sponsored by international organizations and national governments, and a large series of reports, theses and papers have been devoted to this subject. A number of the related publications are listed in the References.

In spite of this intense activity, the impact on practical design of hydrometric networks has been minimal. There are several reasons for this. The main reason resides with the way hydrometric networks have developed historically. Under ideal circumstances a hydrometeorologic network should evolve through the gradual implementation of an inventory network, followed by a planning and design network, and an operational and monitoring network*, which includes also forecasting elements (Figure 2.1a). However, due to the actual historical development, in Canada the network was initiated by the establishment of a design, monitoring, and forecasting network, around which it is necessary to develop the other network levels (Figure 2.1b). Such conditions are not particular to Canada. Similar conditions have been observed in most countries of the world. Other reasons include: the use in the analysis of the worth of data of economic principles which are rarely valid in practice, the application of unduly complicated computational techniques to estimate the value of hydrologic data, and administrative inertia.

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* These three networks correspond basically to the three network levels delineated in Rodda et al (1969).

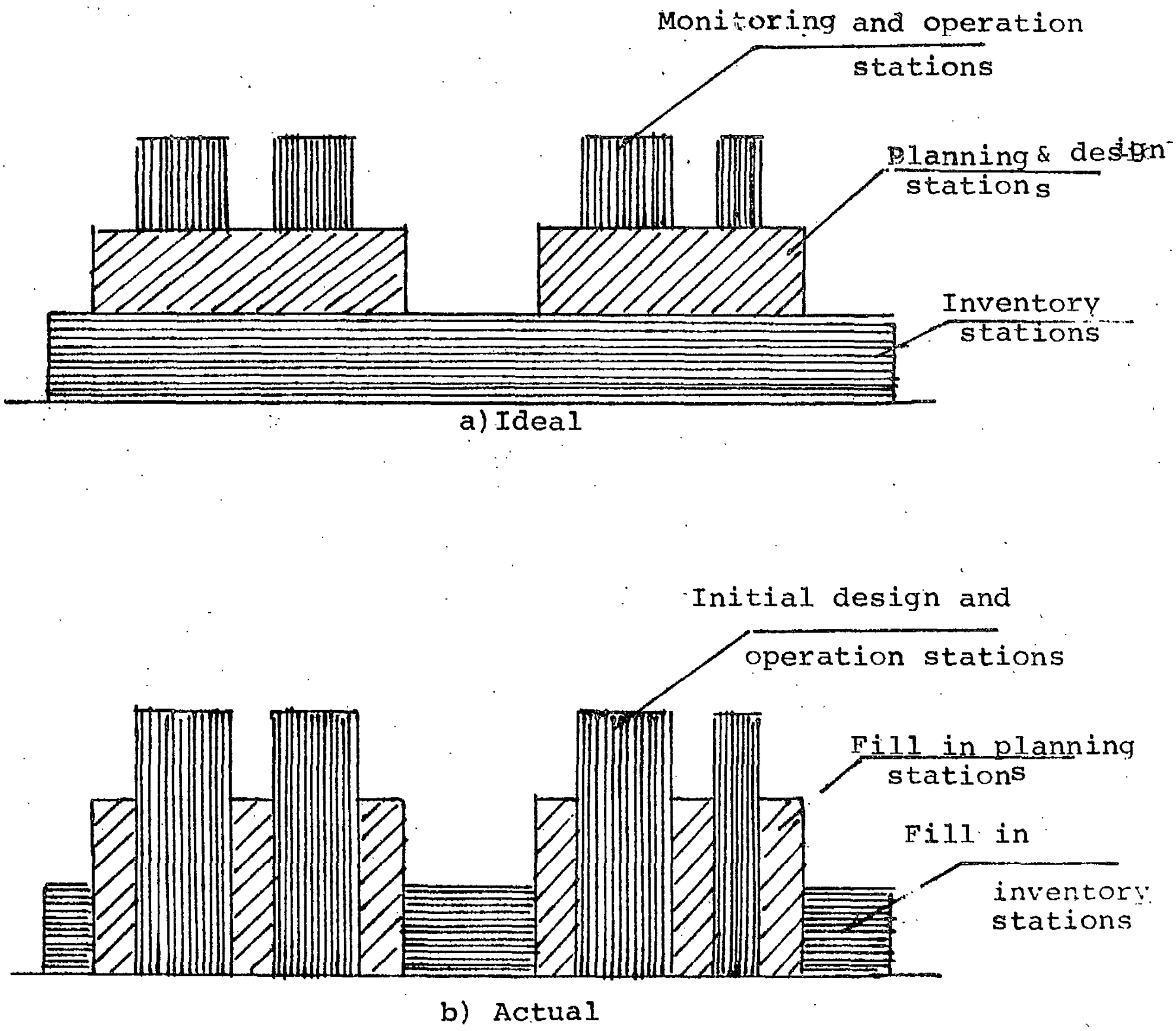

Figure 2.1 Ideal and actual development of hydrometeoro-
logical networks

One of the economic principles frequently applied in studies of hydrometric network design is the one stating that economic return from the operation of a station is obtained when marginal cost is equal to marginal benefit. Such principle is correct only when it may be assumed that the budget for operating the network is unlimited. As discussed in Chapter 5, when the budget is limited the optimum is usually reached at lower marginal costs (and higher marginal benefits). Several studies have used the Bayesian approach in decision theory to analyze the economic worth of hydrologic data and provide a basis for decision regarding the operation or discontinuation of hydrometric design stations (Davis and Dvoranchik, 1970, Attanasi and Karlinger, 1977, Moss and Dawdy, 1980). The results of these studies indicate that the technique does not provide a practical tool for such analysis, because of its computational complexity. Simplifying assumptions required to reduce this complexity to a manageable level, also reduce much the practical significance of the results. Because of this the Bayesian approach has not gained practical acceptance in Canada.

Environment Canada and its predecessors have also carried out a number of studies in network design (Shawinigan 1969, 1970, Ingledow 1970, Solomon 1974, 1974a). The recommendations of these studies have been applied in practice only to a limited extent, except in British Columbia, where they have been followed more extensively (Kreuder, 1979). In the last few years Environment Canada has been involved in studies for the design of the hydrometric network of the Mackenzie River basin (Mackenzie River Basin Commission, 1981). More recently Environment Canada has requested Shawinigan Consultants Inc. to carry out a study of the hydrometric network of the Northwest Territories (Shawinigan, 1982).

Environment Canada is currently reconsidering the recommendations made by consultants in the studies of hydrometric network planning. In addition Environment Canada has undertaken work on the consolidation of the square grid data bank that has been developed in the course of the hydrometric network studies, with the view that it could possibly be used in the practical design of the consolidation of the hydrometric network of Canada. In connection with these activities, Environment Canada has considered appropriate to carry out a state of the art review on hydrometric network design. This task has been entrusted to Professor S. I. Solomon. As requested by the terms of Reference of the job, a seminar on hydrometric network design was organized in Ottawa in the offices of Environment Canada. This Report represents the state of the art review on hydrometric network design, which was presented orally and discussed at the above mentioned seminar.

2.2 Purpose

The purpose of this Report is to present a review of the techniques available for the design of hydrometric networks, with emphasis on Canadian experience and on techniques that appear to be more adequate for application in Canada. The objective of the review is to prepare the ground for the updating and implementation of the recommendations of the studies on hydrometric network planning that have been carried out during the last 15 years, taking into account the recent technical developments that affect directly or indirectly network design.

2.3 Scope

The scope of this Report is limited to a review of the state of the art in hydrometric network design based on techniques of network planning contained in the Canadian studies or the available international literature. The scope of this Report is limited to the discussion of principles of the techniques reviewed. The analytical basis of the techniques is neither presented nor discussed. Furthermore, the scope of this Report is limited to surface water resources.

3. NETWORK CLASSIFICATION

3.1 General

Hydrometric stations can be classified in many ways (Rodda et al, 1969, Solomon, 1970, WMO, 1979). Network design could benefit from a simplified classification according to the purpose of the station. From this viewpoint hydrometric stations can be classified according to the following main purposes, which relate basically to the corresponding stages of water resources development:

- inventory (assessment of the resource availability)
- planning
- design
- operation and monitoring
- forecasting

3.2 Inventory Stations

Hydrometric stations installed for the purpose of inventorying the water resources of a country (region) are normally related to a wider national objective, that of taking stock of the available natural resources of the area. By their very nature they are installed usually in undeveloped areas* and therefore the hydrologic regime they measure is relatively stationary and the basins are in a natural (or at least stabilized) state. For such stations statistical processing does not present particular difficulties and information transfer using well established techniques is possible.

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* In some cases water resources inventory is being attempted after development has already been initiated. The hydrologic interpretation of data is different in such situations, but the network design principles are not different from these to be applied in undeveloped areas.

For such type of stations the normal network design strategy calls for establishing a number of benchmark stations to be operated indefinitely and thus explore the temporal variation of the resource* and a number of short-term (5 - 10 years) stations which can be located in accordance to the variation of the geophysical conditions that cause a variation in hydrology, and which provide information on the space variation of the resource. For such stations it is not possible to apply cost/benefit analysis as benefits from such data are shared by a large number of users. The only objective criterion to judge the network adequacy is the error of estimate of various hydrologic characteristics at an ungauged site. This in turn depends upon the error of measurement, the error of sampling, and the error of interpolation. When funds are allocated for such stations it is necessary to investigate the possible reduction of the error from each source versus the corresponding cost, and allocate funds available in such a way as to maximize the error reduction. Conversely, if a target error is set, the minimum amount of funds that are required to reach it can be estimated, as well as the corresponding allocation between activities aimed to reduce error of measurement, of time sampling, and of interpolation.

3.3 Planning Stations

The information obtained from inventory stations may be occasionally adequate for planning purposes. However, when the information is not sufficient, the network has to be strengthened in the area of interest and this usually involves, among others, installing a number of planning stations,

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* To ensure that the hydrologic regime remains undisturbed, it is advisable to locate benchmark stations in conservation areas, wild life reserves, national parks and similar protected areas.

i.e. stations whose main objective is to estimate the characteristics of water bodies considered for development within the planning time horizon. In such cases, it may be necessary to postpone planning decisions until data collected at the new stations provide the information required for making the decisions. It should be noted that in many cases planning stations are required because the inventory stations do not provide sufficiently accurate information on all the water bodies characteristics required for planning purposes. In some cases additional information is required because of non-stationarity of the hydrologic regime due to man's activities.

The types of hydrologic data required for planning water-resource projects vary according to the characteristics of the project and of the water resource involved. Depending on the nature of the project, the planning of such a project may require any combination of the following data and information on the relevant surface water resources:

Daily and/or instantaneous maximum and minimum levels of rivers and lakes (in addition hydrographs of flood levels for various probabilities);

Daily concentrations and/or total sediment discharge. Water quality determinants (in addition relationship between flow and water quality);

River channel characteristics: variation of area, width, and depth with level (flow), water velocity, slope, etc.;

Possible and actual changes in all the above types of data due to man's activity (reservoirs and other hydraulic structures, changes in land use, land cover, etc.).

Although the optimum level of accuracy can be determined by a detailed cost/benefit analysis as shown in 5.2 and 5.3, an indication on the adequacy of available data can be obtained initially by simpler techniques such as the one described in WMO (1975), pp. 111-117, and using typical analysis such as that carried out by Klemes (1977).

3.4 Design Stations

Hydrologic information required for design purposes is usually more limited in scope than that required for planning. However, such information is usually required at very specific locations. In some cases accuracy requirements are specified by the technologic design and therefore not subject to economic analysis. Of course, the critical accuracy percentage level depends upon the type of project element, and the ratio between the estimated mean value of the characteristics and the corresponding dimension of the project. Thus for example if a 1 m^3/s diversion has to be made from a river whose minimum flow with a probability level acceptable to the designer is 2000 m^3/s, an error of even 90% in estimating the minimum flow is not significant. However, if a hydroelectric plant is to be built on a river with a fully regulated flow of, say, 100 m^3/s and the margin of financial return is only 15% per year, an error of 10-15% cannot be tolerated in estimating this mean flow. Similarly, if a spillway of a concrete dam is designed for a given flow, say, 10,000 m^3/s, and can be surcharged without significant damage up to 12,000 m^3/s a standard error of 10-15% in the maximum flow estimation would be considered perfectly acceptable. However, if the dam is an earth one and would be overtopped at 12,500 m^3/s, the same range of error may not be considered tolerable.

The value of hydrologic information used for design purposes is occasionally amenable to economic analysis in a similar manner as that for planning purposes. However, in the usual case, it is not possible to postpone design until the additional information corresponding to an optimum has been obtained. Consequently, it is necessary in most cases to anticipate during the preliminary stages the information requirements and the corresponding desirable accuracies. When design is involved, practical experience shows that in the majority of cases improved accuracy of information has a value exceeding many times its cost, and therefore installing a gauging network geared towards the design requirements usually is economically beneficial.

Unesco/WMO (1981) provides indications on the types of data required for various types of water-resource projects, significance of the hydrologic information, and the limits of tolerable errors as indicated by current practice. According to Unesco/WMO, water-resource projects can be considered to consist of a number of typical project elements which, when grouped in accordance with specific local requirements and objectives, generate the given project. The types of water-resource data required by project elements are easier to define than for the whole project. The project elements can be grouped into structural and non-structural elements.

The main structural elements are:

(a) Modifiers of the water balance components (increasing or decreasing runoff, precipitation, evaporation, soil moisture, through surface treatment, cloud seeding, etc.);

(b) Redistributors of water in space (water collection, transportation and distribution intakes, canals and pipes, intakes, outlets);

(c) Redistributors of water in time (surface and sub-surface reservoirs and other water storage structures);

(d) Extractors or suppliers of water energy (turbines and pumps);

(e) Water confiners (dams, dykes, flood walls, flood-proofing structures, etc.);

(f) Water relievers (spilling facilities);

(g) Quality improvers at source (reducing soil erosion, salinisation, etc.);

(h) Quality improvers at a use point (water supply and sewage treatment plants, cooling towers and lagoons, etc.).

The main non-structural elements are:

(i) Water-resources-related legislation and standards;

(j) Zoning (for flood management, preservation of water resources, runoff and soil erosion management, wild life and fish habitat protection, etc.);

(k) Insurance (in relation to flooding of permanent or temporary structures (cofferdams), non-performance, accidents and disasters);

(l) Flow and level forecasting (flood warning, operation of reservoirs, etc.);

(m) Population relocation.

Although occasionally information on all the characteristics of the pertinent water resource is of significance for a given water resource project element, a number of characteristics are normally extremely important for it, others are only moderately important and others are of no interest at all. For example, for water relievers information on maximum flow is of extreme importance, information on sediment characteristics (which may block gates) is of moderate importance, and information on minimum flows is not relevant at all. Although the significance of each water-resource characteristic for a given element may vary with local conditions (and may also vary in time) it is possible to indicate which characteristics are normally extremely and moderately important for the various elements of water-resource projects (Table 3.1 from Unesco/WMO, 1981).

Each project and project element has its particular tolerance limits for the error of each pertinent hydrologic characteristic. However, Unesco/WMO (1981) considered it possible to recommend approximate tolerance limits for various projects elements based on experience and the consideration of the inherent error characteristics of the data. These approximate tolerance limits are indicated in Table 3.2 .

The values shown in Table 3.2 could be used to set average adequacy targets for meteorologic and hydrologic information for design purposes but should not be construed as binding for specific individual projects and/or particular hydrologic characteristics at individual gauging stations. In fact, it is strongly recommended that whenever it can be demonstrated by cost/benefit analysis or typical studies of the type discussed in Chapter 5 that higher error tolerances are acceptable, these should be used.

3.5 Operation and Monitoring Stations

3.5.1 Operation stations

Operation hydrologic stations are stations that measure hydrologic characteristics of water bodies related to a water-resource project and whose data are used in making operational

Table 3.1

- SIGNIFICANCE OF VARIOUS WATER-RESOURCES CHARACTERISTICS FOR THE PLANNING OF VARIOUS ELEMENTS OF WATER-RESOURCES PROJECTS (V - VERY IMPORTANT, M - MODERATELY IMPORTANT)

Water-resources characteristics / Water-resources project element	Precipitation				Evaporation	River water levels			River flow				Channel chart			Sediment	Groundwater			
	Storms	Time Series	Snow	Quality		Time Series	Max.	Min.	Time Series	Max.	Min.	Quality	Cross section	Plane	Velo. distr.		Levels	Yield	Hydr. char.	Quality
a) Modifiers of water balance	V	V	V		V				V	M	M						V	V	M	
b) Redistributors of water in space					M	V	V	V	V	V	V	V	V	V	V	V	V*	V*	V*	V*
c) Redistributors of water in time	M	V	V	V	V	M	M	M	V	M	V	V				V	V*	V*	V*	V*
d) Extractors or suppliers of water energy				M		V	M	V	V	M	V	M	M		M	V	V*	V*	V*	V*
e) Water confiners				M		V	V	V	M	V		M	M	M	M	M				M
f) Water relievers				M		V	V	V	V	V		M	M	M	M	M				M
g) Quality improvers at source	V	V	V	V	V							V	M	M	M	M	V*	V*	V*	V*
h) Quality improvers at use points									V	M	V	V	M	M	M	M	V**	V**	V**	V**
i) Water related legislation and standards	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M
j) Zoning	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M	M
k) Insurance	V	M					V	V	M	V	V	V					V*			
l) Flow and water quality forecasting	V	V	V		M	V	V	V	V	V	V		M		M		V*	V*	V*	

* If project or element deals specifically with groundwater; ** If treated water is disposed into the ground.

Table 3.2

APPROXIMATE TOLERANCE LIMITS OF ERRORS OF WATER-RESOURCES DATA FOR VARIOUS TYPES OF WATER-RESOURCES PROJECTS (per cent, except for water levels for which the tolerance is in cm)

Water-resources characteristics / Water-resources project element	Precipitation				Evaporation	River water levels			River flow				Channel chart			Sediment	Groundwater			
	Storms	Time series	Snow	Quality		Time series	Max.	Min.	Time series	Max.	Min.	Quality	Cross section	Plane	Velo. distr.		Levels	Yield	Hydr. char.	Quality
a) Modifiers of water balance	30	10	40		40				5	15	15						5	10	20	
b) Redistributors of water in space					50	5	10	5	5	10	10	20	5	5	5	20	5	10	20	25
c) Redistributors of water in time	25	10	40	25	30	10	15	10	5	15	20	20				20	10	20	20	25
d) Extractors or suppliers of water energy				25		5	10	5	5	15	10	25	5		5	20	10	20	15	25
e) Water confinors				25		5	10	5	5	10		25	5	5	5	30				30
f) Water relievers				25		5	10	5	5	10		25	5	5	5	20				30
g) Quality improvers at source	35	15	40	20								20	5	5	5	20	10	10	20	30
h) Quality improvers at use points									5	15	10	20	5	5	5	20	10	10	20	20
i) Water related legislation and standards	40	20	50	30	40	10	15	10	10	15	15	25	20	20	20	30	20	20	30	30
j) Zoning	40	20	50	30	40	10	15	10	10	15	15	25	5	5	5	30	20	20	30	30
k) Insurance	25	10					10	5	5	10	10	20					10	20	20	
l) Flow and water quality forecasting	25	10	40	20	30	5	10	5	5	10	10		10		10		10	10	20	

decisions. For example, stations measuring water levels of a reservoir to make decisions on the operation of turbine gates of a hydroelectric plant or of the spillway gates are operation stations. Similarly, stations measuring the concentration of dissolved oxygen (DO) in a storage reservoir and a river downstream that are used to make decisions on the amount of water to be released to maintain the DO concentration downstream within tolerable limits, are operation stations.

