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Alternative Solutions to Water Resource Development – A Case Study

D.R. Basco
K.M.A. Rahman

Texas Water Resources Institute

Texas A&M University

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ALTERNATE SOLUTIONS TO WATER RESOURCE DEVELOPMENT -- A CASE STUDY

Principal Investigator

David R. Basco and K. M. A. Rahman

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ABSTRACT

This study was undertaken in an effort to develop procedural methodology for the consideration of alternative solutions for water resources development in a short period of time with a view toward reduction of total costs involved in prefeasibility studies. A review of related literature was made to reveal the status of the need for the development of a comprehensive method to evaluate possible alternatives.

Three techniques were developed in this research effort to estimate the investment costs of a reservoir project, a levee project, and a basin conservation reservoir project in an economic region. The application of the methodologies were illustrated by a case study. The cost of a reservoir project in the case study area determined by the method developed in this investigation was in excellent agreement with the Corps of Engineers' estimate using conventional methods. In general, the dependability of all the three methodologies were considered good for use in order-of-magnitude estimates.

Selected solutions for water resources development problems in the Navasota River watershed were analyzed. The cost of water supply by desalination in the service area of the proposed Millican reservoir was computed following the procedure recommended by the Office of Saline Water. The investment costs of the alternatives were

compared. The multipurpose reservoir project for fllod control, water supply and recreation was found to be the least costly project. However, levees for flood protection in the lower Brazos River basin and desalting for water supply appeared to possess more intangible environmental benefits although the estimated cost of this multipurpose alternative was somewhat higher in comparison to the multipurpose reservoir project.

Evaluation of intangible cost factors (environmental, aesthetic, etc.) was not possible in want of scientifically amenable procedures. Total costs (combined tangible and intangible factors) of all alternative plans could not be estimated. As a result, no attempt was made to recommend any specific alternative to the proposed action in the case study area.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER I

INTRODUCTION

Water is a basic necessity for the survival of most forms of life on earth. In an age of population explosion and rapidly growing urban industrial expansion the development of a better water resource and land use management program is of vital importance. Planners need an efficient methodology capable of producing a viable resource development program. There exists also a need to manage wisely our water resources. The main problem is that water is not often available at the right time, in desired locations, and of adequate quantity with acceptable quality. Nature picks its own time and place for distribution of this resource. Planners seek to modify the hydrologic cycle by structural measures that force the movement of water in time and to places better meeting known human needs in desired quantity of specified qualities. Furthermore, it requires new and imaginative actions to promote wise use of river water and river-valley lands.

The planning of a water resource project is a complex task. It takes a minimum of 3 to 5 years from conception to construction and may take several times that long for large projects (32). A good deal of time is required to collect the necessary data and to analyze possible alternatives in order to develop a satisfactory plan. In addition, the complexity of water resource development problems stems from a variety of proposed solutions. These solutions may be structural, non-structural, or a combination of both, e.g., reservoirs, levees, flood plain zoning, or flood insurance. Although some of these solutions are not difficult to express in quantitative terms, there are factors which are not yet amenable to much more than a qualitative description, e.g., environmental impact. Moreover, most projects involve many controversial agencies, local people, or some special interest group and sometimes it becomes quite difficult to work out mutually acceptable compromises. However, a compromising formula within the framework of our present state of knowledge is necessary to iron out the differences of various groups and to arrive at a workable solution on the basis of some agreed criteria, e.g., benefit-cost ratio. As an illustration, it can be said that the solution which will provide the optimum benefit-cost ratio (evaluated with the present state of knowledge for assigning economic value to various factors) is acceptable.

The complexity of water resources planning increases manifold for multiple purpose projects that regulate streamflow, supply water for various purposes, generate hydroelectric power, protect highly industrialized and urbanized areas against flooding, satisfy recreational needs of large number of people, or attempt to attain other objectives. There are many possible alternative directions in which water resources may be developed and utilized. Each of these directions represents a solution to a problem that demands the optimization of a predetermined criterion, subject to a set of constraints. The policy selected by any optimization procedure is a product of the input data. Better results can be expected only if the input data are reliable.

It is recognized that each water resource development project is unique for a particular area of interest. No generalized solution can be recommended for a regional problem. What seems optimal under one set of circumstances at one time may not seem favorable at another.

It is realized also that no technique has yet been developed to assign economic value to the impact of a water resource project on the environment. Werner (95) states that the hopeful science of predictive ecology is not yet even in its infancy. Environmental knowledge will continue to expand, however, when a decision is made, it must be based on the best knowledge available at that time.

Complete economic details of all aspects of water resource planning cannot be evaluated with the present knowledge of estimation. All water resource development projects involve investment of financial resources. Since financial resources are not unlimited, some decision must be made relative to the expenditure which can be permitted. In order to achieve optimum utilization of national finances, all potential alternative solutions to water resource problems, including their environmental effects, must be investigated. Recognition of the full spectrum of potential alternatives for analysis is of paramount importance if the most efficient course of action is not omitted at the outset. The available techniques for evaluating costs of different alternatives to fulfill a set of objectives are costly and time consuming. It would be fairly correct to say that almost all water resource development projects are governed primarily by economic considerations. When several alternative projects would serve more or less the same purposes, a cost comparison must be made before making a decision. Therefore, it is imperative to make preliminary estimates of cost of each alternative so that the promising solutions can be screened for more detailed investigation and analysis.

The broad and general goal of this study was to develop procedural methodology for the consideration of alternative solutions for the development of water resources in a short time frame with a view toward reduction of total costs involved in prefeasibility studies. This would enable the participants of project evaluation teams to compare more potential solutions which may lead to more efficient decisions. Documentation of the procedure followed and the publicity of the decision taken by the planning team would help the beneficiaries to judge, with minimum controversy, the viability of the decision.

The central purpose of this investigation is to present a procedural framework by which prefeasibility cost estimates or order-of-magnitude estimates of proposed solutions can be obtained for typical situations. It must be realized that the procedures suggested are not intended to displace detailed engineering estimates for feasibility purposes.

This study has the following specific objectives:

1. Development of methodology for making an efficient evaluation of order-of-magnitude estimates for flood control for:
   (a) a reservoir project,
   (b) a levee project, and
   (c) a basin conservation reservoir project.
2. Illustrated application of the methodology.
3. Estimation of costs of a desalting project that could substitute for the water supply provided by the proposed Millican Reservoir in the Navasota River watershed.
4. Documentation of results with a view toward illustration of the order-of-magnitude costs of separate alternatives.

A case study approach is considered to be the most effective technique to illustrate the methodology. The proposed Millican Reservoir project in the Navasota River watershed was used in this research. It is to be noted that only those factors that influence the order-of-magnitude estimate of costs for each potential alternative have been considered.

The relationships developed to achieve the first objective would be of great value as input data for a system analysis. The estimated costs will be useful for a prefeasibility cost comparison and they will help the project evaluation teams to decide on the most viable solution. In particular, the results will contribute important elements to fill the gaps in the development of a more complete methodology for comparative studies of alternative solutions for problems of water resources development.

CHAPTER II

REVIEW OF LITERATURE

The control and utilization of available water resources has always been a part of man's endeavor to improve civilization. Since 1802, the United States Federal Government has conducted programs to develop the country's water resources (15).

Both levees and reservoirs were commonly used for flood control relief measures in the early 1900's. Alvord and Burdick (2), as early as 1918, noted that all reasonable remedies for flood protection should be evaluated to ascertain the best course of action. However, they did not mention what were the reasonable measures and how to evaluate them. Then, just recently in 1972, Buras (5) stated that technological development for comprehensive formulations of problems in water resources planning, utilization, control and conservation is only a relatively recent development.

Although studies concerning the evaluation of alternative solutions for water resource development have been made over a long period of time, systematic work with concerted effort in this direction started around 1950. A comprehensive survey of available records, research reports, journals and texts revealed the state of technological progress for the formulation and evaluation of water resources development problems. This review is presented below.

The report of the President's Water Resources Policy Commission (52), published in 1950, was among the pioneering documents for comprehensive formulation of problems in water resource planning. The Commission stated that the procedure for evaluation of water resource development should be revised and values should be assigned to public benefits and costs. Methods of comparing competing alternatives should be determined and all effects should be evaluated in comparable terms or units. This was necessary to answer the most common and important question regarding which project was the best of the possible alternatives.

During the period 1951 to 1955, the United Nations' Economic Commission for Asia and the Far East published a number of reports (68, 69, 70, 71) on the flood problems and various flood control measures suitable for that region. Qualitative discussions on various elements of water resources development, objectives of multipurpose reservoir projects, aspects of watershed management, fisheries problems, recreation and health considerations were presented, but no means to compute the economic value for any of the measures was found in these reports. In 1954, Peterson (47) published a book, Big Dam Foolishness, which emphasized that big dams were not the only answer to flood control. Other measures might be more feasible economically, as for example, small reservoirs of the type used in soil conservation practices. This comment was made on the basis of some practical data, however, he did not develop any procedure for estimating costs.

Eckstein (15) published a book in 1958 explaining the economics of project evaluation on the basis of benefit-cost analysis and considered this as a promising evaluation method for public expenditures. Unfortunately, the book did not contain any procedure for the ready estimation of cost of individual alternatives. Moore (42) suggested that structural and nonstructural (land use planning) measures should be practiced together to alleviate flood damages. The President's Water Resources Council (51) in 1962 also pointed out that multipurpose projects should consider all relevant means including nonstructural as well as structural measures. The Council did not suggest any methodology for comparative evaluation of possible plans. Thus, the question remains on how to assign value to the alternatives.

Grant (21) published in 1960 one of the most useful and pioneering treatises on application of engineering economy in structuring and handling alternatives. He emphasized that the physical consequences of each alternative should be evaluated in money units so that the comparison can be made on a uniform basis. Hall and Buras (23) in 1961 showed an analysis of a sequential multistage decision process in water resources engineering through the application of dynamic programming. In a book published in 1962, Maass, et al. (41) presented methodology for designing water resource systems. They stressed the following steps in the methodology: (a) identification of objectives of design, (b) translation of these objectives in design criteria, (c) formulation of plans to fulfill the criteria and (d) evaluation of the consequences of plans. As a new approach of high efficiency, the process of analysis was computerized. However, no specific method was spelled out in these publications for rapidly estimating the economic value of individual alternatives.

Linsley and Franzini (39) stated that long-range and sophisticated planning was essential for the efficient use of water. Water resources development is influenced by social, economic, ecological and political considerations as well as basic engineering facts. They presented the basic system and structural measures for engineered development of water resources. Nonetheless, a methodology for rapid evaluation of individual solution was missing.

In 1965, James (28, 29) devised a procedure, based on economic criterion, for incorporating nonstructural measures in flood control planning. The optimization equation (minimize C) was

$$C = C_F + C_S + C_P + C_L ,$$

where C= total cost of plan, C_F = cost of flood damage, C_S = cost of structural measure, C_P = cost of flood proofing and C_L = cost associated with flood induced adjustment in land use. It was noted that the major obstacle to achieve an optimum solution of the above equation was the inability to evaluate adequately the component costs. James recommended to use reasonable estimates of costs based on approximate procedures, but he did not explain or present a procedure. In 1966, Rachford (53) developed a computer program to perform the computational task of the procedure suggested by James. The only structural measure considered in this program was channel improvement. In 1968, a number of reports were published by James (30), Villines (93), Dempsey (14) and Cline (9) of the results of their research activities, at the Water Resources Institute, University of Kentucky. The basic objective of the groups' efforts was to demonstrate how the digital computer could be used in flood control planning. The computer programs they developed did not include the use of a levee as a measure of flood control. Moreover, they did not mention how suitable these programs would be for multipurpose projects. It was concluded that more research was necessary for the extension of the programs and for the evaluation of unit cost for each alternative to be used as input data in the optimization technique.