In some cases the location equipment, frequency of measurement, and technique of data transmission of operation stations are required technologic components of the given project and as such are not subject to network design. However, in some cases, when the possibility of committing a large error of measurement in gauging the relevant characteristics exists, it is possible to evaluate by cost/benefit analysis if it is worthwhile to increase the measurement accuracy. For example, one single level gauging station on a reservoir presenting a slope of the water surface due to hydraulic or meteorologic conditions may provide erroneous data about the changes in storage. This may lead to erroneous operational decisions. Reporting of negative inflow in several reservoirs in Canada is usually a consequence of such erroneous estimation of change in storage. Similarly, the measurement of the DO concentration in one single point of the reservoir may lead to erroneous operational decisions when water quality is non-homogenous, particularly when there is thermal stratification. In such instances the increase in number of measurement points (sometimes also in frequency of gauging or improved equipment) may lead to improved operational decisions. Assessing the value of the additional stations may be then possible using the techniques described in Chapter 5.

3.5.2 Monitoring

Monitoring stations are gauging stations installed with the purpose of observing and reporting characteristics of water resources affected directly or indirectly by man's activity. Monitoring stations are not leading directly to operational decisions (otherwise they would be categorized as operation stations). However, in time, and through indirect processes, e.g. legislation, they may lead to changes in operational rules.

Most monitoring stations are established on the basis of legal or similar decisions and are usually not subject to network planning decisions. However, as in the case of operation stations, when monitoring stations may be affected by significant errors or their usefulness is in doubt, they could be evaluated from the viewpoint of their usefulness for the purpose for which they were installed. For this, one would have to consider the additional costs of obtaining more accurate information and the additional future benefits (or loss reductions) resulting from changes in operational rules based on the more accurate information.

3.6 Forecasting Stations

Forecasting stations could be considered as a subset of operational stations. Such stations are installed with the purpose of enabling decisions to be made using estimations of water resources characteristics over a certain future period of time. Since through the operational decisions made on the basis of hydrologic forecasts it is possible to generate significant benefits (or losses) and since, within certain limits, it is possible to improve the accuracy and increase the forecast lag, forecasting hydrologic stations can be designed on the basis of economic evaluations.

Several parameters related to forecasting stations (networks) can be varied separately or simultaneously in attempting to evaluate economically forecasting station networks. These parameters are:

(a) number and location of gauging stations,
(b) frequency of measurement,
(c) measurement equipment,
(d) data transmission,
(e) equipment and techniques of data processing,
(f) use of additional measurement techniques such as radar, satellites,
(g) forecasting model.

Data dissemination and the warning and flood damage prevention techniques are also factors in determining the benefits of the forecasting system.

4. INVENTORY NETWORK DESIGN

4.1 Definition

The definition of the hydrometeorologic network as a whole was given in an earlier paper (Solomon, 1971), and is reproduced here for ready reference. A hydrometeorologic network can be defined as an organized system of stations, equipment, techniques, and staff which measures, collects, processes, and interpolates data, and obtains and disseminates information on the time-space variation of all components of the water cycle. Both the quantity and quality aspects of each water cycle component are the object of such network. In the context of the above definition, data are defined as numerical expressions of measurements of some dimension of a natural or man-made object or phenomenom. Information is defined as the result of processing on various ways of time (space) series of data which provides insight on the characteristics of the object or phenomenom and which can be used in decision making. Thus, for example, daily flows over a period of time are data; the mean, standard deviation, and other statistics of these flows represent information.

When only one component of the water cycle, such as river flow, precipitation, or groundwater is the object of the network, it becomes a hydrometric network, a meteorologic network, or a groundwater network respectively. Historically many networks, including those in Canada, have been developed separately, each one for a single component of the water cycle. When co-ordination between such networks is possible, their design, implementation, and operation can follow the same approach as that of a fully integrated hydrometeorologic network. Such co-operation is assumed to be possible in Canada at least of the level of the inventory network as this level of the network is practically all controlled by Environment Canada.

As indicated in section 2.3 this Report is concerned with the hydrometric network only. However, most design aspects of the hydrometric network apply as well to other networks that have as their object various segments of the water cycle.

This Chapter discusses that portion of the hydrometric network that deals with the general inventory of surface inland water resources, which is designated for convenience inventory hydrometric network (IHN). It corresponds to the hydrometric portion of the baseline hydrometeorologic network in Mackenzie River Basin Commission (1981) and Solomon (1981), the Natural regime network in Shawinigan Engineering (1970) and Level I network in Rodda et al (1969).

The IHN should be considered the backbone of the hydrometric network operated by Environment Canada. There are two reasons for this: the first is that traditionally the Federal Government has carried out surveys for inventorying land, mineral and water resources, and it is in the best interest of all levels of governments that it continues to do so. The second relates to the fact that a good IHN provides the necessary basis for the development of the other hydrometric networks, and may - to some extent - even substitute for them. A good comprehensive IHN could be designed and operated most efficiently by the Federal Government, because it has similar characteristics in all provinces and territories, i.e. it is independent of socio-economic conditions, and therefore significant economies of scale are possible.

4.2 Objectives and Characteristics

The objectives and characteristics of a baseline hydrometeorologic network (BHN) have been described in an earlier paper (Solomon, 1981). These are basically similar to those of the IHN except for the comprehensiveness characteristic of the BHN. However, since they are so important for this Report, for convenience we reproduce them here.

4.2.1 Objectives

The main objective of the baseline hydrometeorologic network is to provide an inventory of the water resources available under natural, prior to development, conditions. As such, it provides the basic information for deciding which water resources could be considered in the development process. However, it is neither rational nor possible to require that the baseline network should provide the information required to make decisions for selection between possible alternatives or about the economic feasibility of development projects.

In a rather simplified manner, the baseline hydrometeorologic network can be considered to be similar to a general geologic survey of an area. The latter may indicate the potential mineral resources which should be considered for development but is not sufficient (in the general case) to provide information on which of the potential mineral resources should be actually developed, or what could be the cost and benefit from such development. After the areas of potential resource development have been identified more detailed investigations (corresponding to the level 2 network in the hydrometeorologic field) are normally carried out to provide the required inputs into the decision making process for transforming a potential resource into an actual one. The major difference between a hydrometeorologic baseline network and a general geologic survey relates to the fact that water resources present a space-time variation that requires to be assessed, whereas mineral resources vary, prior to development, only in space. Of course, the above discussion assumes that water resources are not included in the general category of mineral resources.

In addition to the main objective of providing an inventory of a natural resource prior to development, the baseline network should also provide the required data and information for assessing the impact of development on the natural and social environment in general and on water and other renewable resources in particular. Although this second objective may be also pursued by other general geophysical or biologic surveys (such as for example soil and vegetation surveys), because of the exceptional role of water in nature, this second objective is of great significance in the case of the BHN.

An additional objective which is occasionally considered in designing the BHN is obtaining data and information for a better understanding of the processes governing space-time variation of water resources. As will be shown in the next sub-section such understanding is required for achieving the first two objectives discussed above, and therefore considering it as a separate objective may detract occasionally from the rational design and implementation of a BHN.

4.2.2 Characteristics

The discussion of the network objectives leads to the conclusion that the BHN should have certain characteristics essential for fulfilling these objectives. These are:

- Universality: the whole area should be covered without regard to presumed development potential. Such presumptions may be erroneous and development requirements may significantly change in time.

- Comprehensiveness: all water cycle components should be included.
- Interpolability: the data and information should be available at any point of the area covered. This requires the development of interpolation model(s), and of inputs that are required by such model(s).
- Scientific interpretability: the network should provide the possibility of relating the water resources processes to their causative factors. This is required for data interpolation and environmental impact prediction.

4.3 Components of IHN

In view of the objectives and characteristics of the IHN, it should be considered to consist, similarly to the BHN (Solomon, 1981), of three main components:

i) a set of gauging stations and required staff, equipment and techniques for carrying out measurements on the various components of the water cycle, and primary processing, storing and disseminating the data (measurements);

ii) models for data, interpolation and interpretation;

iii) auxiliary data required as inputs into the above model(s).

The general practice in most countries, Canada included, is to consider that component (i) by itself constitutes essentially the BHN. The other two components are either ignored or relegated to a minor role in selected areas. (Again a notable exception is the B. C. Region, see Kreuder 1979).

4.4 Economic Design Objectives and Design Parameters

4.4.1 Economic design objectives

In view of the objectives and characteristics of the IHN, from an economic viewpoint only one of the following two economic design objectives could be considered in the design of the IHN:

a) Obtain a target accuracy of interpolation of the pertinent hydrologic characteristics within a given time horizon for a minimum budget;

b) Obtain maximum accuracy of interpolation within a given time horizon, for a given budget.

4.4.2 Design parameters

The following design parameters could be considered:

a) The number of stations for each sub-type;

b) The distribution of the stations within each region;

c) The suite of measurements (parameters to be measured) at various sub-types of stations;

d) The frequency, technique, and standards of measurement for various parameters and sub-types of stations;

e) The equipment to be used at various sub-types of stations;

f) The duration of operation of the various sub-types of stations;

g) The interpolation model(s) to be used for each parameter (this may also vary regionally);

h) The types of auxiliary data to be used as inputs for various models;

i) The number and qualifications of the staff;

j) Routing of measurement teams and location of regional offices and sub-offices.

In some cases various design constraints, such as availability of skilled personnel, or possible constraints in equipment procurement may come into play. However, such constraints do not generally apply in the case of Environment Canada.

4.5 Design Approach

4.5.1 General considerations

The complexity of the selection process of design parameters to meet the technical and economic objectives is such that it would not be practical to try to set up and solve simultaneously for all parameters. Therefore it is advisable to consider some of the design parameters as given and solve for the remaining ones. The assumed given design parameters (which are considered constant when solving for the first iteration) can be then varied in one direction or another and the results used to optimize for the parameters considered initially as given. The procedure can be iterated several times until the improvements obtained are too small to warrant further iterations.

However, in the design of a country-wide IHN it is not practical to apply this iterative technique to all design parameters. To remain in a practical domain the network design parameters have to be divided into two groups: the first includes the more general design parameters, such as number of stations, regional density of stations, duration of sampling, frequency of measurement, type of equipment, and selection of model(s) of interpolation and of type of auxiliary data. The second includes design parameters that are of a more specific type, such as distribution and routing of operational staff, timing of flow measurement and sediment and water quality sampling, and number and duration of velocity measurements. The first group of design parameters can and should be established at the national level. The second group may be subject to standardization (such as duration of velocity measurements) or could be left to be decided in the operation stage. In fact, there is continuous significant progress in improving measurement techniques as reflected for example in IAHS (1982) and in techniques of team routing and primary processing (Moss and Thomas, 1982). Therefore, any attempt to optimize operation of a network during its design stage would be rapidly invalidated. Consequently the design stage should concentrate on the station density, location parameters, length of sampling, and technique of interpolation of information, leaving the other parameters to be optimized in the operational stage.

The next sections are therefore devoted to assessing the error of interpolation in terms of the above mentioned design parameters, and to related procedures of achieving the design objectives (given in 4.4).

4.5.2 Errors of hydrologic information related to planning decision variables

Most of the techniques of interpolation of hydrologic parameters using auxiliary data rely on least squares optimization procedures. Consequently they are similar to the multiple regression technique for which Matalas and Gilroy (1968) have derived the following expression for the estimated error variance $\hat{V}'_{tt}$ of a prediction at a site not used in the multiple regression analysis:

$$\hat{V}'_{tt} = (1 + \frac{1}{N_s} + \phi''_{tt})\sigma^2(\delta)$$

$$+ \{\rho' + \frac{1-\rho'}{N_s} + (1-\rho')\phi''_{tt}\}\sigma^2(\varepsilon). \qquad (4.1)$$

where N_s is the number of data used to develop the multiple regression between the hydrologic characteristic and the independent variables (auxiliary data) considered;

ϕ''_{tt} is the (t,t) element of the matrix ϕ'';

$\phi'' = \phi'(\phi')^T$; and

$\phi' = X' A^{-1} X^T$, an $N' \times N_s$ matrix,

in which

N' is the number of ungauged sites for which estimates are made;

$X = |(x_{jk} - \bar{x}_k)|$ is an $N_s \times m$ matrix, m being the number of independent variables (auxiliary data) used in the multiple regression;

x_{jk} is the k-th independent variable for the j-th gauged basin;

$$\bar{x}_k = \frac{\sum_{1}^{N} x_{jk}}{N_s} ;$$

$X' = |(x'_{jk} - \bar{x}_k)|$ is an $N' \times m$ matrix;

x'_{jk} is the k-th independent variable in the j-th ungauged basin;

$A = X^T X$;

A^{-1} = the inverse of A;

$\sigma^2(\delta)$ is the average error variance due to space sampling error in a given regression;

$\sigma^2(\varepsilon)$ is the average error variance associated with the time sampling error at the stations used in the regression;

ρ' is the average correlation between the series of data z_{ij} used to define the given hydrologic characteristic Y_j (a mean, a variance) where z_{ij} is the i-th observation at station j (the cross-correlation coefficients).

Equation 4.1 is based on a series of assumptions in accordance with classical regression analysis, namely:

(1) For any set $X_m = | x_1 \ldots\ldots\ldots, x_m |$ the variance of y is homoscedastic, i.e. the variance is the same,

(2) For a given set X_m the associated y's are normally and independently distributed with mean, $\bar{E}$,

$$\bar{E} \{ y | X_m \} = \alpha + \sum_{k=1}^{m} \{ \beta_k (x_k - \bar{x}_k) \} ,$$

where

α is the multiple regression constant and β_k are the multiple regression coefficients, and variance $V \{ \delta + \varepsilon | X_m \} = \sigma^2(\delta) + \sigma^2(\varepsilon)$.

(3) X_m are error free variables.

None of the above assumptions holds for the hydrologic variables which form the object of our analysis. However, Eq. (4.1) gives some insight into the causes of increased error variance at an ungauged site, which are related particularly to the differences between the physiographic characteristics (independent variables) of the ungauged basin considered and the average characteristics of the sample used to establish the correlation. This indication is important and has been considered in the delineation of model-homogenous hydrologic regions (Section 4.5). The further implication of Eq. (4.1) is that the total error variance is smaller when the cross correlation is smaller, an indication first that dense networks are inefficient, and that sampling should be done in such manner that cross correlation be avoided as much as possible. This shows that from the viewpoint of IHN design it is interesting to obtain a decrease of the cross correlation by stratified sampling, that is by installing gauges in basins having distinct hydrologic regimes. This can be arrived at by means of delineation of physiographically homogenous regions (Section 4.5).

If the error variance due to interpolation is determined by split sampling, the expected value of the component of the error variance

$$\left(1 + \frac{1}{N_s} + \phi''_{tt} \right) \sigma^2(\delta)$$

can be estimated by $\sigma^{*2}(\delta)$, the error variance determined from the validation portion of the sample, and assuming that calibration and validation data are error free with respect to time sampling or other error sources. This assumption is less disturbing if all data series have been extended by multiple (usually trivariate) cross correlation to a common period of record.

Numerous computations of the value of ϕ''_{tt} made on the basis of sets of physiographic characteristics obtained within "model-homogeneous regions" as defined in Sub-section 4.5.5 show that ϕ''_{tt} is proportional to $\frac{1}{N_s}$ and has values which oscillate around an average* of $\frac{1.5}{N_s}$. Consequently:

$$(1 - \rho') \left(\frac{1}{N_s} + \phi''_{tt}\right) \simeq \frac{2.5(1 - \rho')}{N_s} \qquad (4.2)$$

---

* On the basis of the assumption that the physiographic characteristics are normally distributed, the expected value of ϕ''_{tt} is $\frac{1.33}{N_s}$. Because of the skewness of the actual distributions of the physiographic characteristics, the coefficient of proportionality is closer to 1.5 .

Furthermore, in the variance component $\sigma^2(\varepsilon)$ we shall consider, in addition to the error variance due to time-sampling error $\{\delta^2(t)\}$, also the error of measurement $\{\sigma^2(m)\}$. Thus, for our purposes, Eq. 4.1 will be used as:*

$$\hat{V}_{tt} = \sigma^{*2}(\delta) + \{\rho' + \frac{2.5(1-\rho')}{N_s}\}\{\sigma^2(t) + \sigma^2(m)\}$$

Starting from the generally accepted assumption that the error variance of the estimate of an expected value is proportional to the reciprocal of the number of independent measurements, the following notations are introduced:

$$N_s\sigma^{*2}(\delta) = E_s^{*2}$$

$$N_t\sigma^2(t) = E_t^{*2}$$

$$N_m\sigma^2(m) = E_m^{*2}$$

in which

E_s^* is an error variance parameter characterizing the error due to information transfer;

E_t^* is an error variance parameter characterizing the error due to limited sampling in time;

N_t is the average number of independent observations in time carried out at the hydrologic stations;

---

* $\sigma(t)$ and $\sigma(m)$ are assumed to be uncorrelated.

E_m^* is an error variance parameter characterizing the error due to the limited number of flow measurements;

N_m is the average number of independent flow measurements carried out at the hydrologic stations.

With the above notations, Eq. 4.2 can be written as:

$$E^2 = \frac{E_s^{*2}}{N_s} + \left\{ \frac{E_t^{*2}}{N_t} + \frac{E_m^{*2}}{N_m} \right\} \left\{ \rho' + \frac{2.5(1-\rho')}{N_s} \right\} \qquad (4.3)$$

(in which, for the sake of simplicity we replace the symbol $\hat{V}_{tt}$ by E). The values of all the error variance parameters depend on the hydrologic characteristic considered, the network configuration, and the hydrology of the area. In addition, E_s^* depends on the method of information transfer considered, and E_m^* on the operation policy of the network and on the stability of the measurement sections. In view of the specific character of the values of the error variance parameters it was suggested by the writer that, for the purpose of hydrologic network planning, these parameters should be determined empirically by split sampling techniques.

It should be noted that, in the discussion which follows, the error variance parameters will be considered in terms of "relative error variance parameters" as the absolute error obtained from the empirical calculation is always divided by the estimated value of the variable considered. For the sake of brevity, however, we shall generally use the term, "error variance parameter". To facilitate the interpretation of the relative error variance parameters, the following quite realistic example is given. Let us assume:

$$E_s^* = 3, \quad E_t^* = 0.3, \quad E_m^* = 1, \quad N_s = 100,$$

$$N_t = 20, \quad N_m = 200, \quad \text{and} \quad \rho' = 0.5 .$$

Then Eq. (4.3) gives the following expected relative error:

$$E = \sqrt{\frac{9}{100} + (0.5 + \frac{1.25}{100})\ (\frac{0.09}{20} + \frac{1}{200})}$$

$$E = 0.308 \quad \text{or} \quad 30.8\ \% .$$

4.5.3 Estimation of error variance parameters

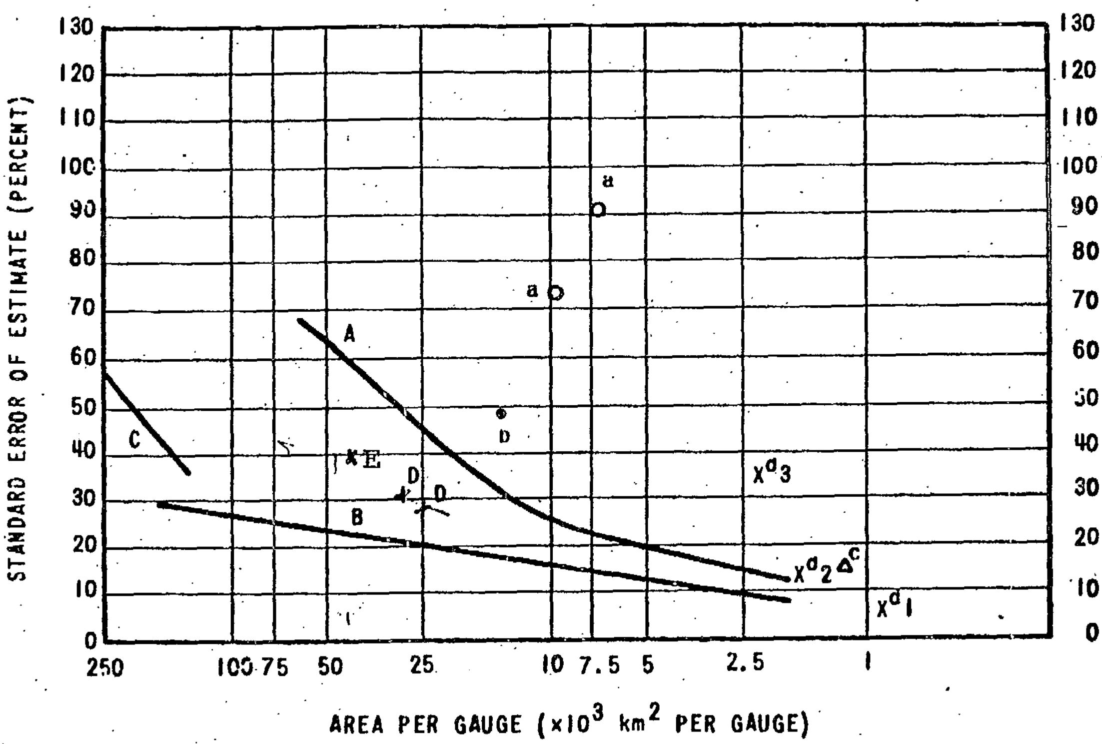
As was mentioned in the previous sub-section, the method suggested for estimating the error variance parameters E_s^*, E_t^*, and E_m^* is based on the application of a split sampling technique and consists of the following:

The sample of data available is divided into two parts: A calibration sample, which is used to derive the given model for information transfer; a set of validation samples, which is used to estimate the expected value of the error variance, and from this the variance parameter. The data on dependent and independent variables used for validation as well as for calibration are assumed to be error-free, the only error being related to the model. Of course, this assumption is very far from being true. However, any consideration of errors in the data would make the problem intractable from a practical viewpoint. As will be shown in the following sub-sections, some allowance is made in the estimation of the error variance parameters to account to some extent for such simplifying assumptions.

4.5.3.1 Error due to information transfer in space

The values of E_s^* characterizing the error due to information transfer for various techniques (isoline maps, conventional regression analysis, square grid technique, regression analysis using independent variables determined by the square grid technique, and parametric models) and for various hydrologic parameters can be inferred from Figures 4.1 and 4.2 (McMahon, 1979). Results of studies carried out recently for Northwest Territories by Shawinigan (1982) and by Solomon and Associates in the Amazon River Basin are added on that graph. From the analysis of the graph it appears that E_s^* varies primarily with the technique used, the square grid technique or regression analysis with independent variables estimated by the square grid technique giving generally the best results.

Another factor that may play a role in the results is the data error. As indicated earlier, carrying out the analysis for estimating the model error, the hydrologic and auxiliary data are assumed error free. In fact measurement errors (to a lesser extent time sampling errors) as well as errors in estimating the drainage areas of the basins, may play an important role in the actual value of the error. This explains why errors of interpolation for areas where measurement is difficult (the Arctic in Canada, the equatorial forest in Brazil) appear larger than in other areas (Figure 4.1). In addition, in the Amazon River basin the delineation of the river basin area is questionable in many cases (see Figure 6.1).

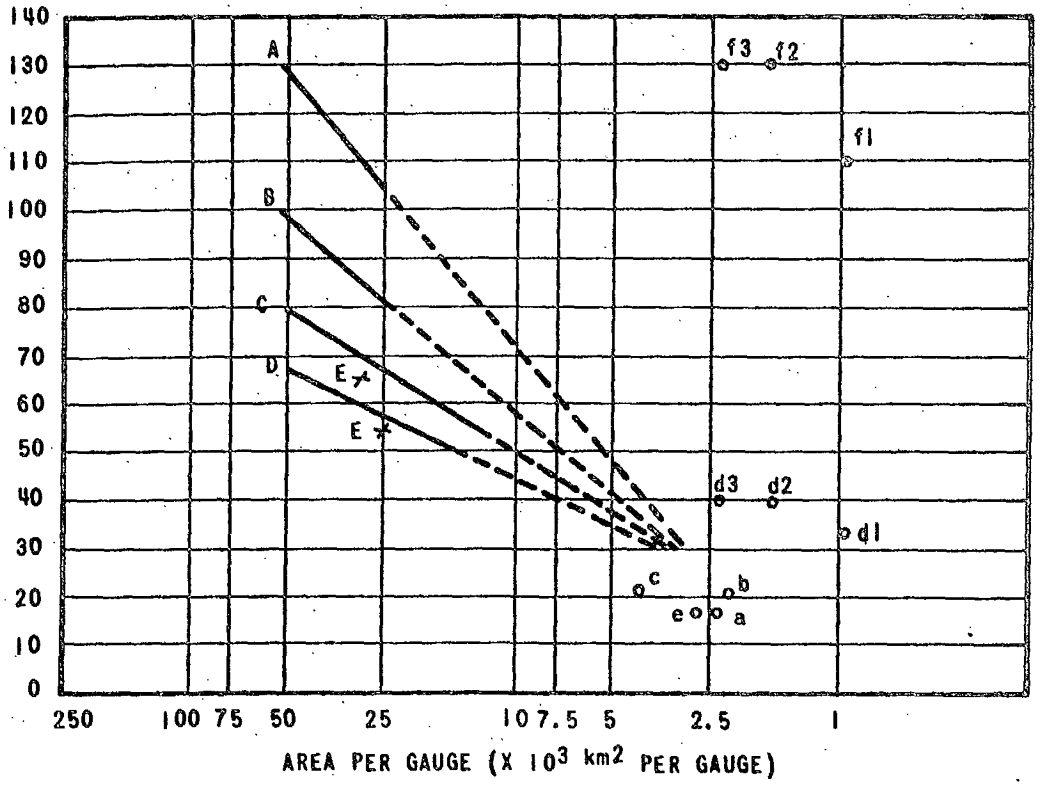


MEAN ANNUAL RUNOFF

A	REGRESSION BETWEEN MEAN ANNUAL FLOW AND PHYSIOGRAPHIC CHARACTERISTICS (SHAWINIGAN, 1970, FIG. 4.2.11)
B	SQUARE GRID METHOD - MEAN ANNUAL RUNOFF (SHAWINIGAN, 1970, FIG. 4.2.11)
C	GRID SQUARE PARAMETRIC MODEL (SHAWINIGAN, 1970, FIG. 4.3.2)
Δ	ISOLINE VALUES (SHAWINIGAN, 1970, FIG. 4.2.11)
b	REGRESSION BETWEEN MEAN ANNUAL FLOW AND PHYSIOGRAPHIC CHARACTERISTICS (LEITH, 1976)
c	ISOLINE VALUES (COULSON, 1967)
d_1, d_2, d_3	REGRESSION BETWEEN MEAN ANNUAL FLOW AND PHYSIOGRAPHIC AND CLIMATIC CHARACTERISTICS FOR EASTERN, SOUTHERN AND WESTERN REGIONS OF USA RESPECTIVELY (THOMAS AND BENSON, 1970)
D	SQUARE GRID METHOD MEAN ANNUAL RUNOFF - Mackenzie River Study, Shawinigan, 1982
E	Amazon River Study, WMO, 1983.

Figure 4.1 STANDARD ERROR OF MEAN ANNUAL RUNOFF VERSUS GAUGING DENSITY

(Based on a figure from MacMahon, 1979)



A. WINTER-SPRING MAXIMUM RUNOFF (SHAWINIGAN, 1970, FIG. 4.2.16)

B. SUMMER-FALL MAXIMUM RUNOFF (SHAWINIGAN, 1970, FIG. 4.2.16)

C. WINTER-SPRING MINIMUM RUNOFF (SHAWINIGAN, 1970, FIG. 4.2.16)

D. SUMMER-FALL MINIMUM RUNOFF (SHAWINIGAN, 1970, FIG. 4.2.16)

a. 20-YEAR RETURN PERIOD PEAK DISCHARGE FOR SAINT JOHN RIVER BASIN (DEPT. OF ENVIRONMENT, OTTAWA, 1972)

b. 20-YEAR RETURN PERIOD PEAK DISCHARGE FOR NEW BRUNSWICK (ACRES, 1971)

c. MEAN ANNUAL FLOOD FOR NEW BRUNSWICK-GASPE REGION (COLLIER AND NIX, 1967)

d1,d2,d3 REGRESSION BETWEEN 20-YEAR FLOOD AND PHYSIOGRAPHIC AND CLIMATIC CHARACTERISTICS FOR EASTERN, SOUTHERN AND WESTERN REGIONS OF USA RESPECTIVELY (THOMAS AND BENSON, 1970)

e. MEAN ANNUAL FLOOD FOR NOVA SCOTIA (COULSON, 1967)

f1,f2,f3 REGRESSION BETWEEN 7-DAY 20-YEAR LOW FLOW AND PHYSIOGRAPHIC AND CLIMATIC CHARACTERISTICS FOR EASTERN, SOUTHERN AND WESTERN REGIONS OF USA RESPECTIVELY (THOMAS AND BENSON, 1970)

E. MEAN JUNE DAILY MAXIMUM RUNOFF. Mackenzie River Study, Shawinigan, 1982.

Figure 4.2 STANDARD ERROR OF PEAK AND LOW FLOWS VERSUS GAUGING DENSITY

(Based figure from MacMahon, 1979)

4.5.3.2 Error due to a limited sampling period

Wallis and Matalas (1972) have investigated the error of estimate of various statistics of hydrologic characteristics obtained from time series of auto-correlated data and time series of flow data extended by correlation with other time series of flow data obtained at long-term gauging stations. Because of the unavailability of an analytical solution for estimating the variance of the error of estimate of hydrologic characteristics obtained from auto-correlated data augmented by multiple correlation, Wallis and Matalas used a Monte Carlo technique for obtaining such estimates.

As the technique used by Wallis and Matalas is based on several assumptions on the statistical characteristics of the time series of data which do not correspond entirely to the actual properties of the data, and as their work is not providing a complete analytical solution, the writer decided to use an empirical approach for the derivation of E_t^*, based on the following considerations:

It is practically impossible to determine the time sampling error because one does not have an "error-free" infinite length value to compare with in the general case of time series of hydrologic data. Furthermore, the "effective number of years of record" is reduced and the error variance accordingly increased because of the effect of serial correlation (Yevjevich, 1972). In most cases in Canada, data for a period of record longer than 50 years are not available. If one uses a 50-year record to obtain the "error-free" data, then one underestimates to a certain extent the error due to limited time sampling. Furthermore, the use of the coefficient of serial correlation to decrease the number of years of record is questionable, since this coefficient generally decreases as the period of record increases, and values obtained from available records would over-estimate the error values if the results are applied to longer periods. However, if one uses the actual number of years of record as the "effective number of years of record", this leads to another underestimation of the error variance.

In summary, if one elects to estimate the error variance parameter E_t^* assuming that the longest period of record, e.g. a 50-year period, produces the error-free data (from the viewpoint of time sampling error) and uses the actual number of years of record as the "effective number of years of record", an under-estimation of the actual values of E_t^* would be obtained. Let k denote the ratio between the square of the actual error variance parameter E_t^{*2} and the value estimated on the basis of the above mentioned assumptions $E_{t,50}^{*2}$. The value of k is larger than one and varies probably between 1.50 and 2.0. It was shown that in Equation 4.3 giving the total error variance in terms of its components, the error variance parameter E_t^{*2} is multiplied by a factor $\{\rho' + \frac{2.5(1-\rho')}{N_s}\}$. Since for the usual values of ρ' and N_s ($\rho' = 0.6 - 0.7$, $N_s = 50 - 100$), the value of this factor varies between 0.62 and 0.705, we can write:

$$E_t^{*2} \simeq (1.5 \div 2.0)\, E_{t,50}^{*2}$$

$$\{\rho' + \frac{2.5(1-\rho')}{N_s}\}\, E_t^{*2} \simeq (0.62 \div 0.705)\, E_t^{*2}$$

$$\{\rho' + \frac{2.5(1-\rho')}{N_s}\}\, E_t^{*2} \simeq k\, \rho'\, E_{t,50}^{*2}$$

Note that N_s is the number of stations within a model homogeneous region. Note also that E_t^{*2} increases almost proportionally to ρ', the coefficient of cross correlation.

On the basis of the above considerations, the writer suggests that, for practical planning purposes, it would be acceptable to replace in Equation 4.3 the term $\{\rho' + \frac{2.5(1-\rho')}{N_s}\} E_t^{*2}$ by $k \rho' E_{t,50}^{*2}$, computed using the above stated assumptions. Consequently, the computation of $E_{t,50}^{*}$ was carried out using earlier results (Solomon and Davis, 1970) to estimate from samples of data of 50 years and longer values of $E_{t,50}^{*2}$ for a number of statistics of hydrologic characteristics (Table 4.1).

Moss (1979) provided graphs that relate total error of estimate of various hydrologic parameters to length of record and number of stations in a given area (Figure 4.3). Number of stations in a given area can be readily translated into station density, by dividing the number of stations by the study area. By reading the graphs shown in Figure 4.3 along a constant sampling time one obtains values relating error of interpolation to station density, i.e. a relationship similar to that shown in Figures 4.1 and 4.2 . By reading the graphs along a constant value of number of stations one obtains values relating error of sampling to number of years of record, i.e. a relationship similar to those shown in Figure 4.4 . In fact, Figures 4.1, 4.2 and 4.4 could be used to synthesize figures similar to Figure 4.3 for interpolation techniques other than that used by Moss (multiple regression with conventionally estimated physiographic characteristics).

TABLE 4.1

EXPECTED VALUES OF $E^*_{t,50}$ OBTAINED FROM DATA AT SIX GAUGING STATIONS IN CANADA

	Long-term mean	Coefficient of Variation
Annual Runoff	0.32	0.40
Spring "	0.60	0.45
Summer "	0.68	0.42
Fall "	0.57	0.60
Winter "	0.57	0.54
January "	0.56	0.65
February "	0.57	0.54
March "	0.56	0.60
April "	0.67	0.75
May "	0.69	0.27
June "	0.71	0.48
July "	0.69	0.40
August "	0.57	0.40
Sept. "	0.57	0.60
October "	0.63	0.96
November "	0.63	0.66
December "	0.59	0.60

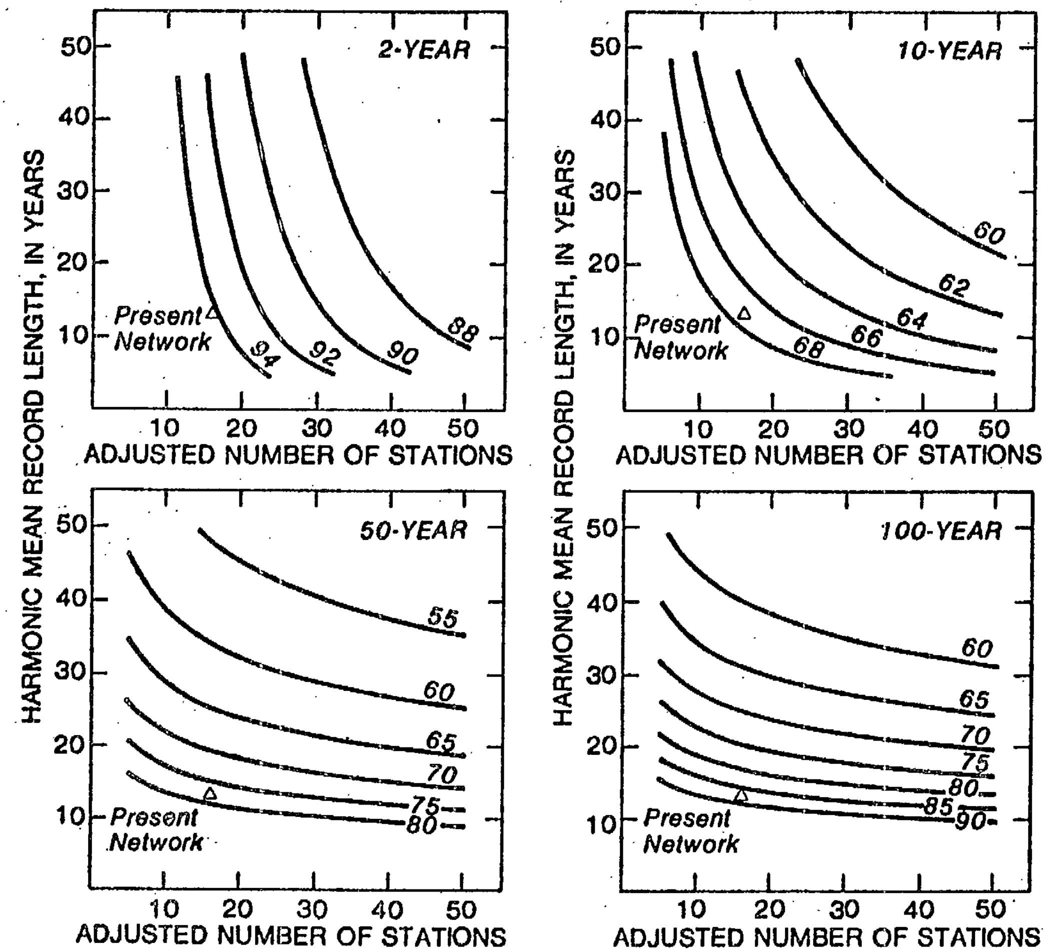
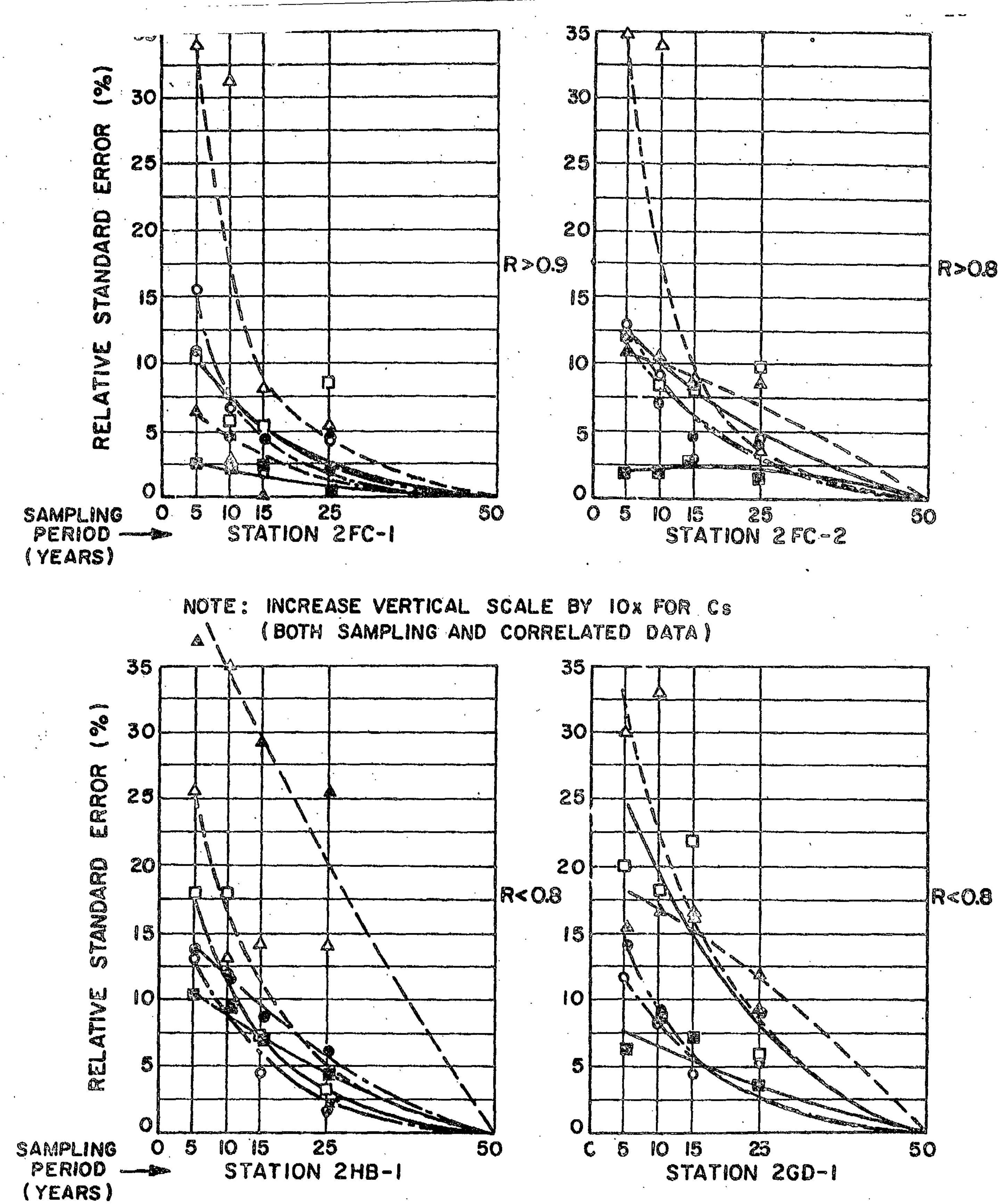