Kuiper (36) urged investigation of alternative proposals in dealing with an engineering project and to select the most promising one. He said that capital costs could be obtained from detailed cost estimates. However, it appears that detailed cost estimates for all alternatives may be too costly and time-consuming for prefeasibility studies.

In 1966, Hufschmidt (26) summarized the outcome of a water resources research program at Harvard University. The main activities of this group were concentrated on theoretical and conceptual aspects of planning. In one of the conclusions it was suggested to conduct case study research to evaluate actual water resource planning problems. In a subsequent paper (27), he advocated that current methods of planning be changed substantially to accommodate environmental considerations. He further urged the conduct of extensive research and experimentation on new methods and approaches for computing benefits, costs and production functions. The Task Force of Federal Flood Control Policy (63) also corroborated this idea of developing new methods.

Koenig (35) in 1966 presented a cost estimating technique for a conventional water supply situation, i.e., a reservoir. He computed the unit cost of water, and operation and maintenance cost of the reservoir based on a statistical analysis of the costs from over a thousand reservoirs in the United States. He did not consider other alternatives for single or multipurpose projects, but the attempt should be commended.

In a United Nations' publication (72), a panel of experts in 1968 noted that new processes have emerged in desalination and the cost of desalted water has been decreasing continuously. The economic viability of desalting as a competitor of conventional water supply is increasing. Kuiper (37) pointed out desalting as a more promising alternative to importing water from Canada to United States. The meaningful observation here is that unless all possible alternatives are evaluated and compared, it is difficult to decide on the best solution. Therefore, development of a methodology for comparing alternatives is of paramount importance.

In 1968, Whipple (96) criticized a proposed national scheme for flood insurance. He suggested that flood plain zoning would be more acceptable and would maintain environmental quality. In another publication (97) he further mentioned that the real solution should be based on a combination of structural and nonstructural means. A similar contention was expressed in a paper by White (98). White also pointed out that engineers must perfect new techniques to weigh alternatives. The general opinion was that effective comparison would require fashioning new methods of comparative appraisal.

Goddard and Weathers (19) emphasized the need in 1969 for comprehensive studies of all applicable alternatives and combinations thereof. James (31) reiterated the application of the two computer programs developed under his supervision, as cited earlier (9, 14, 30, 93), for evaluating alternatives. However, he observed that with available methodology, systematic evaluation of all combinations of structural and nonstructural measures was not possible.

In 1970, Hall and Dracup (24) described a system engineering approach to water resources development. Stephenson (62) used linear programming to optimize the design and operation of a reservoir system. He did not evaluate the possible alternatives for the example areas. Aron and Scott (3) demonstrated that dynamic programming could be used to determine the optimum use of water in a conjunctive system involving several sources. Grant and Ireson (22) cautioned that comparison between alternatives should be expressed in terms of money. The objectives of water resource system can probably be described as a spectrum of goals, rather than the maximization of a single scalar numerical quantity. The outcome of the systems approach of optimization technique will not be of much practical value unless all the component costs are reasonably correct. The references cited in this paragraph did not outline any rapid method to evaluate individual alternatives.

In 1971, Kelnhofer (34) concluded that poor plans would likely be adopted when all alternatives were not presented. The research community must find ways to put our knowledge to use more rapidly. In a report to the 92nd Congress, a Special Task Force (60) suggested that a multi-objective approach should be used in water and land resources planning. The Task Force recommended the use of a systematic process to formulate alternatives and then tradeoffs would be used to guide decisions. Goddard (20) advised that reason should replace emotion and scientific evaluation should be used instead of an empirical value judgement. All these qualitative remarks implied that the development of a time and money saving methodology for comparative study of potential alternatives was a necessity.

James and Lee (32) presented the mathematics of economic analysis of water resources development. In general, the materials were presented in a manner for developing a philosophy of planning. They recognized that it was of basic necessity to express the consequences of alternative courses of action in commensurable units; otherwise, economic comparison would not be of any relevant value.

For optimization of water resources systems, Chow (6) developed two computer programs: (a) the Discrete Differential Dynamic Program (DDDP), and (b) Multi-Level Optimization Model (MLOP). These programs do not, however, evaluate the cost of individual elemental alternatives at different stages of development. Parker (46) discussed and presented a procedure for a detailed estimate for dam construction but it would be too costly to estimate the order-of-magnitude of cost for all alternatives of a project.

Waste water reuse was considered to be a possible source of water supply. In 1972, the Alamo Area Council of Governments (1) developed computer programs for the preliminary design and costing of waste water renovation by lime-clinoptilolite-carbon processes for expected conditions in San Antonio in the year 2000. It was found that the treated waste water was only ten percent more costly than the conventional supply.

In a report, Summary Analysis (July, 1972), the U.S. Water Resources Council (92) stated that appropriate methods and techniques should be used to provide reliable estimates of the consequences and feasibility of each alternative plan. It was not explained what was meant by appropriate methods and techniques.

Copp (10) demonstrated that flood damages could be alleviated without following the traditional solution of flood control by levees and dams. The nonstructural solution was more acceptable to the people of Pullman, Washington, although the Corps recommended channel improvement and construction of dikes for easement of flood damages. This shows that unless potential alternative solutions are evaluated the best course of action cannot be decided.

General Clarke (8) in 1972, as Chief of the Army Corps of Engineers, issued a policy statement to give special attention to the full exploration of alternative solutions which would preserve resource options for future generations. He strongly encouraged broad public participation in Corps planning. Possible alternatives should be spelled out and the tradeoffs involved should be presented. In September 1972, Sargent (55) presented an approach called "Fishbowl Planning" to involve local interests in planning of public works. He noted that conducting a detailed technical checkout of all alternatives would be too expensive. An active participant in devleoping "Fishbowl Planning", Sellevold (57) pointed out that the analysis involved both an objective and subjective weighing of alternatives, one against the other, on the basis of a set of objectives established through the process of public involvement. He stated further that some of the objectives were subject to a value judgement.

Buras (5) presented and emphasized the applicability of a system approach to water resources planning problems. He mentioned the uses of queuing theory and probability analysis to such problems. However, he did not state how to assign values to the hardware elements (as he defined), for example, dams, reservoirs, levees, etc., to be used in the system analysis. It was noted by Johnson (33) that so far what has been developed in system engineering methods for water resources planning was mostly theoretical in nature and had very little real world problem applications. He observed a glaring lack of documentation in the literature of cases where the techniques of system analysis had actually been implemented in the planning, design or operation of water resource systems.

A review of the Water Resources Research Catalog (45) for 1972 revealed that no other research was undertaken to achieve the objectives of this dissertation.

In 1973, Beard (4) observed that considerable progress had been made in the application of operations research in water resource systems. However, many problems remain for multiobjective evaluations of physical output and application of operations research techniques. Our challenging efforts should continue to grow to meet the objectives. No doubt, we have been progressing but many problems still are to be solved.

The summary of the related literature presented above indicates that efficient techniques are not now available to rapidly estimate the economic value of different alternatives for a water resources development problem. The review further reveals that there are definite feelings of the necessity for the development of efficient methodologies. A time and money saving methodology is needed for comparing all potential alternative solutions.

CHAPTER III

CONCEPTS AND METHODOLOGY

General

Each water resources development project is unique for a particular area of interest. It is quite difficult to develop or to recommend an accurate generalized solution for problems for all regions. Nonetheless, there are many similarities among the regions of human occupations on earth. As an illustration it can be said that from the beginning of history man has found it favorable and convenient to settle and to establish his communities on the banks of rivers because the flat alluvial plains have been productive, and the rivers themselves have provided food, water and a means of transportation. As civilization progressed and the size of communities grew so did the need to exercise some control over the natural behaviors of rivers. It is recognized also that the sites where man attempts to control and develop water resources have many similar conditions which form a problem of common nature.

In dealing with alternative solutions of water resources development projects some cost factors in various alternatives have common characteristics. For example, a reservoir project involves the following cost factors: aquisition of lands, construction of dam, relocation of utilities, engineering investigation and design which are all irrespective of the location of the reservoir. The cases of other possible alternatives are similar. Logically, it appears that each cost factor may bear closely on a common ratio to the total cost of the alternative and may be estimated from projects of similar character for order-of-magnitude cost comparison. An accuracy of plus or minus thirty percent is expected in this type of estimate (43). Consequently, a general concept for making economic analysis for such problems seems to be possible and such techniques or methodology will be useful and valuable.

Reservoir

Background of concept. The location of a reservoir is governed by geologic, hydrologic, topographic and geographic consideration.

Geologic factors decide the type and safety of the structure. As for example, earth dams are suitable on foundations of clay, soft sandstone or variable sedimentary strata. The question of suitability of a dam site is common to all reservoir projects whether for water supply or other purposes. The hydrologic setup of the region (basin) primarily defines the resource availability or the yield of the scheme.

The dimensions of dam and reservoir, the availability and characteristics of construction materials, etc., are greatly dependent on the topography of the earth at the site. Geographic factors determine the amount and value of the necessary lands, the investment in the conveyance and distribution system in case of a water supply project, the transportation facility and the accessibility of the site, the magnitude of undesirable disturbance of property, the intensity of undue interference with amenities, submergence of historical monuments, etc. It is apparent from the foregoing that the topographic and geographic factors are economic variables which have direct bearing on the total cost of a project. Therefore, the optimum combination of these factors will result in the minimum cost of the scheme.

Creager, Justin and Hinds (11), and Pickels (49) suggested that the general location to be adopted is that which, at reasonable cost, will be best suited to the purpose for which the structure is intended. It is further understood that all planners attempt to achieve these objectives. This implies that the topographic and geographic setting of the site should be such that the valley will have adequate storage capacity with minimum construction volume of dam and with least disturbance and dislocation to the nearby areas. This will result in the implementation of a project with reasonably minimum capital investment. An examination of the decision aspects of water resources planning has revealed that, until now, all projects have been accepted or rejected on the basis of economic criteria, e.g., benefit-cost ratio or net excess benefits.

The above discussions strongly suggest that it is possible to develop some general relationship between the total cost or the component costs and some parameters that represent the topographic and geographic setup of reservoir sites. The selection of parameters for development of relationships is a matter of choice and perception of the individual investigator.

Concept. In the context of this investigation three conceptual hypotheses are developed. They are stated below.

(1) There exists a relationship between the storage capacity and the total investment cost of reservoirs constructed in a particular region provided the type of dam used and the purposes of project are similar.

(2) It is hypothesized that there exists a relationship between the storage capacity and land area submerged in a reservoir project in a particular topographic and geographic setting.

(3) The major cost components of reservoir projects with similar purposes and particular type of dam bear closely on a common ratio to the total cost of project. Specifically, the land cost is expected to be most amenable to this hypothesis.

Verification of concept. It has been realized that data from a large number of similar projects are necessary to test the contentions expressed in the preceding hypotheses. Among many agencies the United States Corps of Engineers has been most actively involved with reservoir construction and other water resources development works. The author visited the office of the Corps of Engineers, Fort Worth, Texas, and worked for some time with Corps' personnel to gather data on existing and authorized projects. In addition, a significant amount of information was collected from the literature. The data sources have been cited where they appear in this dissertation.