Figure 4.3 Accuracy in percentage of regression estimates of flood frequency in northwestern Arizona.

(from Moss, 1979)



LEGEND — SAMPLING DATA — CORRELATED DATA
MEAN FLOW
CV
CS

Figure 4.4

RELATIONSHIP BETWEEN RELATIVE STANDARD ERROR OF LONGTERM AVERAGE FLOW, COEFFICIENT OF VARIATION AND COEFFICIENT OF SKEW OF ANNUAL FLOW VERSUS LENGTH OF SAMPLING PERIOD.

4.5.3.3 Error of measurement

The error of measurement could be assumed to consist of two parts, one related to the error of flow measurement (E_{mf}), the other (E_{mN_m}) related to the number and range of flow measurements available for obtaining the stage-discharge relationship. Other sources of error (such as errors in stage records) can be ignored in the planning process since they are related mainly to operational problems, and since many sources of error of such type are currently eliminated by introducing automated and more reliable equipment. Assuming independence between the two sources of errors, one can write

$$E_m^2 = E_{mf}^2 + E_{mN_m}^2$$

The error of measurement was studied by Dickinson (1967). His study is significant mainly from the viewpoint of design and operation of the network. Relevant from the viewpoint of network design are the following conclusions:

"The magnitude of measurement error in a single discharge measurement and the degree of minor random shifting of a rating curve are reflected in the co-efficient of variation (relative standard error) from regression for the sample data. The average co-efficients for the upper portion of the curve are +9.37 and -8.40 percent; and the average co-efficients +17.3 and -10.9 percent on the lower portion.

Summer mean daily discharge values can be expected to be estimated within +2.40 and -2.33 percent error bounds, at the 80 percent confidence level; and within +3.82 and -3.65 percent, at the 95 percent level. Mean daily estimates in the winter months can be said to be within +8.60 and -8.34 percent error bounds, at the 80 percent confidence level; and +13.7 and -13.1 percent, at a 95 percent level.

Future summer discharge measurements* may be expected to lie within +12.9 and -11.2 percent of the rating curve values, 80 percent of the time, and within +21.5 and -16.9 95 percent of the time. Single discharge measurements made in the winter months can be expected to fall within +47.6 and -39.6 percent of the curve, with 80 percent confidence; or within +76.9 and -59.6 percent, with 95 percent confidence.

If a single rating curve is employed to estimate all mean daily discharges during the summer, the percentage error bounds on the total summer flow, on each monthly flow, and on each mean daily value are equivalent.

During the winter months, when daily flows are estimated independently, the percentage error bounds on a monthly flow estimate are a function of the percentage error bounds on a summer mean daily flow estimate, and the relative magnitude of the monthly estimate and the divisive discharge value.

The percentage error bounds on an annual discharge estimate are negligibly affected by the errors in winter flow estimates.

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* These are equivalent to validation date in our terminology.

They are approximately equal to the product of the ratio of summer to a total annual flow and the percentage error bounds on a mean daily discharge estimate in the summer months."

From a planning viewpoint, the important conclusion is that the error of an average does not depend in the case of summer flows on the number of data averaged. Dickinson's conclusion on the errors during the winter months is slightly confusing and difficult to apply practically because of the introduction of the concept of "divisive discharge value" (which is defined by Dickinson as the discharge arising from velocities less than one foot per second and/or depths less than one foot). In fact, most methods of determining daily flows in winter in Canada (see for example, Rosenberg and Pentland, 1966) do not imply an independent estimation of each flow, and consequently in this case also the errors are strongly correlated and independent of the number of days used for the computation of the given statistics.

An element which has not been investigated by Dickinson, but was considered by the writer because of its significance for planning, was the error due to differences between the true rating curve and the rating curve obtained from a limited number of flow measurements. Although for particular cases this difference will depend on a large number of factors, such as the range of the measurements, the shape of the cross-section, the hydraulic conditions at the cross-section, etc., on the average the most important factor is probably the number of flow measurements*. At any rate, the latter factor is the

---

* Assuming that operational measures are taken at all gauges to expand as much as possible the range of measurements.

one which is most important from the viewpoint of planning the network, whereas the others are design and operation factors.

In view of this, a special investigation was carried out to estimate the relationship between the error of estimate of a daily flow and the number of flow measurements used for the estimation of the stage-discharge relationship. The investigation was based on flow values determined at stations with well defined stage-discharge relationships which were compared with flow values obtained by using stage-discharge relationships determined on the basis of various samples of flow measurements drawn from the "population" of flow measurements available. This made it possible to determine the error of flow estimate as a function of the number of flow measurements.

The stage-discharge curves were computed using the same equation as that used by Dickinson,

$$Q = b_1 \; (H - H_0)^{b_2}$$

where Q is the discharge, H, the stage, H_0 the stage at zero discharge, b_1 and b_2 parameters; b_1 and b_2 were determined by trial and error, using an optimization technique (the H_0 selected was that for which the coefficient of correlation of the regression of $\log Q$ versus $\log (H - H_0)$ was a maximum).

The investigation was carried out at 5 stations where the conditions were appropriate. It was found that an equation relating the standard error of measurement E_{mN_m} and N_m, having the form:

$$E^2_{mN_m} = \frac{E^{*2}_{mN_m}}{N_m}$$

can be obtained in each case.

The values obtained for $E^{*2}_{mN_m}$ were:

River and station	$E^{*2}_{mN_m}$
Saugeen R. near Port Elgin	0.27
Otter Creek at Tillsonburg	0.92
Young Creek near Victoria	0.74
Thames R. at Byron	0.20
Gull R. at Norland	0.07

Given the large variation of the value of $E^{*}_{mN_m}$ obtained at the various stations, it is recommended that further research should be carried out on this subject. However, for the practical applications an average value $E^{*2}_{mN_m} = 0.44$ could be used. Furthermore, because from the above given value of $E^{*2}_{mN_m}$ and from the above quoted conclusions of Dickinson's work, it would appear that the error of flow measurement is approximately equal to the error due to limited number of flow measurements for the usual number of flow measurements at a station (~ 200 - 300) it was decided that in further analysis the term $\{\rho' + \frac{2.5(1-\rho')}{N_s}\} E^2_m$, which for most frequent values of ρ' and N_s can be estimated to be approximately equal to $(0.62 \div 0.70)\ E_m^2$, be replaced by

$$k'\ \rho'\ \frac{E^{*2}_{mN_m}}{N_m}\ , \quad \text{where} \quad k' \simeq 1.4 \quad \text{since}$$

$$E_m = \sqrt{E^2_{mN_m} + E^2_{mf}} \simeq 1.4\ E_{mN_m}$$

$$\text{and } (0.62 \div 0.70)\ E_m^2 \simeq E^2_{mN_m} = \frac{E^{*2}_{mN_m}}{N_m}$$

Recent studies carried out at the USGS (Moss and Thomas, 1982) show that it is possible to reduce significantly the error of measurement by optimizing the routing of the measurement teams to increase the probability of measuring extreme flows, and by improving the techniques of estimation of the stage-discharge relationships. The latter is achieved by using a simple hydraulic model to account for changes in water surface slopes during flood events, and by applying a Kalman filter to the mathematical relationship between stage and discharge measured to minimize the relationship error. Further improvements could be achieved in the routing of measurement teams using telecommunication systems and meteorological forecasts as discussed in Chapter 6.

In summary, the analysis carried out in the last three sub-sections indicates that it is acceptable to estimate the total error variance at an ungauged station from the following relationship:

$$E^2 = \frac{E_s^{*2}}{N_s} + \rho' \left(k \frac{E_{t,50}^{*2}}{N_t} + k' \frac{E_{mN_m}^{*2}}{N_m} \right) \qquad (4.4)$$

with the error variance parameters estimated as shown in Sub-sections 4.5.3.1, 4.5.3.2, and 4.5.3.3 .

It should be emphasized that this formula represents an estimation of the expected value of the error variance acceptable for application for planning purposes and not for individual estimates of the error at a given site. In this latter case, one has to consider a formula of the type suggested by Matalas and Gilroy (1968), which takes into account the deviation of the characteristics of the given basins under consideration from the average characteristics of the sample used to calibrate the model.

4.5.4 Design of the natural-regime network

A tentative solution to this problem was presented by the writer in earlier papers and reports (Solomon, 1971, 1972, 1972a, 1976). The following treatment of the problem represents a slightly modified version of the earlier solution.

4.5.4.1 Error reduction

Assuming that the network consisting at present of N_{si} stations has been already operated for an average number of N_{ti} years, and that an average number N_{mi} of flow measurements was already carried out, the problem is to investigate the planning strategy for a supplementary number N_{ts} of years. During this period a percentage p of stations will be discontinued, but the total number of stations will be increased up to N_{sf}, and the total number of flow measurements per station up to N_{mf} by making n_{ms} flow measurements per year at each station during the period N_{ts}.

The error reduction can be estimated from the expression:

$$\Delta E = \sqrt{\frac{E_{si}^{*2}}{N_{si}} + \rho_i'\left(k\frac{E_{t50,i}^{*2}}{N_{ti}} + k'\frac{E_{mN_{mi}}^{*2}}{N_{mi}}\right) - \frac{E_{sf}^{*2}}{N_{sf}} - \rho_f'\left(k\frac{E_{t50,f}^{*2}}{N_{tf}} + k'\frac{E_{mN_{mf}}^{*2}}{N_{mf}}\right)} \tag{4.5}$$

where

$$N_{tf} = \frac{N_{si}\,N_{ti} + \left(N_{sf} - \frac{pN_{si}}{100}\right) N_{ts}}{N_{sf}} \tag{4.6}$$

and

$$N_{mf} = \frac{N_{si}\,N_{mi} + \left(N_{sf} - \frac{pN_{si}}{100}\right) N_{ts}\,n_{ms}}{N_{sf}} \tag{4.7}$$

The values of E_{si}^{*}, $E_{t50,i}^{*}$, $E_{mN_{mi}}^{*}$ and ρ_i' corresponding to the current conditions can be readily estimated using calibration-validation techniques as has been shown in the previous section. The values of E_{sf}^{*}, $E_{t50,f}^{*}$, $E_{mN_{mf}}^{*}$ and ρ_f' for various possible changes in data processing and interpolation techniques and station density have to be estimated together with the costs of such changes from research work carried out on methods of interpolation of hydrologic information in areas with dense networks and at stations with frequent measurements.

4.5.4.2 Additional costs

The additional cost ΔC for operating the network during the N_{ts} years will consist of four components:

$$\Delta C = C_0 + \Delta C_1 + \Delta C_2 + \Delta C_3 \qquad (4.8)$$

in which:

C_0 are annual costs which consist of a fixed portion (administration overhead) and a variable cost (which depends on the extent and nature of studies for information interpolation, stage-discharge extrapolation, etc.);

ΔC_1 is the cost of installing new stations;

$$\Delta C_1 = (N_{sf} - N_{si}) C_1 \qquad (4.9)$$

where C_1 is the average cost of installing new stations (a simplification of the study requires the assumption that all new stations are installed at the beginning of the planning period);

ΔC_2 is the cost of operating the network (excluding flow measurements and instrumentation);

$$\Delta C_2 = (N_{sf} - \frac{pN_{si}}{100})\ N_{ts}\ C_2 (PWN_{ts}) \qquad (4.10)$$

where C_2 is the cost of operating one station one year and PWN_{ts} is the average current worth of one dollar over a period of N_{ts} years.

$$\Delta C_3 = \Delta C_{03} + (N_{sf} - \frac{pN_{si}}{100})\ N_{ts}\ (PWN_{ts})\ n_{ms}\ C_3 \qquad (4.11)$$

where ΔC_{03} being the cost of new instrumentation, and C_3 the cost of one flow measurement.

Note: In the above equations, N_{sf} , N_{ts} , n_{ms} , p , and C_0 represent the basic design decision variables for planning the network; E^*_{si} , $E^*_{t50,i}$, and $E^*_{mN_{mi}}$ represent the initial network characteristics in the area considered; E^*_{sf} , $E^*_{t50,f}$ and $E^*_{mN_{mf}}$ represent together with ΔC_{03} the technical-economic improvement potential in the planning and operation of the network and the availability of data from other networks; and PWN_{ts} , C_0 , C_1 , and C_2 reflect the prevailing technical-financial conditions.

4.5.4.3 Objectives and optimization

The various objectives of the planning of the natural regime network can be expressed by the following relationships:

$$\frac{\Delta C}{\Delta E} \rightarrow \min \qquad (a)$$

if we wish to obtain a minimum cost per unit of error reduction without any further constraints;

$$\frac{\Delta C}{\Delta E} \to \min, \text{ with } N_{ts} \leq N'_{ts} \qquad (b)$$

if we wish to obtain a minimum cost per unit of error reduction within a given period of operation of the network N'_{ts} ;

$$\frac{\Delta C}{\Delta E} \to \min, \text{ with } \Delta E \geq \Delta E_i \qquad (c)$$

if we wish to obtain a minimum cost per unit of error reduction but for an error reduction larger than a given value E_i ;

$$\Delta E \to \max, \text{ with } N_{ts} \leq N'_{ts} \qquad (d)$$

if we wish to reduce the error as much as possible within a given number of years N'_{ts} ;

$$\Delta E \to \max, \text{ with } \Delta C \equiv \Delta C_i \qquad (e)$$

if we wish to reduce the error as much as possible within a given total budget;

$$\Delta E \to \max, \text{ with } \Delta C = \Delta C_j \text{ and } N_{ts} \leq N'_{ts} \qquad (f)$$

for same conditions as above, but within a time limit N'_{ts} ;

$$\Delta C \to \min, \text{ with } \Delta E = \Delta E_j \qquad (g)$$

if we wish to obtain a certain error reduction ΔE_j at a minimum cost;

$$\Delta C \rightarrow \min, \text{ with } \Delta E = \Delta E_k \text{ and } N_{ts} \leq N'_{ts} \qquad \text{(h)}$$

if we wish to obtain a certain error reduction at a minimum cost and within a certain time interval.

Other constraints could be introduced according to particular circumstances, e.g. an upper limit for n_{ms} the number of flow measurements per year per station in connection with manpower limitations, etc.

The application of the technique is straightforward in cases (a), (b), (c) and (d) since it involves the optimization of a function which has in the last three cases, one variable subject to a constraint. In the cases (e), (f), (g) and (h), for which, in addition to obtaining an optimum value of the function we have to also satisfy an equation, e.g. $\Delta C = \Delta C_i$ and which are in fact most interesting from a practical viewpoint, the solutions require the introduction of Lagrange's multipliers. Thus, for example, in case (e) the function to be optimized is written as

$$e = \Delta E + \lambda(\Delta C - \Delta C_i) \qquad (4.12)$$

in which λ is a Lagrange multiplier. With this the problem is reduced to the previous case. This optimization can be done by means of a steepest descent (ascent) technique of multivariate optimization for which there are well-documented computer programmes (e.g. CLIMB at the University of Waterloo). The technique is capable of circumventing difficulties related to the fact that some of the parameters of the function(s) to be optimized are not constant and cannot be expressed analytically. In this case, these parameters can be reduced in a tabular form, if their values are obtainable in such a form.

The application of the technique requires assumed initial values of the parameters to be optimized. If the initial values are not appropriately selected, it is possible that a relative instead of an absolute maximum is obtained. This may represent a significant drawback to the method. To circumvent this, it is necessary to obtain an initial "mapping" of the objective function by computing its values for various selected combinations of the values of the variables within their limits of variation. Although the number of such combinations increases rapidly with the number of variables and intervals considered, the computation is not excessively expensive since the computer time involved is not significant.

One of the major advantages of the method consists of the fact that it enables one to estimate the error reduction caused by changes in techniques of information interpolation or other improvements in the planning or operation of the hydrologic network over the usual methods of increasing the accuracy by installing new stations, measuring more frequently and for a longer period of time, etc. The practical relevance and economic value of research thus become obvious.