Storage capacities and construction costs for thirty-three reservoirs in the State of Texas were collected and presented in Table A-1 in Appendix A. These data are plotted also in Fig. 1. The scattering of the points on the graph strongly suggest a correlation between the variables involved. An approximately average curve is fitted graphically through the points. This line represents the relationship stated in the first hypothesis. The general conclusion is that there exists a relationship between total cost of construction and the storage capacity of reservoirs constructed in a particular region.

Data on storage capacities and pool areas at maximum design water surface for thirteen reservoirs with rockfill and earthfill dams were obtained reviewing the reports (73, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85) available in the office of the Corps of Engineers, Fort Worth, Texas. The relevent information is presented in Table A-2 in Appendix A. Figure 2 displays a graphical representation of the data. The distribution of the points clearly

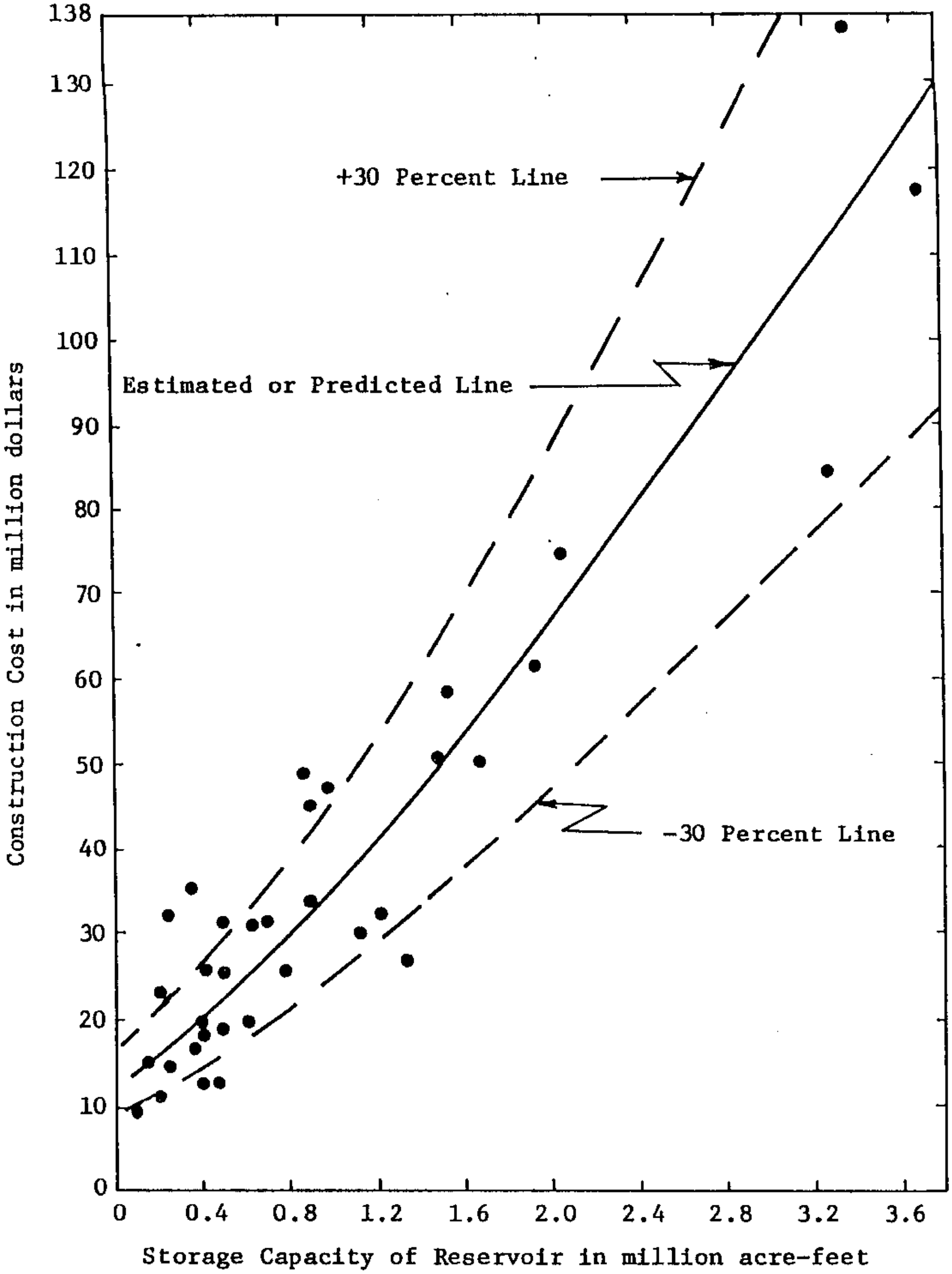


FIG. 1.-RELATIONSHIP BETWEEN STORAGE CAPACITY AND COST OF CONSTRUCTION FOR SOME RESERVOIRS IN TEXAS (July 1965 price level)

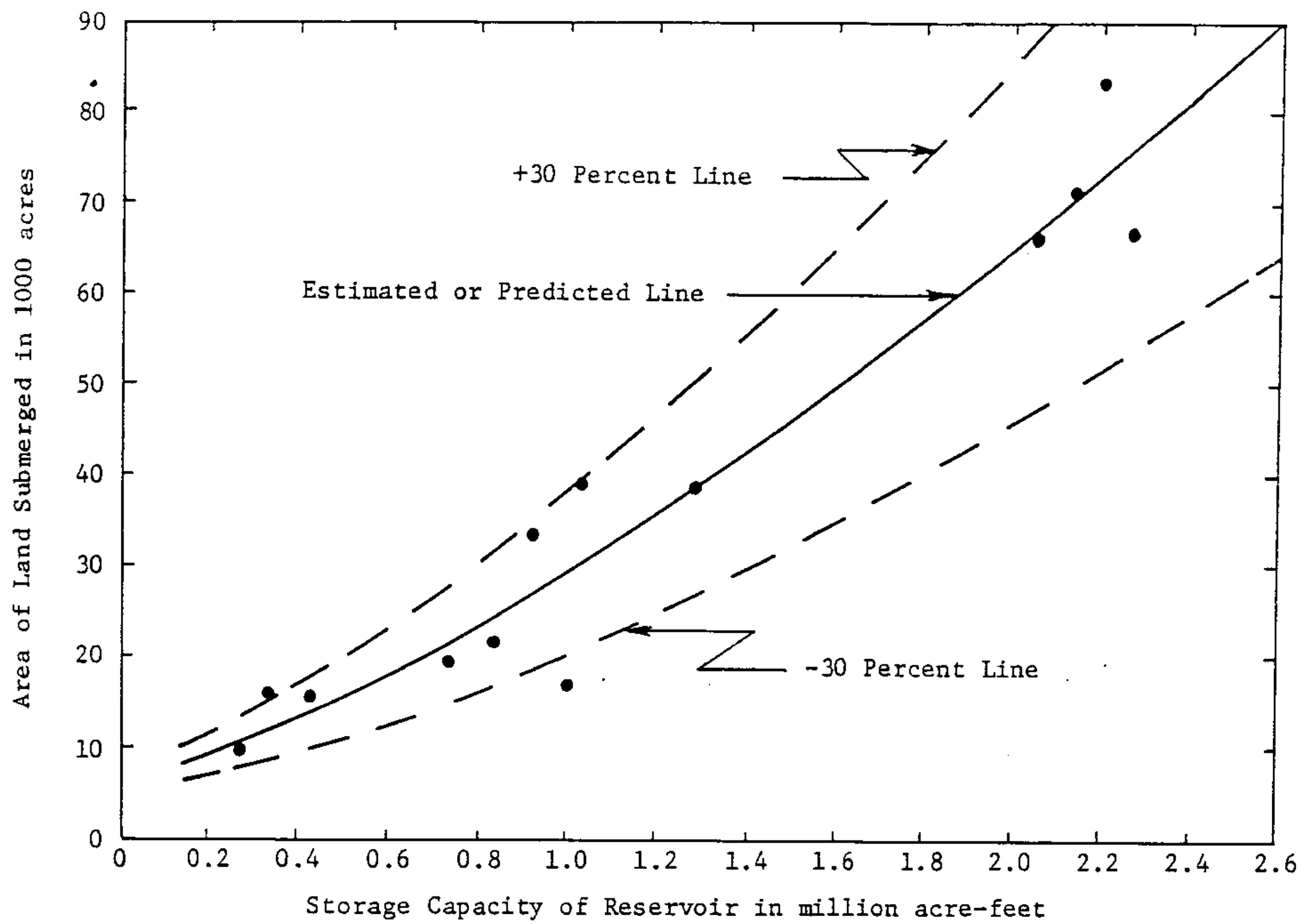


FIG. 2.-RELATIONSHIP BETWEEN STORAGE CAPACITY AND POOL AREA OF RESERVOIRS

indicates the existence of a correlation (at least for reservoirs in Texas) between storage capacity and the land area submerged due to impoundment of water. An average estimated line is fitted graphically through the points. It is observed from the graph that all the points except two fall within a plus or minus thirty percent band of the estimated line. Therefore, the concept expressed in the second hypothesis cannot be rejected.

Total investment cost in a reservoir project, primarily those for water supply and flood control purposes, is composed of many cost components. However, for the purpose of analysis the cost components can be grouped under seven major headings, viz., land, relocations, reservoir and dam, recreation and wildlife enhancement, engineering design, supervision and administration, and miscellaneous. Land cost is one of the most important cost components. A review of a few project reports (73, 75, 76, 77, 78, 80, 81, 82, 83, 84, 86, 87, 88, 89) prepared by the Corps of Engineers was made to investigate the cost analysis and summary of costs. A summary of the investigated cost analyses for fifteen reservoir projects in Texas is presented in Table A-3. The component costs expressed as percentages of the total project cost are shown also in Table A-4 in Appendix A. Table 1, a part of Table A-4, shows the total cost of the projects and land cost as a percentage of total cost. An examination of the tabulated values in Table 1 reveals that land cost bears closely on a common percentage to the total cost for all

TABLE 1.-TOTAL COST AND LAND COST AS A PERCENTAGE OF TOTAL COST FOR RESERVOIR PROJECTS IN TEXAS

Name of Reservoir	Cost of Project, January 1973 Price Level, million dollars	Land Cost as a Percentage of Total Cost	Mean Percentage
South Fork	21.35	23.0	
Aquilla	44.60	22.3	
North Fork	26.25	18.7	
Bardwell	26.35	21.7	
Laneport	57.50	30.0	
Navarro Mills	22.90	22.3	
Proctor	34.10	18.0	26.0
Waco	98.00	30.5	
Lavon	57.00	33.0	
Stillhouse Hollow	44.15	14.5	
Somerville	36.20	33.5	
Kaw	178.00	27.5	
Ferguson No.3	102.00	30.0	
Millican	128.50	37.5	
Navasota No.2	149.50	26.0	

projects. The mean, 26 percent of total cost, is a good estimate of the land cost in Texas. In conclusion, it can be said that Table 1 verifies the third hypothesis.

Methodology. The concepts and their verifications in the preceding paragraphs lead to the development of a general procedural methodology. Essential steps in the procedure are enunciated below:

1. Prepare a register of reservoirs constructed in a region for similar purposes with a particular type of dam (e.g., earthfill dam):
   (a) existing projects, and
   (b) authorized projects - for which detailed reports with definitive estimates are available.
2. Gather data on the parameters selected for the development of correlations from the completion reports of the existing structures and from the final reports of the authorized projects.
3. Group the data and tabulate them on the basis of variables to be related.
4. Develop the relationships as illustrated in the verification of concept.