4.5.5 Areal distribution of stations

Once the number of stations which corresponds to the optimum conditions according to the selected criterion has been determined, the next step consists in selecting the location of the stations in such a manner that the actual operation of the network satisfies as closely as possible the assumptions made in the optimization computations. In a first stage the location of the station is selected in a regional manner, i.e. decisions are made on the number of stations within a certain region and on the approximate size of the basins to be gauged. This first stage is discussed in this Section. In a second stage the selection of the appropriate locations of gauging stations has to be made from the viewpoint of the hydraulic and hydrologic conditions (stability of the section, the presence of a hydraulic control, ice conditions, vegetation, etc.), accessibility, etc. This second stage is of an operational nature and therefore beyond the scope of this Report.

The regional distribution of the gauging stations requires the examination of hydrologic, economic, and environmental-aesthetical considerations, as discussed in sub-sections 4.5.5.1, 4.5.5.2, and 4.5.5.3 respectively.

4.5.5.1 Hydrologic - geophysical considerations

From a geophysical viewpoint, the distribution of the optimal number of stations has two basic aspects. The first is related to ensuring within a region which is "model-homogeneous", i.e. a region in which one can apply basically the same model for interpolation of hydrologic characteristics (or synthesis of hydrologic time series), the minimum number of stations which will make it possible to interpolate (synthesize time series of flow) with the accuracy determined in the frame of the optimization process (4.5.5.1.1). The second aspect is related to the necessity of avoiding the installation of redundant stations within an area which is physiographically and hydrologically homogeneous, i.e. in which the various river basins have almost identical input, physiographic characteristics, and outputs. If this is not achieved the coefficient of cross correlation is very large and the error reduction ΔE small (Equation 4.5). The information gained by installing more than two to four stations within a physiographic hydrologic homogeneous area is small.

4.5.5.1.1 Model-homogeneous regions

The concept of the model-homogeneous region arises from the intention of delineating regions in which the term ϕ''_{tt} in Eq. (4.1) is minimized, and thus the error of information interpolation is also reduced.

The delineation of a model-homogeneous region can be done as suggested by Solomon (1972a, 1976) in the following manner:

a) The hydrologic characteristic for which the regionalization is made is chosen and a model of information transfer is selected for the given characteristic on the basis of available data on obtainable accuracy and costs. The existing hydrometric stations are equally divided into a calibration sample and a validation sample, and the parameters of the model determined from the calibration sample.

b) If the model parameters correlate directly with the physiographic characteristics of the corresponding basins, as for example in the case of multiple regression techniques, one can proceed directly to the estimation by means of the model of the hydrologic characteristic at the validation and calibration stations, and estimate the errors. If the model parameters do not correlate directly to the physiographic characteristics, as for example in the case of parametric models, one first correlates model parameters with physiographic characteristics, synthesizes the former at the validation stations and then proceeds to estimate the errors at the calibration and validation stations.

c) One then separates the whole area into two "super-regions":

(i) The "gauged" super-region which includes all basins which actually have gauges or whose physiographic characteristics fall within the range of the corresponding physiographic characteristics of the gauged river basins.

(ii) The "ungauged" super-region containing all other basins.

A first approximation of the number of new gauging stations (n'_s) which should be allocated to each "super-region" (or region) can be then obtained by using the formula

$$n'_s = 5 \sum_{1}^{j} \frac{C_{vpi} - C_{vsi}}{C_{vsi}} - n_s \qquad (4.13)$$

where j is the number of significant and potentially significant physiographic characteristics. Significant physiographic characteristics are those included in the correlations used for the direct and indirect modeling. Potentially significant are those physiographic characteristics which have not been selected as significant in the model, but for which the coefficient of variation for the gauged basins C_{vsi} is less than 0.5 of the population coefficient of variation C_{vpi}, and n_s is the number of existing stations.

d) The "gauged" super-region is then examined for possible further subdivision. This is done by examining the geographic distribution of the validation errors. If the geographic distribution of the errors as established by visual inspection or preferably by statistical tests (e.g. run tests, Solomon, 1976) can be considered random, then the region is considered homogeneous (the whole "gauged super-region" is considered to belong to one model-homogeneous region). If the geographic distribution of the errors indicate a geographic trend for grouping in various ranges, then this result is construed as an indication that the selected model does not apply to the whole "gauged" super-region, and that a second order subdivision is indicated. A second order subdivision defined at this stage includes all basins which have their physiographic characteristics within the range of the physiographic characteristics of the basins whose errors were included within the chosen error range used as a criterion for subdividing the "gauged super-region".

e) After defining the second order subdivision the procedure is iterated in the sense that the model is applied separately to each second order subdivision and step (d) applied again for possible third order subdivision, etc.

Figure 4.5 shows the result of the application of this technique to Western and Northern Canada (Solomon, 1972a) using the mean annual runoff as the planning criterion and a multiple regression as the model applied. The significant effect of topography in relation to the surrounding oceans on the delineation of model-homogeneous regions is obvious in this case. Similar results are indicated preliminarily by the current studies of the Amazon River basin in the western area of the basin. It is interesting to note that the application of the same technique to the Island of Newfoundland (Solomon et al, 1968, and Shawinigan, 1982) for the same criterion and model as above lead to the delineation of only two regions (Figure 4.6), and of one single region for the whole of Southern Ontario (for the area shown in Figure 4.3 of Solomon, 1972). A similar technique, but using as model regressions between statistics of hydrologic characteristics and physiographic characteristics obtained by applying the square

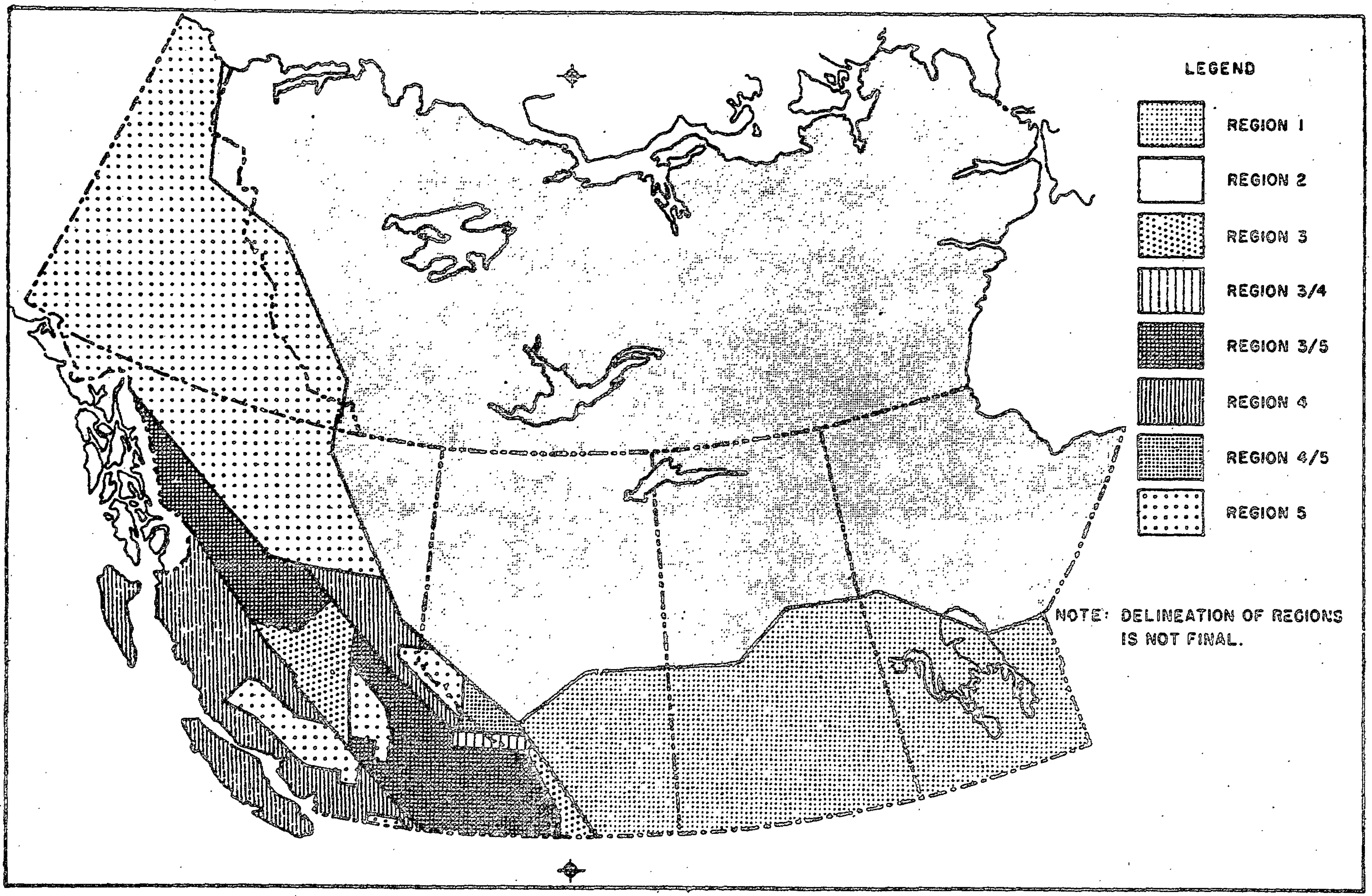
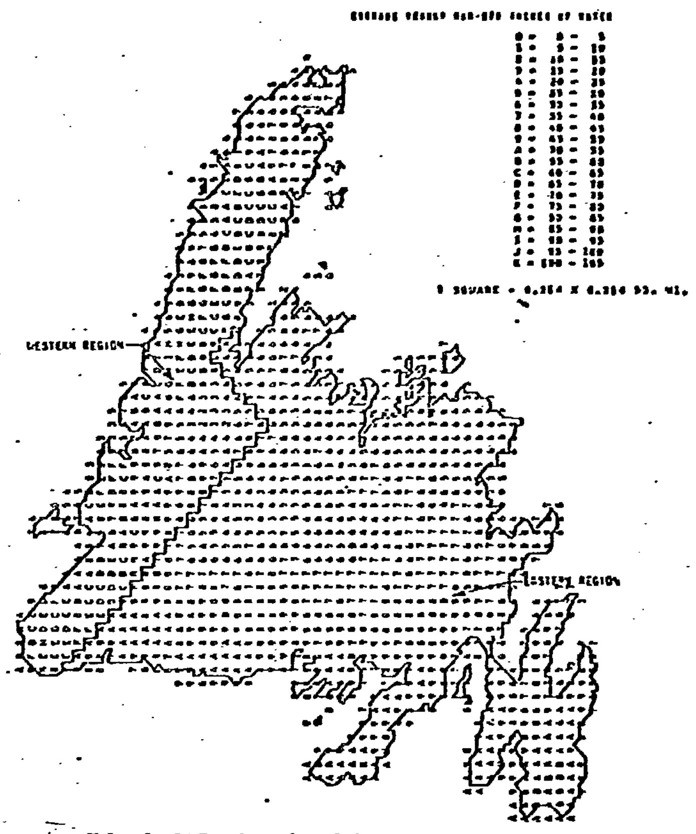


Figure 4.5- MODEL-HOMOGENEOUS HYDROLOGICAL REGIONS IN WESTERN AND NORTHERN CANADA
From Solomon, 1972



Island of Newfoundland. Mean annual runoff distribution.

Figure 4.6 Model homogeneous regions in the Island of Newfoundland

(From Solomon et al, 1968)

grid technique was applied by Leith in British Columbia (Leith, 1976) with similar results.

It should be noted that the procedure requires that each subdivision includes a large number of stations to make the correlations between model parameters and physiographic characteristics meaningful*. It is also required that a large portion (at least 50%) of the stations are reserved for validation, to avoid the influence on the regionalization process of the fact that one obtains a better "curve fitting" effect as the size of the subdivision and the corresponding number of stations (data) decreases. If the regionalization process indeed reflects the regional difference in the hydrologic conditions not covered by the physiographic characteristics considered and by the model of information interpolation chosen, a reduction and randomization of the errors of estimate will be observed for the validation samples when the model is applied region by region. In the contrary case, one usually obtains an increase in the errors of the validation samples, although randomization is not ruled out.

The need for many regions could also be an indication of an unsatisfactory statistical model and/or unsatisfactory suite of physiographic characteristics. Possible improvements in the physiographic characteristics suite and the model should be investigated in such cases.

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* If the number of gauges in a gauged region is not sufficiently large (at least 3 - 4 times the number of significant physiographic characteristics in the model) that region is attached to the "ungauged super-region" and treated accordingly.

Differences in region delineation for various hydrologic characteristics should be expected because of the different physiographic characteristics which determine them (e.g. in the case of maximum and minimum flow). However, large differences between region boundaries of the same area when they are delineated for different hydrologic characteristics are probably in most cases also an indication of the weakness of the model or insufficient sampling or errors in estimating physiographic characteristics considered. For example, in the recent Northwest Territories Study (Shawinigan, 1982) the independent variable percent area of forest was not included among the significant independent variables. A closer examination of this result indicated that area of forest as extracted from the available topographic maps is affected by significant errors.

Once the subdivision (regions) has been determined and the number of new stations estimated for each in accordance to Eq. (4.13), one can check if the total number of stations corresponds to that determined in accordance to the optimization technique applied to the whole area. If the number of stations determined is different from that obtained from the optimization computations, one can change the number of stations allocated in a proportional manner, so as to obtain the number of optimum stations.

The practical application of this regionalization procedure is facilitated very much by the use of the square grid interfacing bank of physiographic land use, and hydrometeorologic data. This data bank makes it possible to apply the procedure in a completely computerized manner.

4.5.5.1.2 Physiographic-hydrologic regionalization

Physiographic-hydrologic regionalization is carried out with the purpose of achieving the equivalent of stratified sampling. Physiographic-hydrologic regions are regions within which the significant physiographic factors as defined for example from correlations between hydrologic and physiographic characteristics or between model parameters and physiographic characteristics vary within narrow limits. One can expect that within regions defined in this manner the hydrologic characteristics which have been used in the selection of the significant physiographic characteristics vary also within narrow limits.

The technique for delineation of physiographic regions can be applied using the following steps:

a) Define correlations between various hydrologic characteristics (selected as criteria for the regionalization or for the network planning) or model parameters and physiographic characteristics and select all significant physiographic characteristics;

b) Analyze the coefficients of variation of each non-significant physiographic characteristic of the sample used and for those physiographic characteristics for which the sample C_v is less than 0.2 compute also the C_v of the population for the given physiographic characteristic. If the population C_v is larger than 0.4 include the corresponding physiographic characteristic in the list of potentially significant physiographic characteristics;

c) Select ranges (shades) for each significant and potentially significant physiographic characteristics (at least three shades for each characteristic), e.g. Shade 1: $X > \bar{X} + \frac{1}{2} Sd_x$

Shade 2: $\bar{X} + \frac{1}{2} Sd_x > X > \bar{X} - \frac{1}{2} Sd_x$

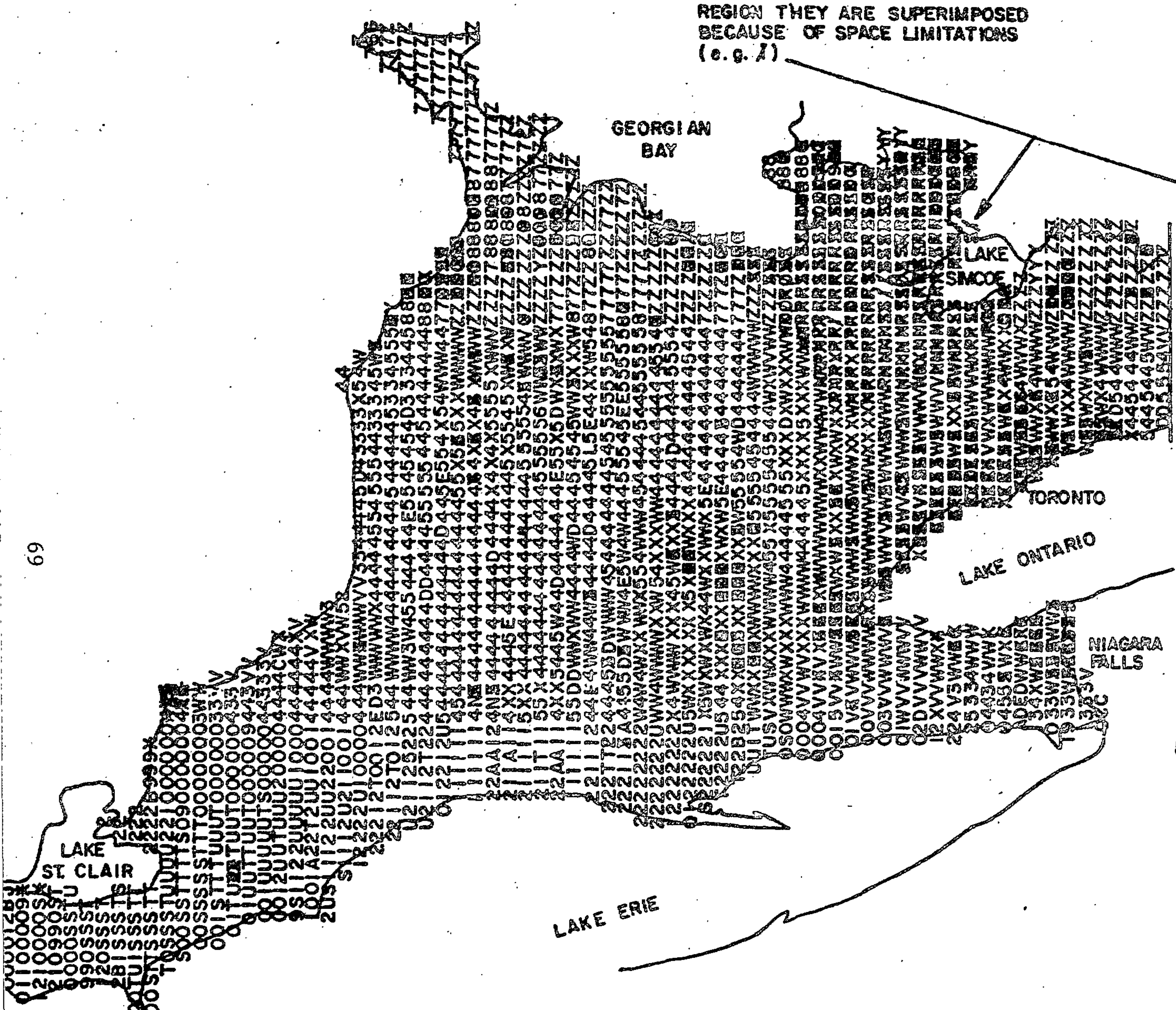
and Shade 3: $X < \bar{X} - \frac{1}{2} Sd_x$

where $\bar{X}$ is the average value of the physiographic characteristics and Sd_x its standard deviation;

d) Define the $N_r = m^n$ physiographic regions where m is the number of shades and n the number of characteristics included in the regionalization, and delineate the area of each region by overlaying the corresponding maps of the physiographic characteristics subdivided into the selected ranges.

The last operation is best achieved by means of computers using data bank techniques. An example of such physiographic regionalization carried out in Southern Ontario is illustrated in Figure 4.7 .