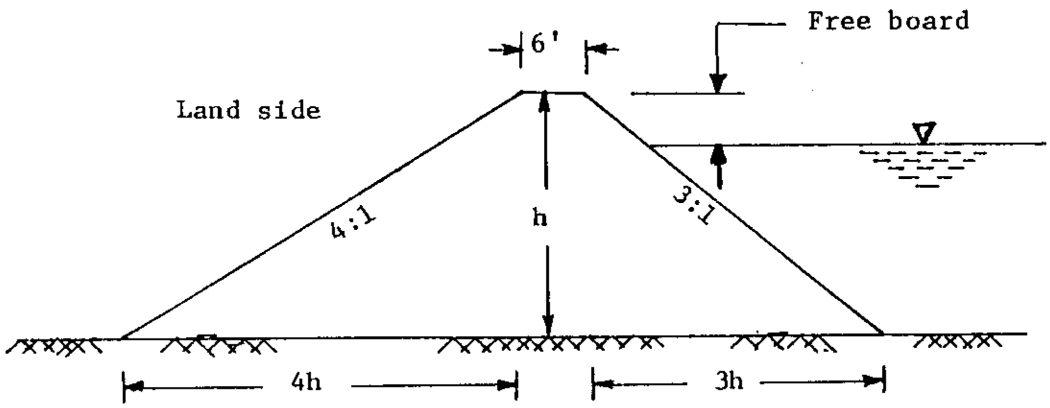
Levees

Background of concept. Levees are earthen embankments placed at varying distances from the banks of a stream to serve as artificial banks during floods when the stream gets out of its natural banks, and to protect the major portion of the productive flood plain from overflow. On the lower reaches of long rivers, levees afford the only sure means of flood protection (49). In general, levees have been used extensively in the sedimentary flood plains of rivers throughout the world. Thus, the materials used in construction of levees would have characteristics varying within certain ranges. The nature of the materials governs the possible side slopes of levees. The standard sections suggested by the United Nations (68) for use in Asia and the Far East have side slopes of 2(H):1(V) to 4(H):1(V) on the river side and 3(H):1(V) to 5(H):1(V) on the land side. The standard section of the Mississippi River Commission has slopes of 3(H):1(V) on each side and the standard section of the Upper Yazoo District provides slopes of 3(H):1(V) on river side and 4(H):1(V) on the land side (49). Therefore, for general estimate purposes a 3(H):1(V) slope on river side and a 4(H):1(V) slope on land side should be typical representative values.

Crown width used for levees varies from 3 to 10 feet. On the Illinois River, the crown widths vary from 6 to 8 feet and the standard section of the Mississippi River Commission has width of about 8 feet (49). The standard sections of levees recommended by the United Nations (68) to be used in Asia and the Far East have the range of crown widths between 2.4 to 10 feet, 5 to 6 feet being the most common. Therefore, it is apparent that a 6 feet crown width can be taken as a representative, typical one.

On the basis of the foregoing information and norm of practices the side slopes and crown width of levees may be assumed as fixed quantities. Then, the volumetric content of levees is a function of the height only. Consequently, the cost of construction is a function of height of levee. However, cost per unit length of levee will vary from place to place as the unit cost of construction varies spatially. Nonetheless, for prefeasibility studies, it may be assumed that the spatial variation in the unit cost of construction within an economic district is not very large. As a result, an average value of unit cost of construction can be adopted for general purposes.

To illustrate further, the following example of cost computation per unit length of a levee with a typical section for the lower reaches of the Brazos River is presented. The crown width is assumed to be 6 feet with side slopes of 3(H):1(V) on river side and 4(H):1(V) on the land side. Unit cost of construction in the region is taken to be $0.86 per cubic yard (65) at the January 1973 price level for common road construction with ordinary compaction. This unit value is considered as a reasonable approximation in the absence of specific data for levee construction.



DEFINITION SKETCH

$$A = \frac{1}{9} \; (6h + 3.5h^2)$$

$$V = 1173.3h + 684.4h^2$$

$$Z = \$0.86 \; (1173.3h + 684.4h^2)$$

$$= 1009h + 588.6h^2 \quad - - - - - - - - - - \quad (1)$$

where, A = cross-sectional area, sq yds,

V = volume content per mile, cu yds,

Z = cost of construction per mile, dollars, and

h = height of levee, feet .

Equation (1) is derived to express the cost of construction per mile in terms of height of levee. This equation is plotted in Fig. 3. Computation of cost data are shown in Table B-1 in Appendix B.

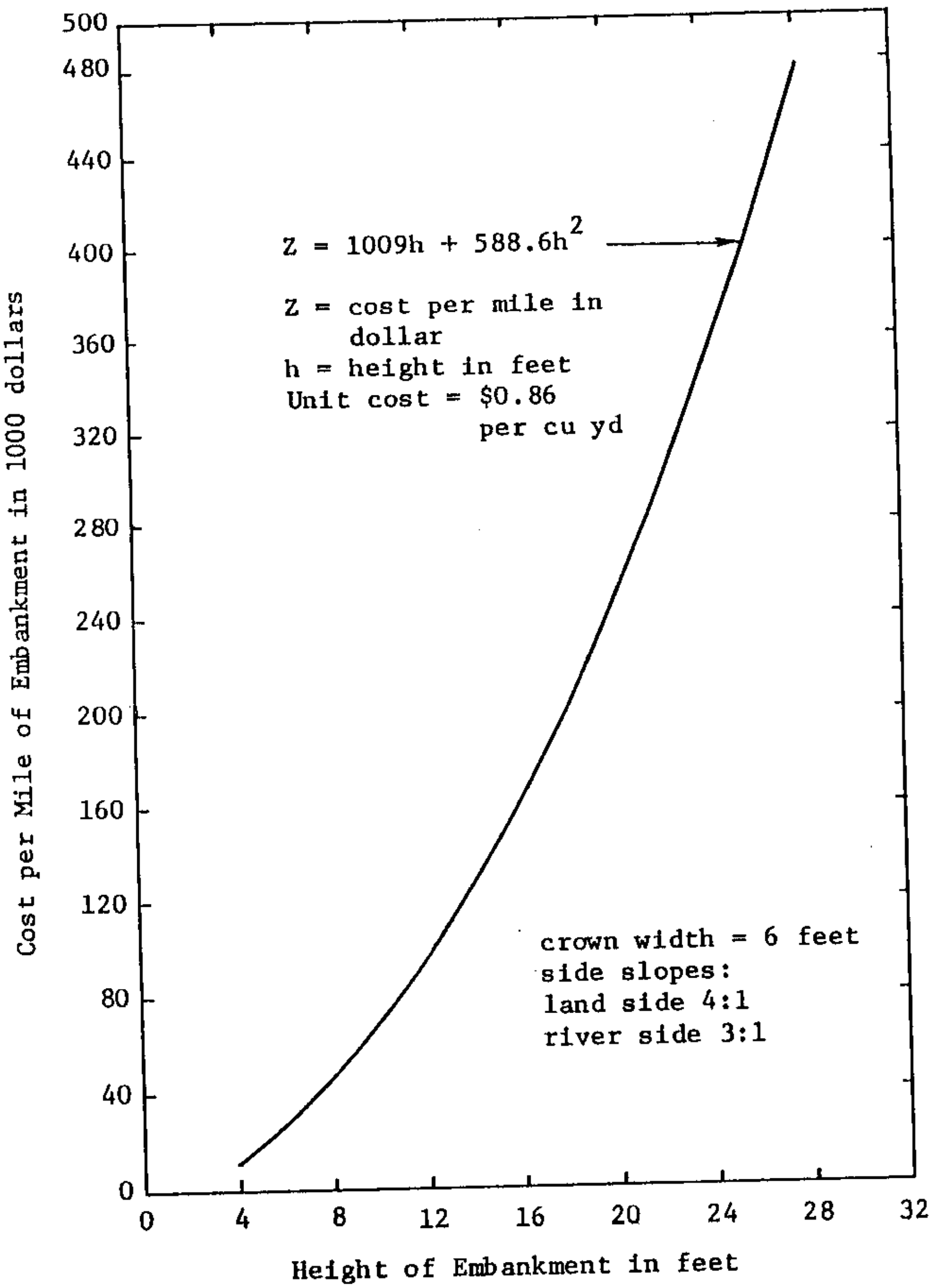


FIG. 3.-RELATIONSHIP BETWEEN HEIGHT AND COST OF CONSTRUCTION PER MILE OF EMBANKMENT (January 1973 price level)

Concept. In view of the preceding discussion it is evident that a general typical relationship can be developed between the height and the cost of construction per unit length of levees. The relationship is hypothesized as follows: there exists a relationship between the height and the cost of construction per unit length of levees build in the same economic region.

Verification of concept. To test the above hypothesis, a large amount of field data or data from other dependable sources are necessary. Offices of the Corps of Engineers, the Soil Conservation Service, and the Bureau of Reclamation were visited and contacted for data on levee construction. Information available from these sources on recent projects was found inadequate for the purpose. Finally, data on fifty-three projects in different parts of Texas were collected from the Biennial Report of the State Reclamation Engineer (100). Relevant data are presented in Table B-2 in Appendix B. The costs of construction were updated by using the Engineering News Record Construction Cost Indexes and these updated costs were plotted against corresponding heights of levees in Fig. 4. The scattering of the points strongly indicates that there exists a relationship between the variables employed. An average estimating line was drawn through the points and this line represented the hypothesized relationship. Therefore, it is shown that there is general relationship between cost of construction and the height of levee based on the above data.

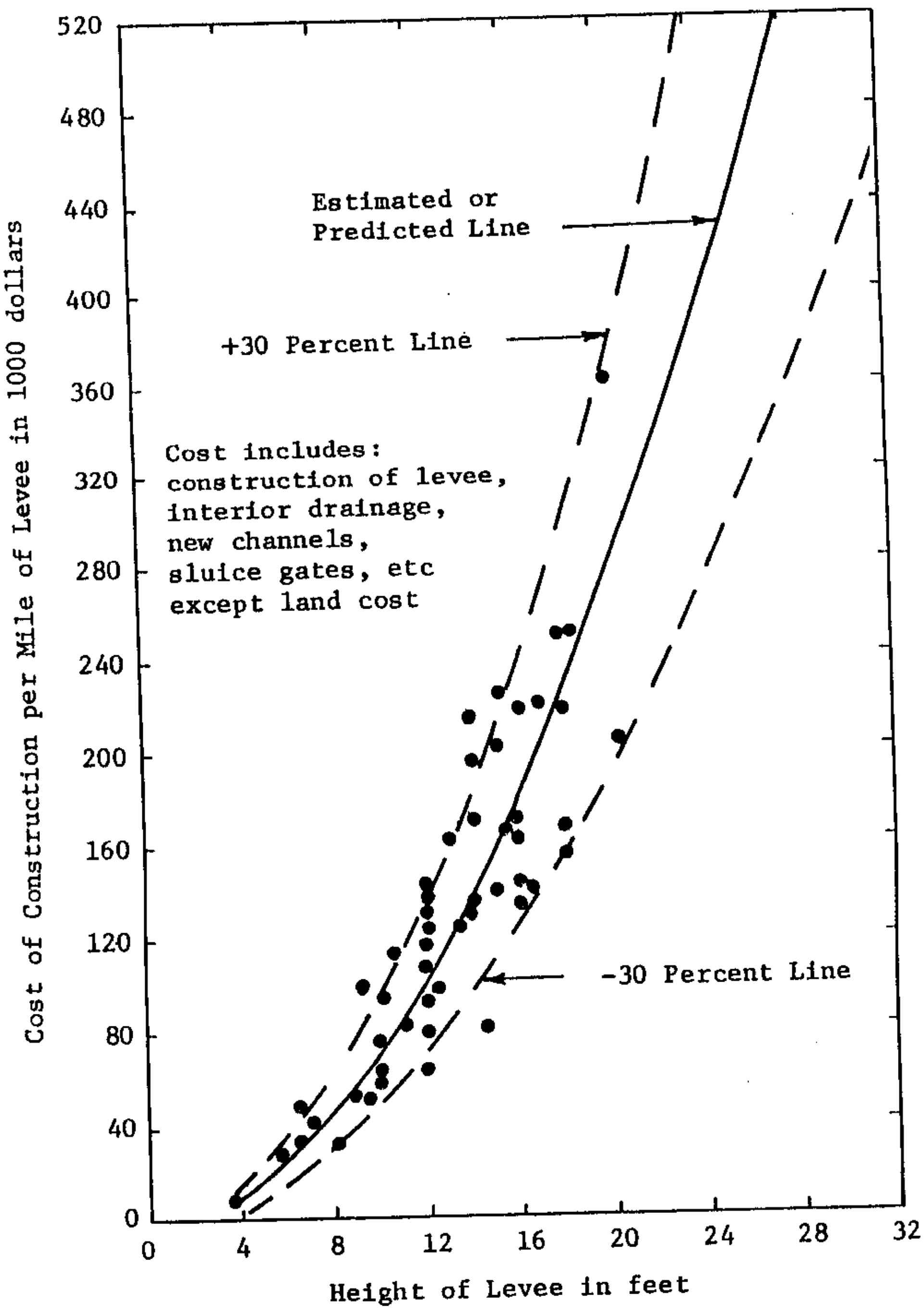


FIG. 4.-GENERAL RELATIONSHIP BETWEEN HEIGHT AND COST OF CONSTRUCTION OF LEVEES IN TEXAS (January 1973 price level)

Methodology. The general approach for the development of a relationship between height and cost of construction of levee built in an economic region is outlined in the following steps.

(1) Collect detailed information regarding the geometry of levee sections and the cost of construction in a region of interest from existing projects and/or finalized projects for which detail estimates are available.

(2) Update cost of all projects to the same price level by using dependable cost indexes, e.g., Engineering News Record Construction Cost Indexes.

(3) Group the data for projects having the same crown width and side slopes.

(4) Tabulate height and cost per unit length of levee with same crown width and side slopes.

(5) Plot the cost per unit length against height of levee on a graph paper and fit a line through the points of the same group. The family of lines are the desired relationships.

Basin Conservation Reservoir

Background of concept. The basic idea of construction of basin conservation reservoirs is to receive and hold flood water within or in the vicinity of contributing sub-basins. In other words, the function of this type of works is to impound water locally at its source, to be released later when the flood has subsided. It is well understood that if detaining reservoirs are provided sufficient in number and in capacity to hold the major part of the great floods, it would be possible to keep the flow of a stream within its banks by releasing the water gradually in a systems approach.

In preparing the plan and installation for basin conservation reservoirs there are mainly two requirements which must be met, one physical and the other economic. The physical condition of the watershed must be such that suitable sites are available for installation of such projects. The cost must be reasonable and less than the benefit. Therefore, from an economic standpoint, the dams must be comparatively short and the areas within the reservoirs should be minimum and should not be of very high value.

The Soil Conservation Service of the United States Department of Agriculture is one of the major agencies that most commonly uses basin conservation reservoirs in their programs of water and related land resources development. The Texas State Conservationist's office was contacted for information regarding the sources of data for this type of project installed and/or planned. It was determined that usually each basin conservation reservoir provided control for an area of 3 to 10 square miles. In most cases 4 to 5 square miles of drainage area were protected by individual structures.

Further search for documented evidence, as presented in this paragraph, would verify the above contention. The Soil Conservation Service installed structural measures for watershed protection and flood prevention on the watersheds of Green Creek, Cow Bayou, and sulphur Creek (59) in the Brazos River basin. The pertinent data for these projects are presented below:

Name of Creek Watershed	Number of Structures	Drainage Area Controlled by All Structures, sq mi	Average Drainage Area per Structure, sq mi
Green Creek	13	46	3.54
Cow Bayou	11	40	3.63
Sulphur Creek	10	79	7.90
All Watersheds	34	165	4.85

It appears that the average drainage area protected by each structure is a good estimate for a typical structure.

<u>Concept</u>. On the basis of the above personal information and actual field data the following hypotheses are developed:

1. The mean area protected by a basin conservation reservoir computed from a large number of projects in a region is a good estimator for a typical value.
2. The cost of installation of a basin conservation reservoir can be estimated, with reasonable accuracy, by the average cost of a large number of such structures in the same economic province.

Verification of concept. Different agencies were contacted and literature was surveyed for related data to examine the contentions expressed in the preceding hypotheses. Data on 2,745 structures in the State of Texas were gathered from the report of the U.S. Study Commission (91). The collected information and necessary computations are presented in Tables C-1, C-2 and C-3 in Appendix C. A summary of the relevant quantities are shown in Table 2 where it is seen that the hypotheses are justified.

Methodology. The concept and its verification lead to the development of a general methodology. The sequence of the process is briefly stated below:

1. Prepare lists of existing projects and/or approved projects with their locations in economic regions.
2. Collect information on: (a) area protected by projects, (b) storage capacity, (c) number of structures, (d) cost of installation and time of cost estimate, (e) maintenance cost with period of maintenance, and (f) regional cost indexes.
3. De-regionalized cost data using regional cost indexes, e.g., Engineering News Record regional cost indexes.
4. Bring cost data to the same price level using reliable and dependable cost indexes, e.g., Engineering News Record construction cost indexes.
5. Find the mean of: (a) area protected, (b) storage capacity, (c) installation cost, and (d) maintenance cost.

TABLE 2.-SUMMARY OF PERTINENT DATA ON FLOOD PREVENTION BY BASIN CONSERVATION RESERVOIR IN TEXAS

Name of River Basin	Total Nos. of Structures	Average Area Protected by One Structure, sq mi	Average Storage Capacity of One Structure, acre-feet	Average Cost of One Structure at 1961 Price Level, dollars	
				Installation	Maintenance
Neches	103	5.45	2106	67,998	3054
Trinity	977	3.17	1120	77,870	2976
San Jacinto	14	3.21	1300	50,907	2035
Brazos	805	5.05	1733	81,382	3420
Colorado	269	5.56	1472	67,968	2734
Guadalupe	70	6.88	2100	142,028	5566
San Antonio	85	4.50	1460	72,550	2860
Nueces	36	10.86	2480	146,783	5758
All Basins	2359	4.47	1480	79,700	3300

6. The mean values computed in step 5 are the typical quantities that are sought.

It is realized that there are places where adequate regional data may not be available to develop the relationships presented in the preceding sections. For dealing with such circumstances, a method of transposition is suggested. Estimate the cost of a project in an area, say region A, where sufficient data are available. The estimated cost of the project in A is then transposed to another area, say region B, where the new project is planned. Cost of project in B is equal to cost of project in A times the ratio of the regional cost index of B to the regional cost index of A. The relation is

$$C_B = C_A \; I_B/I_A \; ,$$

where C_A, C_B, I_A and I_B are cost of project in region A, cost of project in region B, regional cost index of A and regional cost index of B, respectively. This suggested modification adds flexibility to the methodology. Therefore, the methodology can be used for any economic area provided the regional cost index for that area is known.

The methods developed in this chapter will be tested in Chapter V using as an example the proposed water resource development project in the Navasota River watershed, Texas.

CHAPTER IV

DESCRIPTION OF CASE STUDY

The Navasota River watershed project, as proposed by the Corps of Engineers, was selected for a case study. A brief description of the study areas is presented in the following sections. Water resources development problems and their probable solutions are addressed also in this chapter.

The Navasota River Watershed

This watershed is situated in the east central portion of Texas, approximately between $30^{\circ}20'$ and $31^{\circ}49'$ north latitudes, and $95^{\circ}55'$ and $96^{\circ}55'$ west longitudes. The length of the watershed is about 122 miles with a maximum width of 35 miles and the general land elevations vary from about 650 to 185 feet above mean sea level (73, 74). A watershed map with component drainage subbasins is shown in Fig. 5 and the drainage areas at different river miles are given in Table 3.

TABLE 3.-DRAINAGE AREA OF THE NAVASOTA RIVER WATERSHED (Reference 74)

Point of Measurement	River Miles Above Mouth	Drainage Area, sq mi	
		Component	Total
Source	197.4	0	0
Above Easterly Gage	105.7	925	925
Above Navasota No. 2 Dam Site	83.4	401	1326
Above Bryan Gage	68.4	99	1425
Above Millican Dam Site	24.1	675	2100
Above Mouth	0.0	78	2178

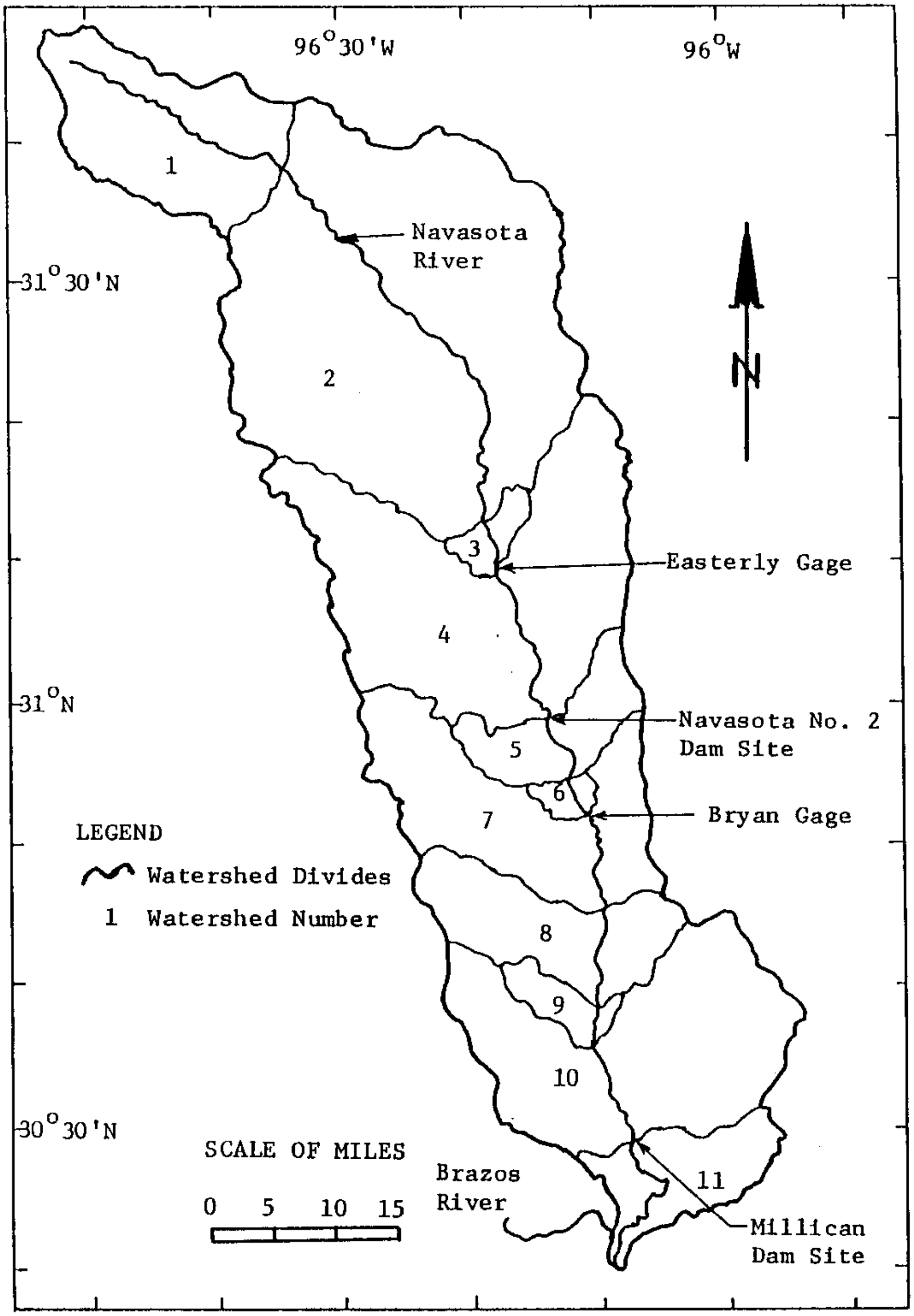


FIG. 5.-NAVASOTA RIVER WATERSHED MAP (After Reference 74)