Once the physiographic regions have been defined, one should test the assumption that basins located in the same physiographic-hydrologic region have a similar hydrologic regime by comparing the hydrologic characteristics (in dimensionless or comparable terms such as runoff) for all basins located completely within the same physiographic region. Following this validation test, the new stations allocated are then distributed to obtain as uniform a distribution between the physiographic-hydrologic regions as possible.



LEGEND

POSITION OF L's

SYMBOL USED ON MAP

L_{BSW}	1	1	1	1	1	1	1	1
L_{ABL}	1 0	1 1	1 2	1 3	1 4	1 5	1 6	1 7
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2
L_{BSW}	1	1	1	1	1	1	1	1
L_{ABL}	2 9	2 A	2 B	2 C	2 D	2 E	2 F	2 G
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2
L_{BSW}	1	1	1	1	1	1	1	1
L_{ABL}	3 I	3 J	3 K	3 L	3 M	3 N	3 P	3 Q
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2
L_{BSW}	2	2	2	2	2	2	2	2
L_{ABL}	1 S	1 T	1 U	1 V	1 W	1 X	1 Y	1 Z
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2
L_{BSW}	2	2	2	2	2	2	2	2
L_{ABL}	2 1*	2 2*	2 3*	2 4*	2 5*	2 6*	2 7*	2 8*
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2
L_{BSW}	2	2	2	2	2	2	2	2
L_{ABL}	3 A*	3 B*	3 C*	3 D*	3 E*	3 F*	3 G*	3 H*
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2
L_{BSW}	3	3	3	3	3	3	3	3
L_{ABL}	1 J*	1 K*	1 L*	1 M*	1 N*	1 P*	1 Q*	1 R*
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2
L_{BSW}	3	3	3	3	3	3	3	3
L_{ABL}	2 T*	2 U*	2 V*	2 W*	2 X*	2 Y*	2 Z*	2 0/
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2
L_{BSW}	3	3	3	3	3	3	3	3
L_{ABL}	3 2/	3 3/	3 4/	3 5/	3 6/	3 8/	3 B/	3 C/
L_{LAT}	1	1	1	2	2	2	3	3
L_{PIN}	1	2	3	1	2	3	1	2

KEY FOR L'S

BSW =	0	<80	>80	FT x 10
L_{BSW}	1	2	3	
ABL =	0	<30	>30 (UNIT=20.KM/4)	
L_{ABL}	1	2	3	
LAT	<30	>30 & <60	>60	
L_{LAT}	1	2	3	
PIN	<200	>200 & <500	>500	
L_{PIN}	1	2	3	

Figure 4.7 PHYSIOGRAPHICAL - HOMOGENEOUS HYDROLOGICAL REGIONS IN SOUTHERN ONTARIO

(from Solomon, 172a)

4.5.5.2 Economic considerations

Two distinct sets of economic considerations have to be taken into account in connection with the distribution of the new stations within the optimized network design:

a) considerations related to the costs of installing and operating the stations;

b) considerations related to economic planning in the area covered by the network.

4.5.5.2.1 Costs of installation and operation

These costs vary with many factors, of which the most important are the size of the river, and accessibility. Since costs increase significantly with the size of the river basin, and since from the viewpoint of accuracy of information interpolation techniques it is more advantageous to have data from basins of small sizes (a few hundred square kilometers) for which the average physiographic characteristics do not display excessive variation, one should prefer, other considerations being equal, the installation of gauging stations on small rivers for the extension of the natural regime network. Furthermore, it is also obvious that from the viewpoint of the installation and operation costs, other conditions being equal, it is preferable to install stations in more accessible areas. However, in practice, there is a tendency to overemphasize the requirement of accessibility, resulting in the installation of stations too close to each other and without taking into account the requirements of regionally balanced distribution as indicated by the statistic-hydrologic and physiographic-hydrologic regions. In all cases the above considerations should take precedence over those of accessibility, the latter criterion being applied only for choosing between two locations of equal merit from the viewpoint of regionally balanced and physiographically stratified sampling.

4.5.5.2.2 Economic development considerations

This problem is analyzed in detail in Chapter 5. We note here only that in areas designated for development the emphasis may shift from the small rivers to the larger, but sampling considerations should not be neglected.

4.5.5.3 Environmental and aesthetic considerations

A significant group of factors which has to be considered in the selection of sites for hydrometric stations is that related to environmental and aesthetic considerations. As stated by Langbein (1972) "many decisions on river and reservoir development are beginning to be influenced as much by environmental factors as by hydrologic factors. There appears, therefore, to be justification for river data to include information on such matters in order that they may be considered in water resources planning in its initial stages."

The problem has two aspects in relation to hydrologic networks: a major one related to the relationship between the environment as a whole and the water resources as a part of this whole, and a minor one related to the integration of the structure and equipment of the hydrologic station and of the activity related to it into the corresponding environmental setting. The latter problem is related to actual station site selection and design and is outside the scope of this Report.

It has to be considered alongside with all other problems of the practical selection of a gauging site after the regional site was selected, such as the problem of site accessibility, section stability, etc. The only reason for which this problem was mentioned here is because it is often overlooked in practice. The first aspect is relevant to the design of the hydrologic network and deserves further discussion.

The relationship between environment and water resources has a regional as well as a local connotation. The regional connotation is related to the complex role played by water in molding the environment, e.g. its role in the determination of the vegetation and animal life of the region, erosional and sedimentation patterns, etc. The planning of a hydrologic network which should be useful for environmental assessment requires, as indicated by Langbein (1972) "an inquiry among landscape specialists, ecologists, and conservationists on the facts which are significant" from an environmental-hydrologic viewpoint. Once these facts have been selected, it might become feasible to delineate "hydrologic-environmental" regions in a similar manner as for the delineation of "physiographic-hydrologic" regions (4.5.5.1.2). Data banks and environmental-hydrologic models could also play a significant role in such regionalization. Once the regions have been delineated it would then be necessary to organize the network so that each region is gauged from a hydrologic viewpoint.

The local connotation is related to the aesthetical value of the river valley landscape, the "riverscape" to use a word coined by Leopold (1969). The local problem could be treated in a manner similar to the regional one, with the difference being that in this case the elements which have to be considered are those which in the opinion of various people have significance from an aesthetical viewpoint. Because of this, regionalization would have a subjective character. This is further complicated by the difficulty of rating these factors and attaching figures to their qualitative variation. Nevertheless, one can ascertain that there are a series of physical, chemical, biologic, and human factors which can be measured and which certainly have a significant influence on the aesthetic perception of riverscapes by human beings. These factors were listed by Langbein (1972). If one could regionalize the river valleys according to the space variation of these factors and, on this basis, set up a network to measure those factors which vary in time, the results of such measurements could be used with subjective ratings set up according to the nature of uses and user preferences to produce information needed in decision-making in this area. A technique of producing such information by means of factor analysis with application to the recreational use of reservoir shorelines has been developed by Jaakson (1970), and could also be of some use in the regionalization process. However, such regionalization currently requires expensive and time-consuming ground surveys. It is therefore suggested that research be carried out on the possibility of "environmental-hydrologic regionalization" by means of interfacing data banks and remote sensing, particularly Landsat photographic and digital data.

5. TECHNICAL-ECONOMIC APPROACH TO NETWORK DESIGN

5.1 General Design Considerations

Most hydrometric networks or individual hydrometric stations that fulfill an economic role, such as planning, design, and operation of water resources projects can be assessed from an economic viewpoint. If the benefits obtainable from the operation of such networks (stations) exceed the costs, it is generally considered that such networks (stations) are economically viable and should be installed and operated as long as this condition holds. If the situation is opposite, the networks (stations) should not be installed or, if already installed, should be discontinued.

However, the decision making process regarding stations with assessable economic value is not as simple as would appear from the above statements. In most cases, the budget for data collection is limited, and in such cases the fact that benefits slightly exceed costs may not be sufficient to justify the network (or station). Furthermore, in some cases, a station (network) may be required for technological purposes. In such case the technical-economic analysis of the station is practically the same as for the whole project, and the network (station) has to be considered economically viable if the related project is. An additional complicating factor relates to the fact that credible technical-economic calculations for individual stations are so complex, and therefore so costly, that carrying out such computations may lead to costs comparable to those of installing and operating the station for a few years. In such case, the carrying out of technical-economic calculations is obviously not advisable.

5.2 General Economic Considerations

When making a cost benefit analysis the following general economic considerations have to be kept in mind:

(a) Costs and benefits have to be present worthed to a given point in time, usually the moment when the project(s) under consideration start to operate. This requires the selection of interest rates over the relevant period (start of investigation to the end of the life of the project) (Howe, 1971). Since the interest rates vary in time and are difficult to predict, their use is usually made in the framework of a sensitivity analysis. When postponing a project for the purpose of carrying out gauging for a longer period of time, the costs of various alternatives have to be compounded by present worthing all of them to the same point in time.

(b) Costs and benefits analyses are made usually by comparing marginal costs to marginal benefits. To be economically viable a project must be selected at a point where the marginal benefit is larger than or equal to the marginal cost. The maximum total benefit is obtained where marginal benefit equates marginal cost (James and Lee, 1971) following a situation in which marginal cost has exceeded marginal benefit (Figure 5.1). The underlying assumption of this optimization technique is that the budget is not restricted.

(c) When several projects are financed from a common limited budget, the optimum allocation of the total budget among the projects is obtained when the ratio of marginal benefit to marginal cost for each project is the same (James and Lee, 1971).

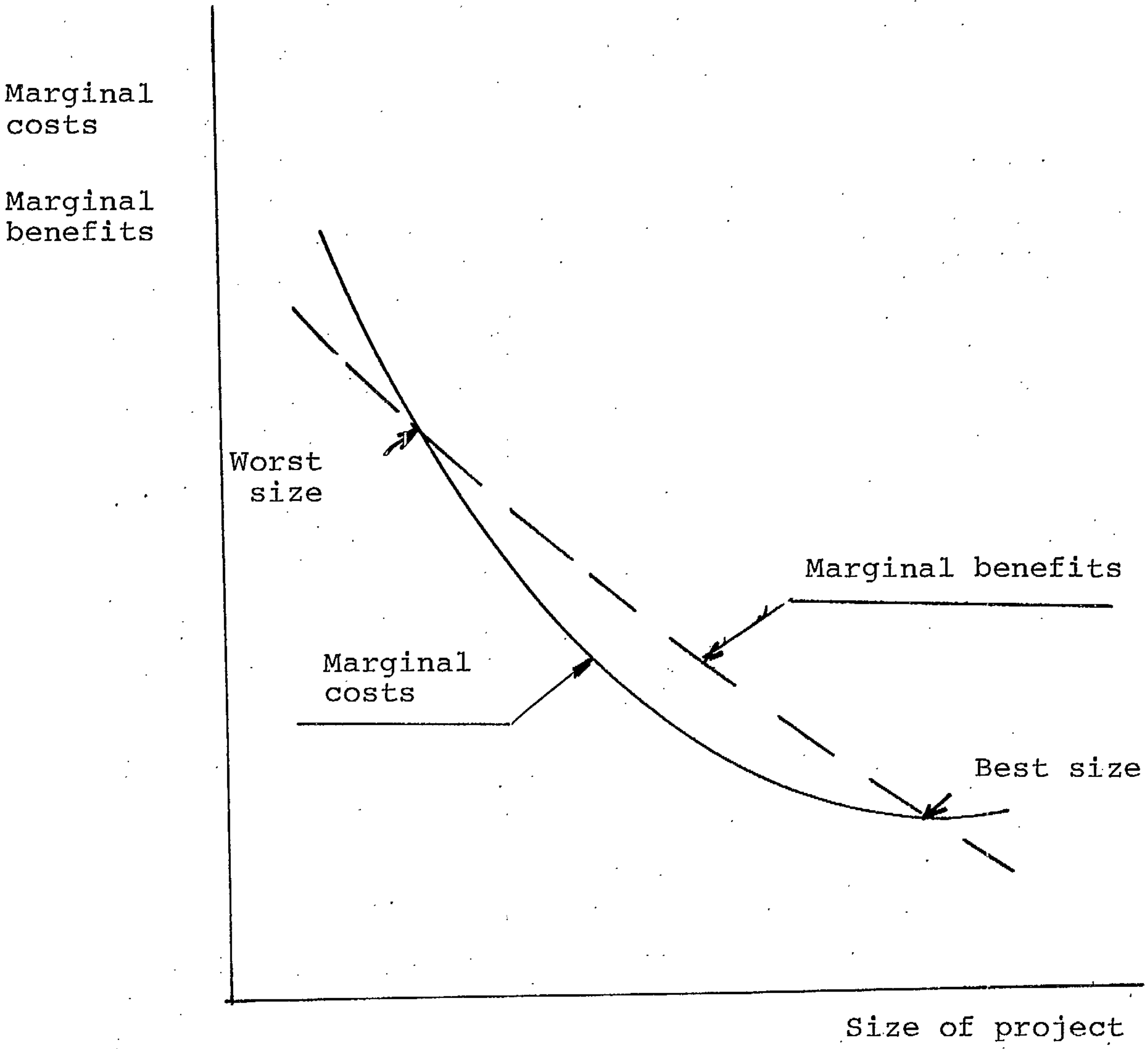


Figure 5.1 Variation of marginal costs and marginal benefits with project size

5.3 Methods of Assessment

Costs of and benefits from hydrologic information have been assessed historically using two different approaches. The first, more approximate, one uses overall figures for a country, region, or basin network in relation to the totality of water-resource projects developed in the corresponding area. The second, usually more accurate approach, involves one or several specific gauges related to a specific project, or to a set of similar projects.

5.3.1 Overall evaluations

5.3.1.1 Estimated benefits

In one alternative of the overall approach (Acres, 1977, in Reynolds 1979) the costs of the water projects are calculated including the costs of the network that have produced the required hydrologic information. Some benefits from the water-resource projects are then apportioned for each category of project to the hydrologic information according to an estimated ratio based on the nature of the project. If this resulting average benefit/cost for hydrologic data collection is larger than one, this is an indication that the investment in hydrologic information was economically acceptable. Comparison of the average benefit/cost ratio to the corresponding ratios of other economic activities, may serve as an indication of advisability of extending and intensifying the network or vice-versa.

The approach has the advantage of being simple and relatively easy to apply. The overall costs of the network are easier to calculate than those of a specific gauging station and overall costs for water-resource projects can be estimated with better accuracy than that of individual projects.

However, the approach has several disadvantages which make its application of doubtful value. The main disadvantage is the very large approximation involved in the assumption that the benefits from hydrologic data represent a given, more or less arbitrary percent, of the total benefits. In actual fact some projects may benefit much more from hydrologic information than others. For example, the design of a hydroelectric plant with very large storage constructed for reasons other than hydrologic (e.g. for obtaining flood protection) in excess of requirements for super annual storage will require data on the long-term mean only, while information on flow variation will be superfluous. For such project the benefit/cost ratio for hydrologic information may be low. Conversely, the design of a project for a run of river hydroelectric plant will benefit very much from improved information on the variation of the riverflow. The benefit/cost ratio from hydrologic information would be in this case very high. It may be argued that through the overall averaging, these variations from a mean value may compensate. It is necessary however to present evidence that the average benefit/cost ratio of hydrologic data tends to the one accepted for the given category of project. This has not been yet produced by the users of this approach.

The second significant disadvantage of the technique is related to the fact that it gives only a very qualitative indication on the benefits that would result from further improvement of the hydrologic information. Thus, it does not provide valid indications on improving network design. Furthermore, as shown by Davis et al (1979), under certain circumstances of design standards and level of hydrologic knowledge, it is possible that improved hydrologic information results in negative benefits*. This situation cannot be detected through the application of this overall approach.

5.3.1.2 Benefits estimated from error reduction

An alternative application of the overall approach (Ingledow, 1970) which eliminates some of the above problems, is based on the assumption that the benefits from increased hydrologic information Bh are related functionally to the percent standard error affecting the hydrologic parameter of interest:

$$Bh = f(Eh) \qquad (5.1)$$

Assuming that the cost $\Delta C\Delta Eh$ of decreasing the percent standard error of the parameter ΔEh can be estimated (in terms of increased frequency of measurement, ΔNm, increased number of stations in the area of interest, ΔNs, and additional number of years of operation, ΔNt) and possibly through the application of a better interpolation technique costing additionally ΔCi :

$$\Delta C\Delta Eh = f(\Delta Nm, \Delta Ns, \Delta Nt, \Delta Ci) \qquad (5.2)$$

it is possible to determine the marginal benefit as:

$$\frac{\Delta Bh}{\Delta Eh} = \frac{f(Eh - \Delta Eh) - f(Eh)}{\Delta Eh} \qquad (5.3)$$

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* This relates basically to the following conditions: For a low level of hydrologic knowledge, the standard replaces conservatively the economically justified design return period by a longer one. As the level of hydrologic knowledge increases the longer design period results in overdesign.

and the marginal cost:

$$\frac{\Delta C \Delta Eh}{\Delta Eh} = \frac{f(\ \Delta Nm,\ \Delta Ns,\ \Delta Nt,\ \Delta Ci\ )}{\Delta Eh} \qquad (5.4)$$

These expressions can be used with the general economic principles outlined in Section 5.2 and with data on planned water-resource projects in a region to determine if the extended operation of the network provides higher benefits than its costs. A similar but slightly different approach was used by GKY (1981) to estimate net benefits from the US National Weather Service forecasting network. Equation 5.2 can be estimated by analyzing network costs in areas with high network densities and long periods of record. The main difficulty in applying the technique in this form relates to difficulties of estimating the actual form of Equation 5.1, i.e. the actual benefits derived from reduction of error in the estimation of the relevant hydrologic characteristic.

Another difficulty in the application of the technique is related to the dynamic aspect of the benefit and cost equations. If increased accuracy relates to additional gauging, the benefits may be decreased because of the postponing of the construction start. On the other hand, error reduction due to an increased number of stations starts to become of significance only after a number of years (when the error of interpolation at the given points becomes larger than the error of time sampling, Matalas and Gilroy, 1968). These difficulties can however be circumvented when the method is applied in the framework of dynamic programming (Solomon, 1976).

5.3.2 Individual evaluations

5.3.2.1 Non-Bayesian techniques

The value of hydrologic data from individual gauging stations was considered initially in studies by Moss (1970), Dawdy et al (1970), and Shawinigan (1970). The latter study was also reported by Reynolds (1979). In these types of studies the length of record of a station is optimized in terms of the net benefits obtained from the data for designing a specific project. At the basis of this type of studies is the definition of a cost - error function translated in a cost - length of record function and a benefit - error function from the project designed with the given data also reduced into a benefit - length of record relationship.

The cost - error function is translated into a cost - length of record function by theoretical statistical equations calibrated on the basis of long streamflow records. The benefit - error function is reduced to a benefit - length function by simulating a very long period of record (T_L) which is split into smaller periods of records (T_S). The latter are then used separately for design of the project considered and the resulting benefits B_S compared with the one that could result from the very long period of record B_L. The difference $\Delta B_S = B_L - B_S$ is then attributed to the additional period of record $\Delta T_S = T_L - T_S$.