The Navasota River

It is a tributary to the Brazos River and has a total length of 197 miles. The flood plain in the upper reach is irregular. From river mile 125 it follows a southerly course to its confluence with the Brazos near Washington, Texas. Downstream from river mile 83, the valley floor becomes wider with many sloughs and lakes, and the flood plain is covered with trees and brush. In general, the Navasota River has a relatively flat slope, the average being 2.6 feet per mile. Approximate streambed gradient and minimum bankful channel capacity for the lower 83.4 miles were taken from the Corps' reports (73, 74), and are given in Table 4. The profile of the river from its confluence with the Brazos to about mile 85 is shown in Fig. 6.

The Navasota River meets the Brazos River at river mile 232 of the latter from its mouth. Because of the flat gradient, the backwater during high flows on the Brazos inundate the lower 24 miles of the Navasota River (73). However, Turner, Collie and Braden, Inc. (67) noted that backwater flooding from the Brazos extended as much as 40 miles up the Navasota.

The Navasota River has a large number of tributaries (74). The principal tributaries are listed in Table 5 in descending order from the source to the mouth of the river.

TABLE 4.-STREAMBED GRADIENT AND CHANNEL CAPACITY OF THE NAVASOTA RIVER *

River Mile from Mouth	Average Fall of Streambed, feet/mile	Minimum Channel capacity, cfs
0-10	1.4	10,000
10-41.5	1.2	4,000
41.5-83.4	1.4	2,500

TABLE 5. -PRINCIPAL TRIBUTARIES OF THE NAVASOTA RIVER IN DESCENDING ORDER FROM SOURCE TO MOUTH *

Tributaries on Right Bank	Tributaries on Left Bank
Christmas Creek	Sandy Creek
Faulkenberry Creek	Plummers Creek
Steele Creek	Big Creek
Lake Creek	Sanders Creek
Camp Creek	Birch Creek
Little Cedar Creek	Brushy Creek
Cedar Creek	Clear Creek
Sand Creek	West Caney Creek
Bowman Creek	Panther Creek
Wickson Creek	Gibbons Creek
Carters Creek	Rocky Creek
Lick Creek	Holland Creek
Peach Creek	
Millican Creek	

* Reference 74

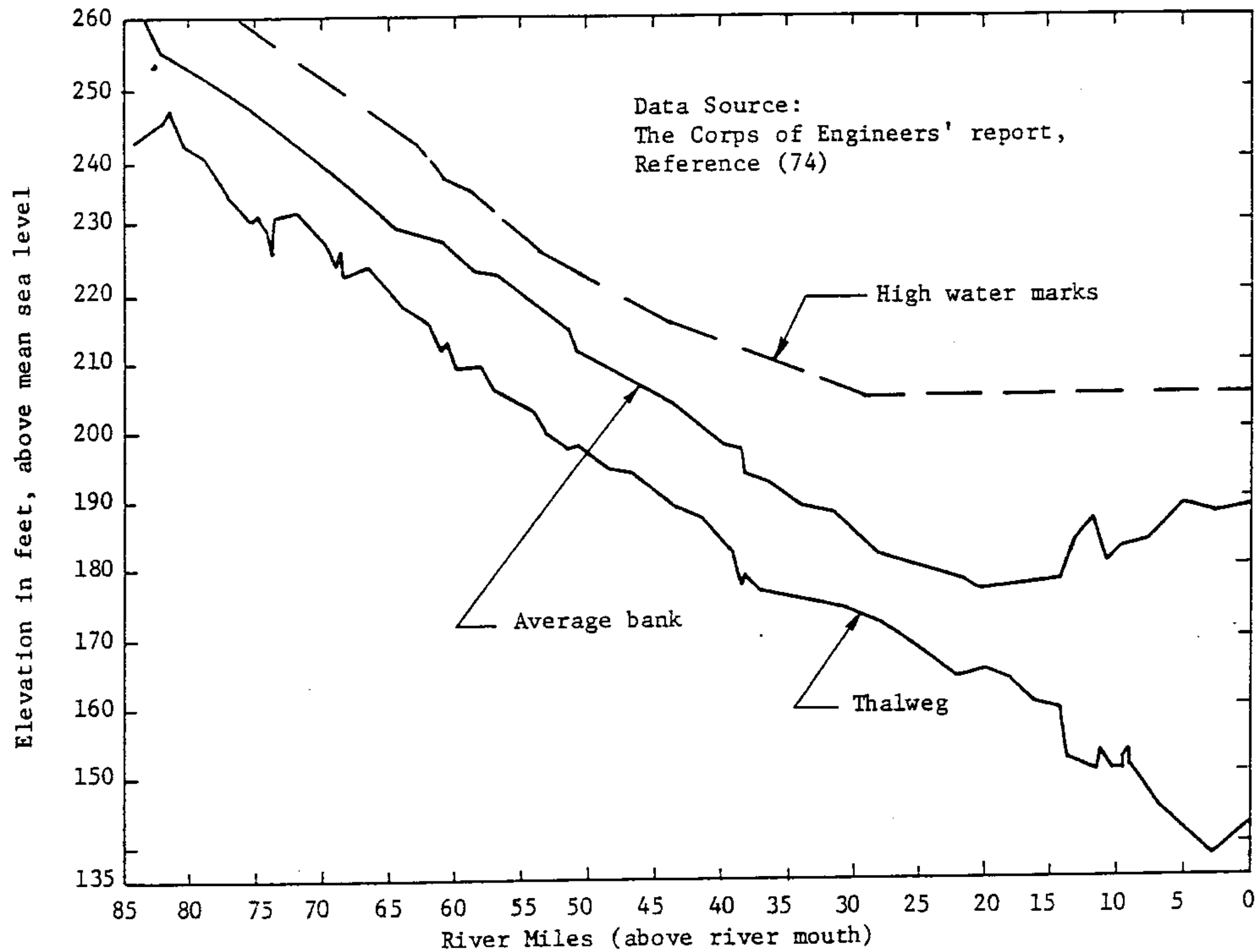


FIG. 6.-PROFILE OF THE NAVASOTA RIVER

The Lower Brazos River

The Brazos River is one of the major rivers in Texas with a drainage area of about 44,200 square miles (91). It is a moderately meandering river. The portion of the river below the Whitney Reservoir is usually termed as the lower Brazos River. In context of this research, the reach below the confluence of the Navasota River was considered necessary for consideration in this research. The river has a very flat slope in this reach. From the natural profile of the channel bottom the slope was found to be about 0.75 foot per mile.

There are pools and shoals in the floodway and in the overbank areas of the stream. During periods of low flow the floodway and the overbank become covered with weeds. The floods produced in the Navasota River watershed overflow the banks of the Brazos River in its lower reaches and sometimes causes serious flood problems. However, the construction and operation of a number of reservoirs in the stream reaches of the Brazos River above its confluence with the Navasota River reduces the probability of extremely serious floods. Nevertheless, the necessity of the flood remedial measures below the confluence of the Navasota River still exists to save life and property during high flood in the Navasota River watershed.

A study of the available records on the lower 240 miles of the Brazos River in its natural conditions was made to determine some of the physical properties, geometric elements and hydraulic characteristics of the stream at various sections. Channel bottom and average bank profiles of the river are shown in Figs. 7a and 7b. The cross-sections at different mileage, in a spacing of about 20 miles, are drawn and shown in Fig. D-1 in Appendix D. The geometric elements were determined from these figures and are shown in Table 6 with the hydraulic characteristics.

Considering the physical nature of the stream and the vegetal cover on the floodway the Manning roughness coefficient was assumed to be 0.05 for the stream at bankful stage and 0.07 for the floodway. The texts by Chow (7) and Henderson (25) were consulted to select these values of roughness coefficient. The values, n=0.05 and s=0.75 foot per mile, were used to compute the last column of Table 6.

Water Resources Development Problems

The water related problems in the case study area have been studied and summarized by the Corps of Engineers. The Corps considered all available information on present and future needs as documented by the State of Texas and by federal agencies. No effort was made to collect additional field information to identify the problems because it was considered an unnecessary duplication of efforts. Due considerations were given to all available reports of the Corps related to the case study area. For the purpose of this research attention was mainly concentrated on flood control and water supply.