Since both costs and benefits are finally expressed in terms of record length, marginal costs and benefits can be calculated (i.e. incremental costs and incremental benefits from one additional year of record). Of course, in such calculations benefits foregone from postponing development and increased carrying charges for the studies have to be considered when applicable. The optimum length of record is obtained when marginal benefits equate marginal costs.

The technique has two major drawbacks. The first relates to economics, namely to the fact that it is applicable only to the case when the budget is not constrained both for the stream-gauging activity and for the water-resource project considered. This difficulty could be circumvented if the analysis is done simultaneously for all stations and projects covered by the given limited budget. In this case, as indicated in 5.2, the benefit/cost ratio for all stations* (and all projects*) has to be the same and is defined using the Lagrange's multipliers technique (James and Lee, 1971). The second drawback is related to statistical - hydrologic aspects of the technique. In the technique described above the basic implicit assumption is that the long-term synthetic record is the only possible ("true") one. Of course this is not correct. If a number of samples of flow records at the same hydrologic stations would be generated, reflecting the many uncertainties involved, the results of the calculations could be completely different for each separate record. The Bayesian approach discussed in the next section addresses itself to this last problem.

5.3.2.2 Bayesian approach

The Bayesian approach is used to introduce decision theory techniques into the process of estimating the value of additional hydrologic data in water resources decision making.

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* The ratio is different for the stations and projects if they are financed from two different budget items.

Omar (1980) discusses the principles of applying decision theory to decision making in the use of agrometeorologic forecasts. The same principles apply to hydrologic forecasts.

The Bayesian approach to the estimation of value of hydrologic information was initially introduced by Davis (1971) and Davis and Dvoranchik (1971). In this approach one considers a set of initial data which can be either a time series or a probability distribution (<u>the prior</u>). The prior is used to design the water-resource project under consideration. The prior reflects either data available or the general hydrologic knowledge of the hydrology and may include subjective assessments. If one waits for a certain period of time to collect more data, the time series (or probability distribution) is modified accordingly and this constitutes the <u>posterior</u>. The changes in the probability distribution introduced by the new data into the initial distribution can be calculated using Bayes' Rule, and this would improve the hydrologic information. Assuming that the design based on the posterior is improving over the initial design, the difference can be attributed to the improved hydrologic knowledge gained by additional gauging. Of course, as in the previous technique any postponement of a project for purpose of gauging increases the present worth of the cost of the data and reduces the present worth of the benefits. The difference between the additional benefits due to a better design and additional costs related to extension of the gauging period is called in Bayesian terminology expected opportunity loss (XOL). The optimum decision would be the one minimizing XOL.

However, at the point of decision making XOL is not known, because it is not possible to know the posterior distribution (time series) before actually carrying out the additional gauging. This difficulty is overcome by considering all possible outcomes for additional measurements and posteriors corresponding to them that are compatible with the given uncertainty of the design parameter(s) x(y,z...). This is called <u>preposterior</u> analysis. The XOL for various values of the parameter x are calculated as a weighted sum (integral) called expected expected opportunity loss (XXOL) in which the weights are the probabilities of occurrence f(x) of various values of x:

$$XXOL(x) = \int_x XOL(x)f(x)dx \ .$$

If two parameters are involved the integral becomes a double one:

$$XXOL(x,y) = \int_x \int_y XOL(x,y)f(x)f(y)dxdy \ .$$

The decision regarding additional length of measurements is taken on the basis of minimizing XXOL. This also provides a measure of the value of additional data which is the difference between the XXOL based on no additional data and the XXOL based on the assumed extended period of record. Under certain assumptions, the calculations described above can be carried out analytically. Currently, this can be done when the following assumptions are acceptable:

(a) there is only one design parameter (and one purpose for the data);

(b) the type of distribution of the design parameter is known;

(c) the XOL for the given design parameter can be defined analytically and is a relatively simple (linear, parabolic) function of the error of its estimate.

Lall and Beard (1981) have developed this technique under the above mentioned assumptions. However, because of these assumptions, the practical value of such development is in doubt.

In practice there are several hydrologic parameters that influence the design of a water-resource project. In addition there is great uncertainty with respect to the distributions of these parameters. Furthermore, there is supplementary hydrologic uncertainty related to the non-stationary character of the hydrologic data. In most cases the development of a water-resource project itself is responsible for the introduction of non-stationarity in the hydrologic data (over and above that related to operational effects). On the other hand there is uncertainty with respect to the design, losses and benefits, etc. Introducing all these uncertainties in the analytical calculations makes them analytically intractable. The huge number of possible combinations of uncertainties makes the use of numerical calculations impractical even with the current availability of extremely efficient computers.

Moss and Dawdy (1980) have combined the Bayesian approach with Monte Carlo techniques. They are synthesizing time series of data and thus overcome the need of multiple integration for all uncertain hydrologic parameters and distributions that may affect the design. In their technique the uncertainty regarding these parameters is built into the simulation technique. The calculation of XXOL is based in this case on summation over the results of various samples. The accuracy of summation can be increased by relating graphically various design parameters to the variation of the hydrologic parameters of interest. When applied in practice, however, Dawdy and Moss did not include the economic uncertainties (related to costs and benefit evaluation).

In spite of this limitation the technique comes probably closest to the one required to reasonably assess the value of hydrologic data. Its cost in terms of computer time is however very high, and because of this, its application should be limited to a number of typical case studies and to comparisons with the simpler techniques such as the one described in 5.3.2.1 .

The Bayesian approach can also be used to assess the effects of set standards on the design of water-resource projects. This has been discussed by Davis et al (1979). Section 5.4 describes some results of the application to actual data of the techniques described in 5.3 .

5.4 Case Studies

There is a limited number of case studies on cost/benefit analysis of hydrologic information published in the literature or in some reports on the value of hydrologic data and/or network design. The methodological background of these case studies is related to the techniques described in the previous section. Therefore, the methodology used in each case study will be only referred to Section 5.3 and relevant references.

All case studies have explicit or implicit assumptions that greatly reduce their usefulness. However, a large sample of such studies are presented here because on one hand they reflect the current practice, and on the other could serve as a basis of comparison with more realistic techniques.

5.4.1 Case studies using overall evaluations

5.4.1.1 Canadian Network (Acres, 1977)

Acres, in a study described in some detail in Reynolds (1979), used the overall evaluation technique based on estimated benefits in cost/benefit analysis of hydrologic data for the whole of Canada. They used in their study projections of expenditures in water-resource projects over a period of one year (1977) based on extrapolation of expenditures on a regional basis observed over the 5 year (1972 - 1976) period preceding the study. Acres introduced a further simplification in this analysis when estimating the benefits from hydrologic information. The benefit estimation is done by assessing for each type of project the percentage of initial investment cost that is sensitive to hydrologic information. In turn a certain percentage of this portion of investment was considered to be equal to the benefit from the hydrologic data. For example, for hydroelectric power development Acres estimated that 65% of the initial investment is sensitive to hydrologic information, and that 5% of this latter investment represents benefits from hydrologic data.

The total benefit indicated by the application of the technique in this manner was estimated at Can.$ 129.6 x 10^6/year. The cost of operating the network at that time was about Can.$ 15 x 10^6/year. The high benefit/cost average ratio could be considered as an indication that the network could further expand. However, because of its many simplifications and assumptions the technique cannot be considered as realistic. It is however interesting to note that the NWS - Northwest Forecast Center - made in 1981 a cost/benefit analysis

(Unpublished Internal Report) using the same methodology (NWS - Northwest Forecast Center, 1981).

5.4.1.2 Atlantic Provinces (Canada) Network (Ingledow, 1970)

Ingledow in 1970 used the overall approach based on benefits estimated from error reduction and its influence on designing water-resource projects in the Atlantic Provinces of Canada. To estimate the cost of error reduction they used an expression for percentage standard error of estimate of a given hydrologic parameter of the form:

$$E^2 = \left[\frac{k_s A}{S} \right]^2 + \left[\frac{k_t}{T} \right]^2 + Em^2 \qquad (5.5)$$

where

k_s and k_t are coefficients related to error of interpolation and error of time sampling respectively;

A is the total area of the region for which the overall estimation of value of hydrologic information is carried out;

S is the number of stations included in area A;

T is the average number of years of record of the gauges in area A;

Em is the error of measurement for the given hydrologic parameter (larger for extreme flows, smaller for average flows, and estimated from the operational conditions of the network).

The reduction of E can be obtained by increasing S or T or both. It is noted however that when S increases the regional average value of T decreases and that if the resulting $(k_t / T)^2$ terms increases more than the decrease of $(k_s A / S)^2$ it is preferable to ignore the new stations for a time.

For the relation between benefits and error Ingledow assumed the following relationship:

$$\Delta C = mnE^2 \tag{5.6}$$

where ΔC is the percentage increase on cost due to the percent standard error E of estimate of the corresponding pertinent hydrologic parameter, and m and n are coefficients depending upon the type of hydrologic parameter and type of project.

Instead of attempting an optimization, Ingledow assumed a certain network expansion (at various rates, lower in well-gauged provinces, higher in those with scarce networks) and calculated the costs of such expansion. For each province, they calculated the portion of investment in water-resource projects for a 20 year period sensitive to errors in various hydrologic parameters. The reduction in error obtainable for the assumed network expansion and the corresponding reduction in cost of the investment sensitive to errors affecting various hydrologic parameters was then estimated. This was considered to represent the benefit from the network. The results of the study show that the overall benefit/cost ratio for the operation of the network over the 20 year period 1975 - 1995 varies from 2.43 for the Maritime Provinces, a well-gauged area, to 21.20 in Labrador, a scarcely gauged area. Another interesting result is that the benefit/cost ratio shows a significant increase from the first to the second decade. This is due to the fact that investment in the network in increasing the number of stations in the first decade, produce benefits mainly in the second decade, when the error of time sampling becomes less than the error of interpolation.

5.4.1.3 Canadian Network (Solomon, 1976)

Solomon has used a similar procedure but in the framework of a dynamic programming algorithm. The objective of that investigation was the maximization of the net benefits from the total network. The maximization was obtained by varying the number of new stations and timing of their installation. The investigation covered the whole of Canada, and the results are summarized in Table 5.1 .

5.4.1.4 US National Weather Service - Worth of hydrologic data for short-term forecasts of floods (Smiedovich et al, 1973)

A series of studies on the above subject were commissioned by the US NWS in 1973 (Smiedovich et al, 1973) and extended later to the problems of the response to the forecast warning (Smiedovich et al, 1975). The Bayesian framework discussed in 5.3.2.2 was used in these studies. However, the complexity of the problem when the technique is being applied to an overall evaluation prevented the study from reaching any numerical conclusions in the worth of such data. It is presumed that the study discussed in 5.4.1.5 was commissioned with the purpose of using a simpler approach to the solution of this problem.

TABLE 5.1

Results of Optimization by Dynamic Programming of Simulated Canadian Natural Regime Network for the 1971 - 1985 Period

(All values in millions of dollars, present worthed to 1971, except net benefit per dollar which is dimensionless and decision variables which have the stated dimensions, N_1 = 2000 stations, interest rate: 6%)

	Results for: the 1971-75 period only	the 1971-80 period only	the 1971-80 period only
Losses due to errors in standard deviation of mean annual flow assuming that hydrological network ceased operation in 1971	766	1667	2663
Losses due to errors in standard deviation of mean annual flow, assuming hydrometric network operating at optimum level	659	1240	1773
Cost of operating the network at optimum level	36	86	126
Net benefits	71	341	754
Net benefits per dollar spent for hydrological network operation (%)	197	396	599
Decision variables: Number of supplementary stations required at end of period	1475	4463	6420
No. of stations to be discontinued in this period	0	253	2911
No. of flow measurement per station/year	8	10	18
Error of standard deviation of annual flow at end of period (%)	18	16	15

5.4.1.5 US National Weather Service Forecasting Network (GKY, 1981)

GKY & Associates, a consulting firm, has carried out a study on the value of hydrologic forecasting networks for the US National Weather Service (NWS). The objective of the study was to develop estimates of those flood forecasting benefits associated with earlier flood warnings over the current or expanded area benefiting from this flood warning. The study used Monte Carlo simulation techniques to estimate probabilities of various benefits and their expected values. Several alternatives available to NWS for increasing the warning lead time were examined. Eight scenarios which were combined in three alternatives were examined. Only benefits from increasing the warning lead time were examined. The effect of accuracy of the level forecasted was not analyzed. The equation for the effective lead time (TE) used was:

$$TE = 0.75\,T - (T1 + T2 + T3) \qquad (5.7)$$

where

T = basin lag time (time of concentration);

T1 = time required for dissemination of information ($T1 = 1.29\,A^{.428}$; A = basin area, sq. mi.);

T2 = time required to carry out the forecast;

T3 = delay time in reporting rainfall measurements.

The relative standard error affecting the lead time (Et) is estimated from the following experimental relationship:

$$Et = 8.2\,T^{-.22}\,A^{-.302}\,GE^{.602} \qquad (5.8)$$

where $GE = \frac{NE}{A}$ with NE = effective number of rain gauges reporting in the given basin.

Damage D was estimated using a relation of the form:

$$\log_{10} D = a + b \log_{10} A + \varepsilon \qquad (5.9)$$

where a and b are coefficients varying from one river forecasting center to another, and ε a normal variate with mean 0 and a standard deviation also varying from one forecasting center to another.

Percentage damage reduction Dr (%) is obtained from a curve relating it to effective lead time TE (error free), and a penalty for relative error Cv in TE is being applied using the relation:

$$Dra = Dr\,(1 - 2\,Cv) \qquad (5.10)$$

with Dra being limited to 0 for $Cv > 0.5$.

The standard errors of various parameters were also obtained from experimental data. On this basis it was possible to design a Monte Carlo approach to the estimation of increased benefits and calculate their expected value for each alternative and sub-alternative. The results of these calculations in terms of incremental benefits of going from one scenario to another were then summarized. The additional costs were unfortunately presented only in terms of the physical elements to be acquired in the framework of the scenarios involved and not as monetary values. Therefore the final conclusions were only of a qualitative value and will require further calculations to provide the ratio of incremental benefits over incremental costs.

5.4.1.6 Lake Champlain Richelieu Forecasting Network (Reynolds, 1982)

A recent study by the International US - Canada Joint Commission discussed by Reynolds (1982) in his presentation at the 1982 WMO workshop on cost/benefit analysis of hydrologic data provides some further indication on the benefit/cost ratio from a flood forecasting system in a given area. The study dealt with the International Lake Champlain where annual damages amount to $ 7.5 x 10^6 ($ 4 x 10^6 on the US side and $ 3.5 x 10^6 on the Canadian side). Flood warning may reduce damages by $ 473,000/year with a benefit/cost ratio of 4.26, higher than any other possible individual flood reduction measure or measure combination.

5.4.2 Case studies of individual evaluations

The overall evaluation case studies presented in 5.4.1 were geared mainly to strategic decisions, i.e. on direction to move on with respect to budgets and major planning. The individual evaluations can be used for tactical decisions on where and when to install new stations or delete existing ones, and even on operational details.

5.4.2.1 The Arroyo Seco near Soledad, Colorado (Moss 1970, Dawdy et al 1970)

This case study was used by Moss, and Dawdy et al, in their complementing papers to illustrate the simulation, non-Bayesian approach to the cost/benefit evaluation of hydrologic data discussed in 5.3.2.1 . As indicated there, they investigated the design of a reservoir on the Arroyo Seco River near Soledad, Colorado for which they generated synthetically (from the available record) a 500 year record which was subdivided in samples of various lengths. The design benefits from the reservoir design on the basis of various lengths were compared among them to assess the additional benefits obtainable from the longer periods of record. The costs of additional gauging were subtracted from added benefits obtained from a longer period of gauging to estimate net benefits from additional gauging. Two designs were considered: one in which the "safe" yield was equal to 50% of the mean, the other, in which it was equal to 70% of the mean. The results of the calculations indicate that the most significant additional benefits per year of record are obtained when the record length increases from 10 to 25 years. They also show that benefits increase when the design involves the higher yield. The very high additional benefit/cost ratio obtained seems to indicate that the values obtained in the overall studies are conservative.

5.4.2.2 Milk River Irrigation Scheme (Shawinigan 1970, Reynolds 1979)

In a study carried out by Shawinigan Engineering (1970) along similar lines as the one discussed above, the time series synthesis was carried out and by generating sequences with well defined built-in errors in the mean and the standard deviation. Benefits foregone because of erroneous design of the reservoir required for the irrigation scheme (correct design was considered to be that corresponding to the historical data) were compared to costs of improving the accuracy of the mean or the standard deviation. It was concluded that benefit/cost ratios are very high. For the given case study in which the main design parameter was the storage volume required to obtain a given yield the error in the variance had a much higher influence on the benefits foregone than the error in the mean.

5.4.2.3 Hypothetical optimization of operation of a reservoir (Klemes, 1977)

Klemes (1977) used a hypothetical reservoir and synthesized time series of data to estimate the value of hydrologic information. He arrived at the following conclusions with respect to the significance of hydrologic information:

a) An optimal policy (optimized by explicit dynamic programming) is largely insensitive to the substitution of a normal distribution to the "correct" log normal distribution.

b) The relative value of an information increment has been found to decrease rapidly with the increase in the total amount of hydrologic information. The value of information contained in a single value input (one year) amounted to about 45% of the value of perfect information and for n = 5 (years) the percentage rose to about 75%.

c) The distribution of the relative values of hydrologic information has been found to be highly negatively skewed. Accordingly, the actual loss in an individual case can not be much lower than the average but can be much higher.

d) An increase in economic uncertainty, reflected in a progressively higher "risk premium" added to the value of the discount rate has a similar effect to an increase in hydrologic uncertainty: both reduce the gain achievable by policy optimization and by a given reservoir size.

e) The degree of uncertainty inherent in any level of hydrologic information can be expressed by an equivalent discount rate; this makes it possible to compare quantitatively the hydrologic and economic uncertainties.

f) The expected economic gain achieved by a given increase in reservoir storage capacity was found to be proportional to the level of information (inversely proportional to the level of uncertainty) used for policy optimization.

g) The degree of optimality of a given policy was found unrelated to the degree to which the underlying input model and its parameters agreed with those of the parent population, but was strongly related to the closeness of the historic series (used to define the policy) to the future series (in other words, real time flow forecasting is much more important than stochastic simulation for reservoir operation).

5.4.2.4 Groundwater quality of coastal aquifer in Israel

Ben Zvi and Bachmat (1982) presented at the 1982 WMO workshop on cost/benefit analysis of hydrologic data a case study involving the estimation of the value of sampling the salinity of a coastal aquifer in Israel. By making a number of simplifying assumptions Ben Zvi and Bachmat arrive at the following expression of the total cost to the user and to the network operator:

$$F(n,k) = \frac{\lambda_1 + \lambda_2}{\sqrt{2\pi}} \left[\frac{1}{k} (\sigma_1^{\,2} + \sigma_2^{\,2}/n) \right]^{\frac{1}{2}} + \left[C_0 + C_1 n \right] k \qquad (5.11)$$

where:

the first term in the right hand side represent the losses related to the total error of estimating the salinity, and the second the cost of carrying out the sampling program;

n is the number of samples per year;

k is the number of years of sampling;

λ_1 and λ_2 are loss coefficients assumed to be obtained from experience;

σ_1 is the observed standard error of estimate due to intra-annual variation;

σ_2 is the observed standard error of estimate due to interannual variation;

C_0 is the annual cost of the program in addition to the costs of sampling and analysis;

C_1 is the unit cost of sampling and analysis.

This expression can be readily minimized by equating the partial derivatives of F in terms of n and k to zero. Assuming an unconstrained budget and that λ_1 and λ_2 are known, this provides a solution to the design and operation of the network. In the more usual case of a constrained budget which is less than the optimum one, the technique reduces to the general approach, i.e. minimization of the error of estimate for a given budget. According to Ben Zvi and Bachmat the optimization of the same program for λ_1 and λ_2 known leads to significant savings. However, the simplifications and assumptions made reduce the practical value of the study, when other applications are considered.

5.4.2.5 Value of water level accuracy for transportation

Van der Made (1982) presented at the 1982 WMO workshop on cost/benefit analysis of hydrologic data a case study on the evaluation of the value of increased accuracy of water levels for commercial navigation on the Rhine. By comparing interpolated and measured water levels, he arrives at the conclusion that the reduction by 1 cm of the standard error of estimate of water level on the Rhine (in the Dusseldorf-Rotterdam reach) represents an annual benefit of Dfl 400,000. This is due to the possibility of increasing drafts of commercial ships when the water level is known more accurately. Van der Made uses a relationship between error reduction and benefits to minimize the total cost of network (network operation + loss because of inaccurate information) leading to an optimum design for navigation. It is noted that van der Made's calculations are based on actual levels, whereas the user requires forecast data for the length of the trip to make decisions about the admissable draft. His calculations lead to high benefit/cost ratios in spite of a very high cost of operating a level gauge (Dfl 150,000/year).

5.4.2.6 Rillito Creek flood control (Davis et al, 1972)

Davis et al (1972) used a Bayesian analysis of the value of data for the design of a flood protection of the flood plain of Rillito Creek in Tucson, Arizona. They applied the technique described in 5.3.2.2 to decide the level of flood protection to adopt and, in the process, to estimate the value of additional information (time) and of additional gauges (space). They used as prior the actual values of the floods recorded up to a certain point in time. In all cases they evaluated if it would be worthwhile to extend the gauging period by one year only, before making a decision. This limitation to one year extension of the gauging period was introduced because evaluating a delay of more than one year makes the numerical integration procedures required by the Bayesian approach intractable. The results of the analysis indicate that over a period of 20 years there is no significant change with time in the sample mean, but the sample variance increases by more than 200%. (This is a characteristic of hydrologic (dynamic) data that may be encountered in many practical situations and may be due to extraneously induced non-stationarity or to the inherent variability of the data. Techniques based on the assumption that variance decreases with sample length are therefore not always applicable to hydrologic data.)

The decision on protection level is unstable and changes with sample length. The expected opportunity loss (XOL) and the expected value of sample information (XVSI) tend to decrease with sample length but not in a smooth manner. The expected net gain of sampling (XNGS) has occasionally (2 out of 10 cases) a negative value, indicating that Bayesian calculations based on one year of additional data may lead to doubtful results. Moreover, as indicated by Davis et al this is due more to the conservatism in calculating XVSI than to the actual lack of value of the data. The general trend in XNGS indicates that high benefit/cost ratios can be expected from gauging up to a period of 20 years, and that this ratio decreases thereafter. This contradicts basically the results obtained looking at one year at a time, when on several occasions the indication is to discontinue the gauging.

Davis et al extended their analysis to include the sensitivity of the results to uncertainty in construction costs and to risk aversion. However, as pointed out by Davis et al themselves, the Bayesian technique is not only limited in its practical application by the fact that it is not computationally feasible to consider the additional value of sampling for periods of over one year, but also by the fact that uncertainty in distribution of data, on the design of the project, and on the economic response are most difficult to be included in the analysis. Moreover, they point out that the worth of data obtained by the Bayesian approach is strictly limited to the specific project confronting the decision makers. In actual fact the benefits extend almost always to a number of users.

5.4.2.7 Benefit and cost analysis from flood forecasting in the Susquehanna River, USA (Day, 1973)

This study analyzed potential benefits and costs from a flood forecasting system in the Susquehanna River (Day, 1973). It was based simply on the evaluation of economic consequences resulting from man's activities in a flood plain without and with a flood forecast. Although the study does not provide indications on the value of incremented information on forecasting, it gives benefit/cost ratio for a forecasting system as a whole, when only damages to residential property is considered. Such ratios range between 3.2 and 7.3 . The response of the flood plain resident to flood warning constituted an element of uncertainty which could not be evaluated. Considering that industrial damages were not included (note that the Three Miles Island Nuclear Plant is in the flood plain of the Susquehanna River) these ratios are probably very conservative.

5.5 Conclusions for Environment Canada

Environment Canada should be interested mainly in the overall network studies. It is neither advisable nor feasible to carry out technical-economic studies for each individual station. Therefore the individual stations study are relevant to Environment Canada only to the extent to which they treat typical situations.

It can be seen from the review of the case studies that in practically all cases it was concluded that the benefit/cost ratio of hydrologic studies for planning, design, and operation of water resources related projects appears to be very high. However, user's involvement is required to assess the benefits. If the users do not pay for the network, they may be tempted to overstate the demand for hydrologic data.

Overall techniques of estimation have been applied with many simplifications and assumptions which make their results less factual. Removal of some of the assumptions and/or sensitivity analysis will be required to make the results of greater practical value.

The overall estimations are applicable to general decisions about designing network development and planning the corresponding budgets in areas considered for development.

The individual estimations require either great simplifications of inputs to maintain theoretical integrity, or significant theoretical concessions, to maintain the application closer to practical conditions from the viewpoint of data inputs. Combining Bayesian techniques with Monte Carlo synthesis explicitly (Moss and Dawdy, 1980) or implicitly (Klemeș, 1977) seems to be one method of overcoming this dilemma, but this has not been yet fully demonstrated in practice.

The individual estimation techniques are applicable to tactical decisions regarding the installation or discontinuation of specific gauges, or changes in their operation program. Such analysis could be replaced by testing the willingness to pay of the users. However, the users should be exposed to the methodology and case studies of technical-economic analysis of the value of hydrologic data.

There are no case studies of cost/benefit analysis of additional hydrologic information based on the analysis of actual projects in operation for some time.

6. ROLE OF ADVANCED TECHNIQUES IN IMPROVING ESTIMATED HYDROLOGIC INFORMATION

6.1 General

All four major sources of errors in estimated hydrologic information (technique of interpolation, length of sampling period, station distribution in space, and measurement and processing procedures) can be partly eliminated by using various advanced techniques. Although some of these techniques actually relate to the operational stage and therefore are outside the scope of this Report, they are briefly discussed in terms of their indirect effect on network planning.

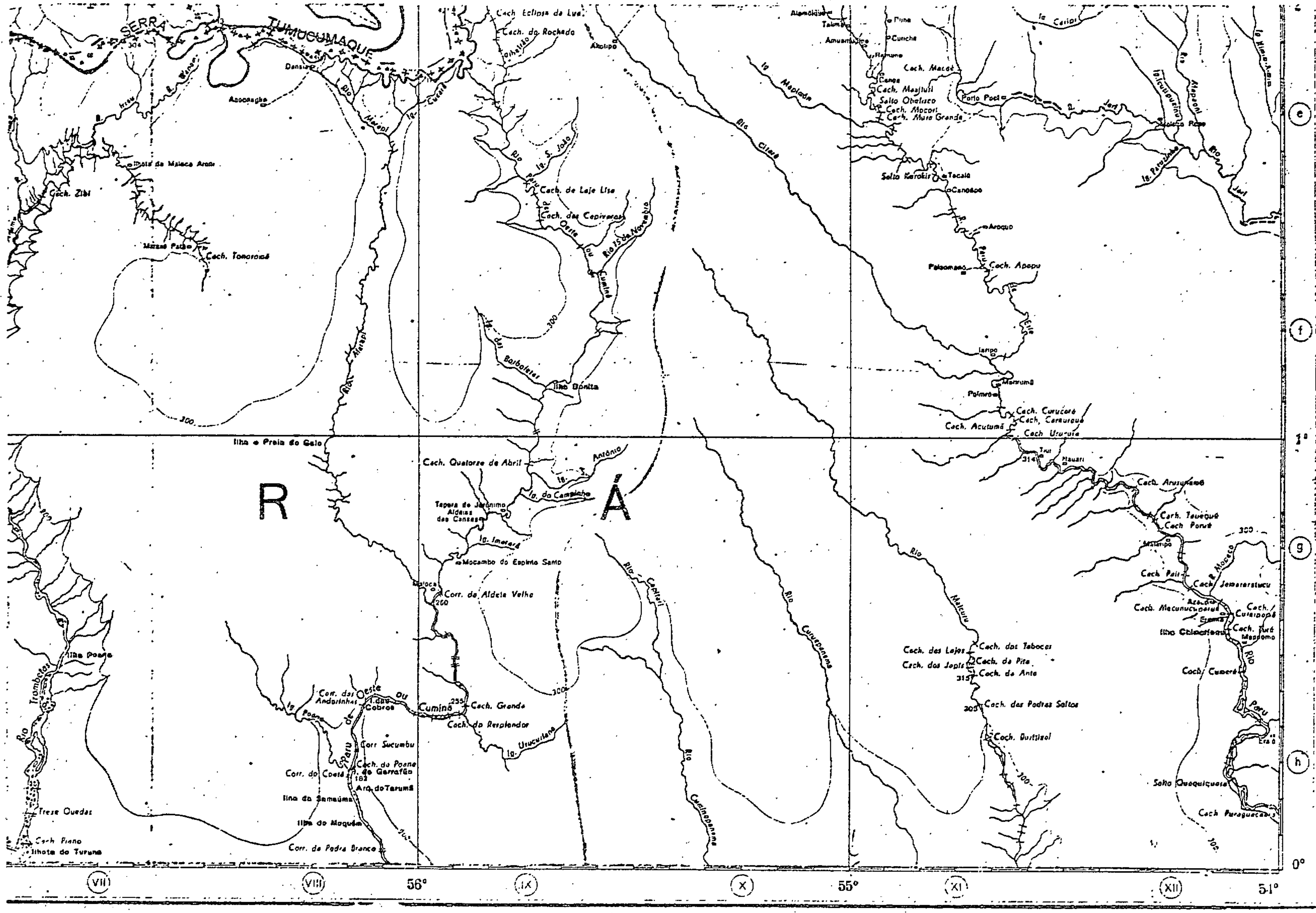
6.2 Inputs to Models of Data Interpolation

The most important possibility of reducing by means of advanced techniques the error due to the interpolation method resides in the use of remotely sensed data to improve the availability and accuracy of land-use land-cover data.

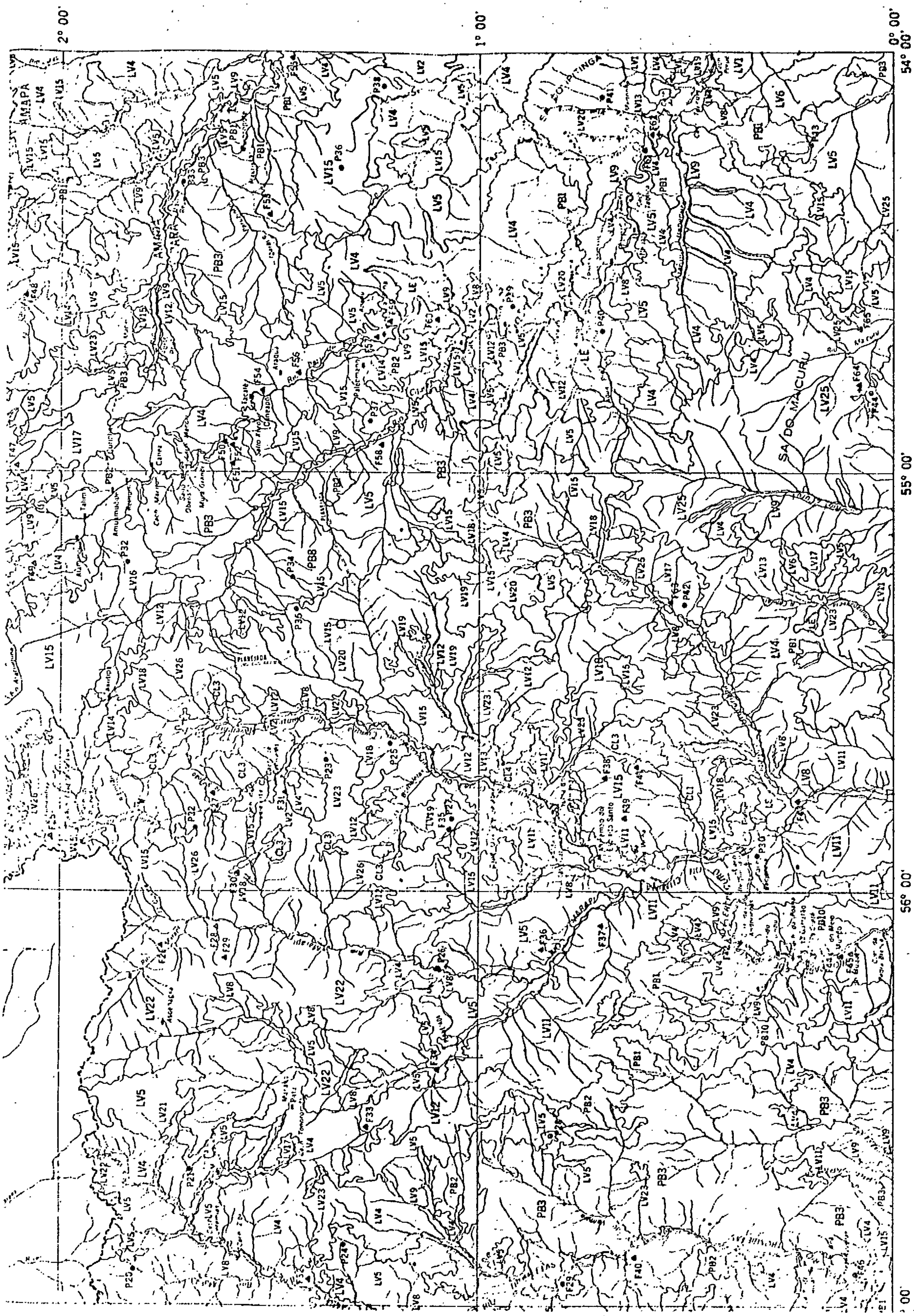
It is generally recognized that the delineation of the drainage network and related basin area, which is one very important input in any model, is a difficult task in areas of dense vegetation. In such areas it is probably required to use radar imagery to obtain a reasonable delineation of the drainage network. Figure 6.1 (WMO ., 1979) shows the significant differences that may be observed between drainage networks delineated using conventional techniques (Figure 6.1a) and the corresponding one from radar imagery (Figure 6.1b).

Figure 6.1

Comparison between river networks as shown in 1/1,000,000 topographic (a) and RADAM (b) maps in the upper Purus-Trombetas region. Note particularly differences in the map representative of the Urucuriana, Cuminapanema, Curuapanema and Maicuru Rivers.



a



b

Another important input in practically any model of data interpolation is the land-use land-cover map of the basin. Although such maps are available generally in Canada, these are not always updated. Landsat can be used to produce relatively accurate land-use land-cover maps showing the broad classes of interest in hydrologic modeling (water, forest, agriculture, urban, barren). Figure 6.2 (in pocket) provides an example of a 1:100,000 land-use land-cover map obtained from computer classification of digital Landsat data (Solomon & Associates, 1981). Of great interest in this respect is the fact, as demonstrated by Harvey and Solomon (1983), that GOES visible digital data can be used to provide land-use land-cover maps with the same accuracy as Landsat photographic imagery interpreted visually, or as conventional topographic maps (for the classes shown on such maps).

Numerous other uses of remotely sensed data obtained by satellites in hydrometric work are possible (Wiesnet et al, 1979, Deutsch et al, 1979). However an analysis of all these potential uses is outside the scope of this Report, and should be considered in the framework of a special investigation.

6.3 Increasing Length of Period of Record

The cost of obtaining data from a station for a given period of time consists of installation and operation and maintenance costs. The equipment cost represents a fixed initial investment, which can be considered independent of the period of time during which the station is operated. The operation and maintenance costs are directly proportional to this period of time. Costs of obtaining data from a station for a longer period of time could be substantially reduced if the operation and maintenance costs could be diminished accordingly. This could be achieved by locating the stations upstream of a natural (or artificial) control to obtain a stable stage - discharge relationship, streamlining the routing of the measurement teams, using more efficient measurement techniques (see for example Schneider and Billings, 1982), and reducing the number of visits to the site. Procedures developed at the USGS for obtaining some of these results have been described by Moss and Thomas (1982).

During the last few years data collection platforms (DCP's) have been installed in various countries, particularly in the USA and Canada. Contrary to expectations the cost of operating such stations have been larger than that of conventional equipment. Discussions with the USGS Chief Hydrologist indicate that this is due mainly to the fact that for these stations any outage is known instantaneously by those responsible for their maintenance and trips to replace the faulty equipment are immediately organized. While this ensures the almost continuous operation of all sensors, the cost of maintenance becomes very significant. It is suggested that DCP's should be used extensively, but in pairs with redundancies both for sensors and radio transmission equipment. The stations should be maintained by replacing faulty equipment only when measurement teams are visiting the station. The redundancy will reduce significantly the outage periods. However, when outages occur, they should be treated as those at conventional sites, i.e. by filling in gaps by correlation with other stations. The possibility of having the data from the stations directly in the computer system, coupled with stable stage - discharge relationships (or with stage - discharge relationships modelled approximately - Moss and Thomas, 1982), could reduce significantly the annual cost of operation, maintenance, and data processing, and thus enable the operation of each station for a longer period of time for the same total budget per station.

Another measure that should be considered is that of obtaining data compaction (Paulson et al, 1982). This means basically elimination of data that do not show significant difference (i.e. larger than a given tolerance limit) from data recorded prior or subsequently to their occurrence.

This will significantly reduce the amount of storage and processing required for each station and make possible the handling of each station for a longer period of time.

6.4 Station Distribution in Space

In Section 4. it was indicated that gauging stations should be distributed in such a manner as to avoid having too many stations in one physiographic region and too few in another. A large number of stations within one region leads to collection of redundant information. A minimum of 3 - 4 stations per region was recommended, as this would ensure on one hand the gauging of intra-regional variation, and the possibility of filling in missing data resulting from station outage by means of cross correlation with other two stations. The technique of delineating physiographic regions was outlined in Section 4. Although this technique is readily applicable when square grid data are available, as is the case in almost all of Canada, its application may lead to approximate results in areas of large variability in physiography, because the grid data are in most areas obtained on large squares of 10 x 10 km. It is very likely that useful elements for physiographic regionalization can be obtained from satellite imagery, particularly Landsat and GOES. While most relevant information would be that obtained from cloud free imagery, it is likely that cloud information could also be used to a certain extent. Figure 6.3 presents a Landsat image showing some features of various physiographic regions in Canada as an indication of the possible use of such imagery for the purpose of physiographic regionalization.

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