It is realized that water resources development projects have regional effects and it is difficult to delineate the zone of

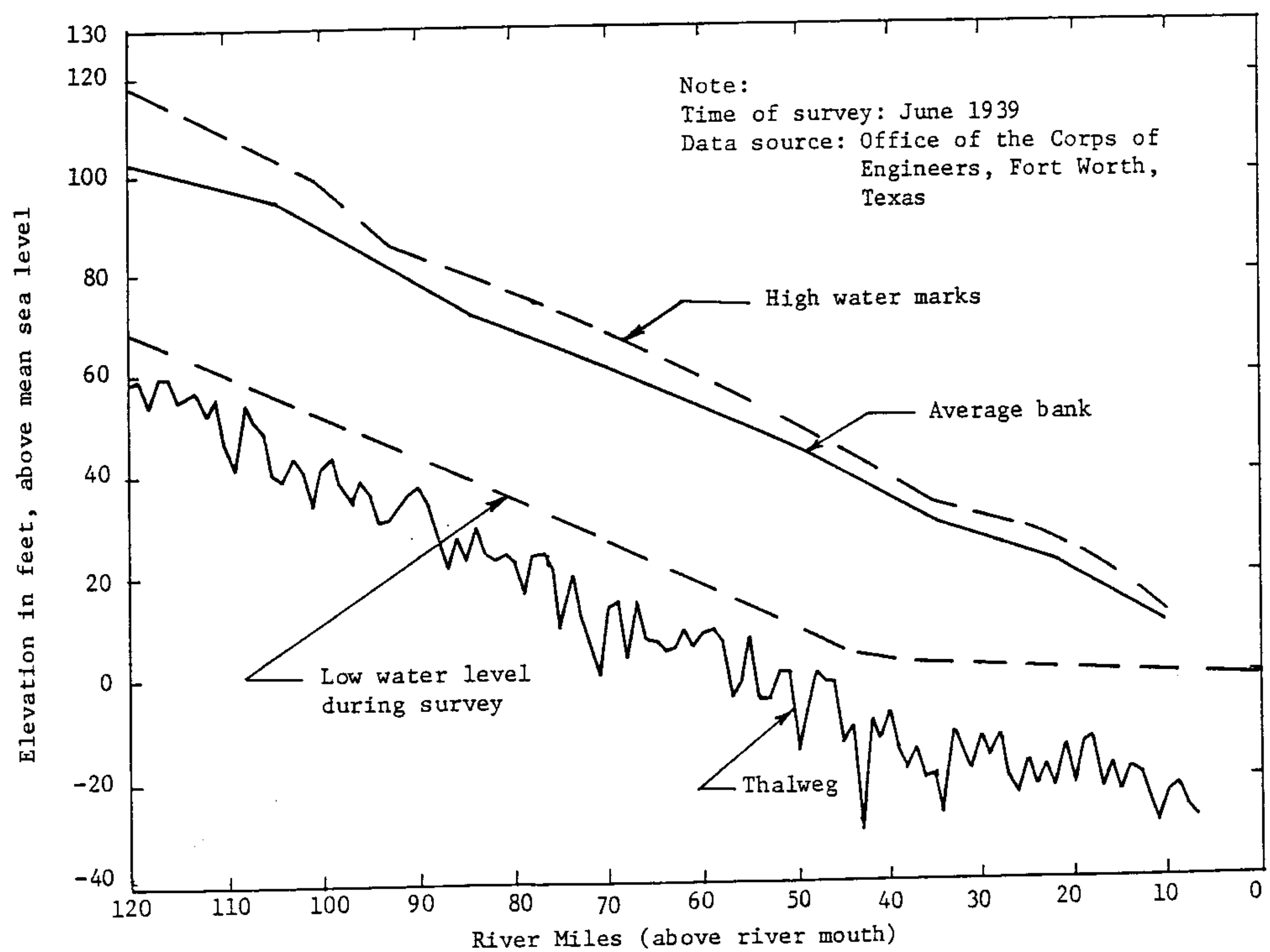


FIG. 7a.-PROFILE OF BRAZOS RIVER FROM RIVER MILE 0 TO 120

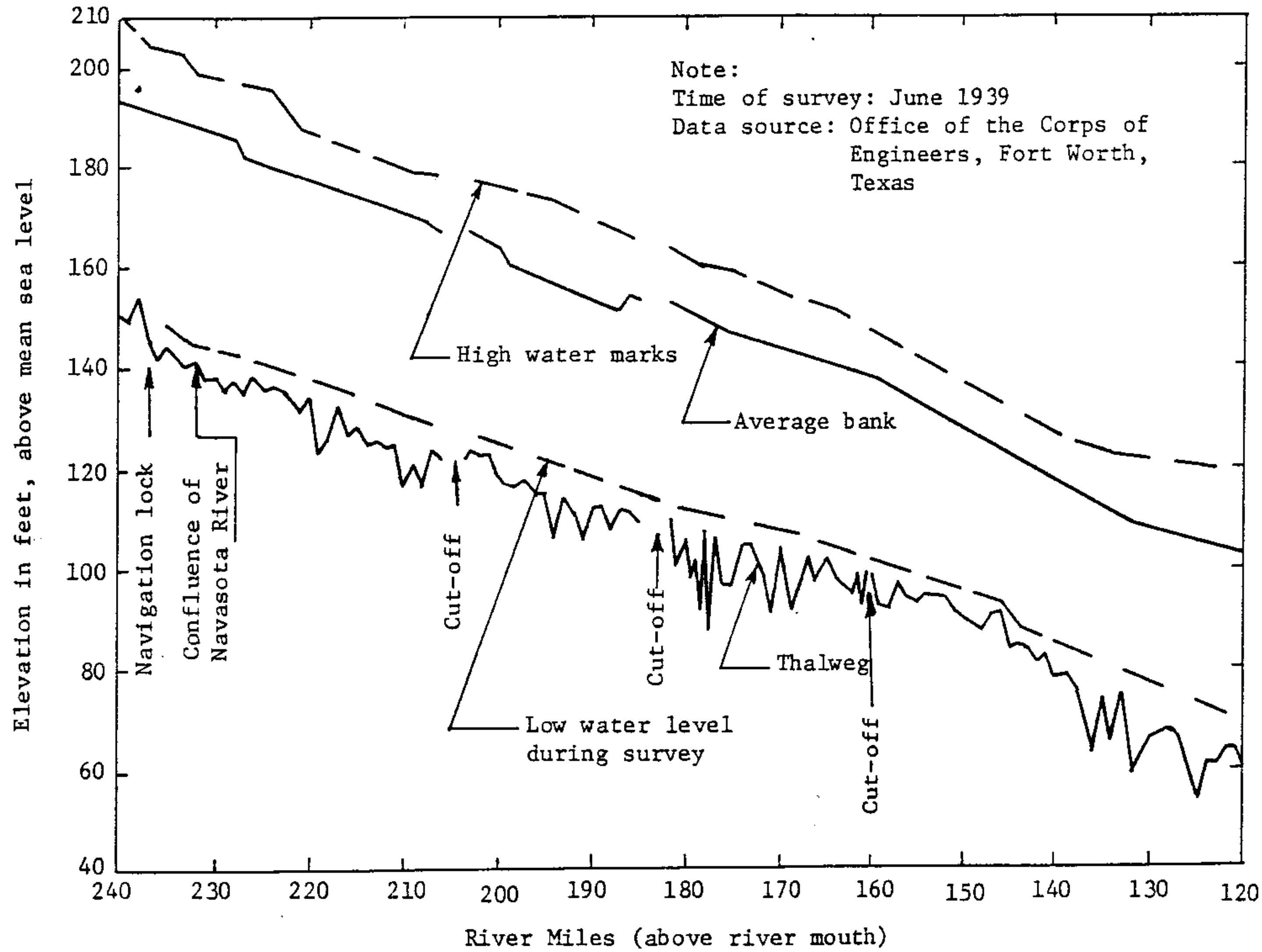


FIG. 7b.-PROFILE OF BRAZOS RIVER FROM RIVER MILE 120 TO 240

TABLE 6. - GEOMETRIC ELEMENTS AND HYDRAULIC CHARACTERISTICS OF BRAZOS RIVER AT SELECTED STATIONS

River-mile from Mouth	Distance between Sections, miles	Stream-bed Elevation from M.S.L., feet	Average Slope of Streambed, feet/mile	Top Width of Flow Section at Bankful Stage, feet	Area of Flow Section at Bankful Stage, square feet	Wetted Perimeter at Bankful Stage, feet	Hydraulic Radius, feet	Mean Velocity of Flow, feet/sec	Discharge at Bankful Stage, cfs
234.02		142.00		600	19,000	617	30.79	3.52	66,950
216.18	17.84	126.50	0.88	490	13,470	513	26.25	3.17	42,700
197.14	19.04	117.50	0.47	750	18,820	785	23.97	2.98	56,300
177.62	19.52	95.00	1.15	890	22,910	930	24.63	3.04	69,700
161.27	16.35	93.00	0.12	810	12,640	820	15.41	2.21	28,000
136.50	24.77	65.00	1.13	685	18,230	678	26.88	3.22	58,700
118.82	17.68	58.00	0.40	805	19,050	815	23.37	2.93	55,800
95.71	23.11	38.50	0.85	1,115	27,380	1,128	24.27	3.01	82,300
79.93	15.78	23.50	0.96	535	14,800	552	26.81	3.22	47,700
60.97	18.96	6.00	0.93	840	21,950	850	25.82	3.13	68,700
38.75	22.22	-15.00	0.95	500	20,070	524	38.30	4.08	86,500
16.86	21.89	-23.00	0.37	740	15,710	756	22.25	2.84	44,700

influence of such schemes. However, the areas considered in this investigation for flood control and water supply were the same as used by the Corps (74): (a) for flood control purposes, the area consisted of the flood plain of the Navasota River downstream of river mile 83.4 and the flood plain of the Brazos River below the confluence of the Navasota, and (b) the area for water supply consisted of the lower Brazos River basin.

Flood problems. Heavy storm rainfalls have caused serious flood problems on the Navasota River watershed. Channel capacities are not adequate to accommodate and to discharge the runoff produced by heavy storms. The overflow from the stream inflicts damages to agricultural lands, utilities and highways. The Corps of Engineers recorded a total of 74 floods (1924-1963) exceeding the channel capacity on the watershed (75). A list of the major floods that have occurred on the basin at Bryan and Easterly gages since their establishment is given in Table 7.

The floods from the Navasota River watershed have augmented the flood situation in the lower Brazos River. A combination of the floods from the upper reaches of the Brazos River and from the Navasota River have created the worst condition in the Brazos below the mouth of the Navasota. Based on historical records during the perios 1903-1962, the Corps (74) stated that twenty-six major floods occurred on the Brazos producing peak discharges ranging from 78,000 to 300,000 second-feet at Richmond, Texas. The effects of

TABLE 7.-LIST OF MAJOR FLOODS ON THE NAVASOTA RIVER WATERSHED (Reference: 67, 74)

Year of Flood	Period of Storm Producing Flood	Peak Discharge, cfs	
		Easterly Gage	Bryan Gage
1929	May 24-31	49.400	No Record
1930	May 8-18	30,100	"
1932	Jan. 4-18	35,500	"
1932	Sept. 4-7	58,100	"
1935	Dec. 4-7	41,700	"
1940	Nov. 22-Dec. 2	34,300	"
1944	April 19-May 5	60,300	"
1957	April 19-May 4	37,700	35,800
1960	Dec. 4-11	33,000	31,200
1965	May 14-23	43,700	35,800
1966	April 22-May 2	39,600	38,200

of other upstream flood control structures on these floods were not explicitly mentioned. However, there was no controversy that floods caused damages to urban areas and highly developed agricultural lands, numerous transportation facilities, utilities and rural nonagricultural properties on the lower Brazos River basin.

It was revealed from the report of the Corps (74) that the estimated value of physical properties in the area investigated for flood control was $279,389,700. The value of physical properties in the Navasota River watershed was only 3.36 percent of this total. The annual flood damages in the study area were $2,916,900 of which only 8.55 percent were in the Navasota River watershed. All estimates were made at the July 1965 price level.

<u>Water supply problems</u>. The U.S. Study Commission (91) showed that there would be an increasing demand for water in the lower Brazos River basin. The Brazos River Authority requested the Corps

to include at least 2,300,000 acre-feet of conservation storage in federally constructed reservoirs on the Navasota River (73). It was cited by the Corps (74) that the State of Texas published a report in May 1961 setting forth a plan to meet the future water requirements that included a reservoir at Millican with 2,400,000 acre-feet of conservation storage.

The Public Health Service, in cooperation with the Corps, estimated the anticipated water needs in the lower Brazos River basin (74). It was determined that the municipal and industrial water demand would increase from 340 million gallons per day in 1965 to 2088 million gallons per day in 2075. Bryan-College Station and Navasota would require 7.18 and 0.45 percent, respectively, of the total amount. The increase in demand was attributed to the industrial expansion and expected population growth in the lower Brazos River basin area.

From the foregoing discussion it is evident that there are flood problems and future need of water, in addition to existing supplies, in the study areas. Based on public hearings the Corps also ascertained that all concerned were in favor of taking some actions to reduce flood losses and to satisfy the estimated demand of water supply. (73, 74). The Corps further proposed and recommended the construction of multipurpose reservoirs on the Navasota River. They did not explore and compare other possible alternatives in the selection of reservoir projects. It was observed in the above paragraphs that almost all the benefits of flood control and water supply from the authorized project would be accrued to the lower Brazos River basin below the confluence of the Navasota. It was not fully evaluated, however, what share of the benefits would be returned to the local interests that would be most affected by the project.

Design flood and water requirements. On the basis of regional analysis for flood control requirements, the Corps determined that the flood of 1929 was approximately the 100-year-frequency flood. A detailed study of the economic aspects of the Millican project by the Corps showed that a project designed for a 100-year-frequency flood provided the maximum excess benefits (74). The Corps used the 1929 flood in their design. Considering predicted additional future water demand, the Millican Reservoir was planned for a safe yield of 193.9 million gallons per day (mgd) in meeting the future needs (75).

The 1929 flood and the water requirements of 193.9 mgd are adopted for analysis in Chapter V of this report.

Suggested Solutions

In view of the geographic location and natural resources of the case study area and its vicinity, a number of alternative solutions appeared to be possible. The probable alternatives are listed in Table 8.

TABLE 8.-SUGGESTED SOLUTIONS FOR WATER PROBLEMS IN CASE STUDY AREA

Flood Control	Water Supply
(a) Reservoir (b) Levees (c) Basin conservation reservoir (d) Channel improvement (e) Zoning and insurance (f) Modification of existing structures (g) Combination of two or more of the above solutions (h) No project	(a) Reservoir (b) Desalting of coastal waters (c) Ground water utilization (d) Recycling and reuse of waste water (e) Ground water recharge (f) Regulations to limit water use and industrial growth (g) Combination of two or more of the above solutions

Subsequent to the enumerated conceptual solutions to the flood and water supply problems, the next meaningful question was which one would be the best. It is a difficult question to answer. Although troublesome, it is nonetheless necessary to find the best course of action to deal with the problems. This requires attention to several aspects of the various alternatives: (i) a uniform criterion must be established to weigh all alternatives, (ii) scientifically amenable methodology should be used to evaluate each alternative, (iii) evaluation of the environmental impact of each alternative must be attempted, and (iv) consideration should be given to the impact of technological advancement on each solution. After performing these analyses, the alternatives should be compared to select the most desirable and acceptable solution.

It should be obvious that a complete and detailed evaluation of all possible alternative measures for flood control and water supply would require a great deal of time and man-hours. Under circumstances of limited funds and manpower the most promising solutions should be evaluated first. Selection of promising solutions are guided by experience and knowledge as long as no scientific technique is available. Analyses of reservoir, levee, basin conservation reservoir, desalting and ground water are presented in the next chapter. The availability of rapid assessment methodologies greatly facilitates this analysis.

CHAPTER V

ANALYSES OF SELECTED ALTERNATIVES

The primary criteria that have been previously utilized for acceptance or rejection of water resources development schemes were generally economic considerations. There have been more than one economic criteria in practice, however, benefit-cost ratio was most commonly used. From economic and cost analyses, the Corps of Engineers (75) estimated a benefit-cost ratio of 2.6:1.0 for the Millican Reservoir. The project was recommended as economically feasible.

In this chapter, for comparison between possible alternatives, it was assumed that each alternative and combination of alternatives would approximately give the same benefit to the study area. Based on this assumption, it was, therefore, necessary only to compare the cost of each project to determine the relative feasibility of alternatives. This approach is utilized in the following analyses.

Reservoir

Objectives of the authorized Millican Reservoir project were: (a) to mitigate the losses due to a 100-year-frequency flood, (b) to meet the future water demand up to 193.9 mgd and (c) to provide recreational facilities and to enhance fish and wildlife. The following pertinent data were taken from the Corps' report (75):

maximum design water surface above mean sea level = 237.90 feet

area of land required = 83,300 acres

storage capacity at design water surface = 2,199,100 acre-feet

average unit cost of land, January 1971 price level = $371 per acre

total project cost, January 1971 price level = $104,809,000.

The cost of this project is estimated also in the following paragraphs using the methods previously discussed in Chapter III for comparison.

(a) From Fig. 1 (p. 24), the cost of construction was found to be $74,000,000 at the July 1965 price level. This cost was updated to the January 1971 price level using Engineering News Record (ENR) construction cost indexes (cost indexes are given in Appendix F). The updated cost is approximately $111,000,000.

(b) From Fig. 2 (p. 25), the land area required was estimated to be 73,000 acres. Using $371 per acre as an average unit cost, the land cost came out to be $27,083,000. It was determined in Table 1 (p. 27) that the land cost was approximately 26 percent of the total project cost. Therefore, the estimated cost of the Millican Reservoir project is $27,083,000 ÷ 0.26 = $104,165,000.

Levees

In this section levees have been designed for flood protection in the Brazos River below river mile 232. The data for the design flood were taken from Plate 16 of the Corps' report (74). The outflow hydrograph at the mouth of the Navasota River is drawn in Fig. 8. It was further revealed from the Corps' reports (73, 74) that the lower reach of the Navasota River was subject to backwater inundation by water from the Brazos River when the flow in the latter exceeded about 30,000 cfs. Therefore, an assumption was made that the flow from the upper reaches of the Brazos at river mile 232 would be maintained at 30,000 cfs by system operation of the upstream reservoirs, particularly when the Navasota itself would be in flood. A base flow of 30,000 cfs was added to the hydrograph of Fig. 8 to determine the total flow at the confluence of the Navasota.

For flood routing purposes, the lower 232 miles of the Brazos was divided into six reaches. The total hydrograph was then routed through these reaches using the Muskingum method. The routing procedure used is not explained here because description of the method is available in the literature (38, 40, 90). Details of routing computations are included in Appendix B. Table 9 shows the peak flows at different sections of the river.

Determination of flows through the river section at various flood stages was necessary to ascertain the spacing and height of levees. Considering the variations in cross-sections of the flow channel and in the profile of the river banks, it was decided that the average conditions of the stream at bankful stage would be used to compute the discharge through the river section. The average

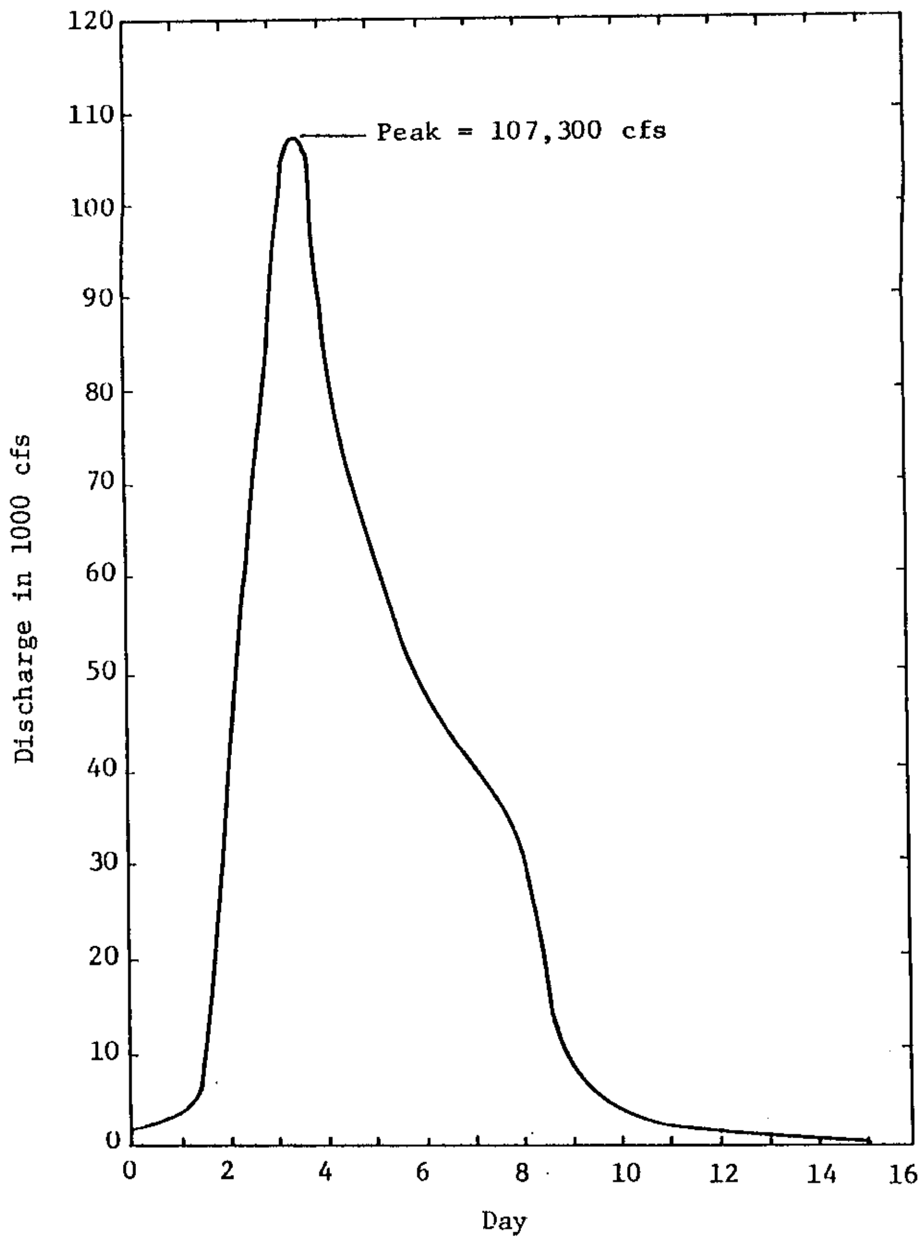


FIG. 8.-HYDROGRAPH OF 100-YEAR-FREQUENCY FLOOD AT THE MOUTH OF THE NAVASOTA RIVER (After Reference 74)

TABLE 9.-FLOOD PEAKS IN THE BRAZOS RIVER DUE TO 100-YEAR-FREQUENCY FLOOD IN THE NAVASOTA RIVER WATERSHED

River Mile from Mouth of River	Peak Flow at Section, cfs	Average Peak Flow in Reach, cfs	Length of Reach, mile	Reach
232.00	137,000			
		131,390	34.86	1
197.14	125,780			
		120,925	35.87	2
161.27	116,070			
		111,645	42.45	3
118.82	107,220			
		104,490	38.89	4
79.93	101,760			
		100,120	41.18	5
38.75	98,480			
		98,480	21.89	6
16.86				

TABLE 10.-PROPERTIES OF RIVER SECTIONS IN VARIOUS REACHES

Reach	Bankful Stage Condition			
	Flow Area, sq ft	Wetted perimeter, ft	Top Width, ft	Flow Capacity, cfs
1	17,096	638	613	55,316
2	18,123	845	817	51,333
3	16,640	771	767	47,500
4	20,410	832	818	61,933
5	18,940	642	625	66,933
6	17,953	640	620	64,600

quantities were calculated utilizing the values presented in Table 6 (p. 50). Geometric elements and hydraulic characteristics of the river sections for the six reaches are given in Table 10 (p. 61).

The data presented in Table 9 showed that two or three reaches might be combined together for design purposes. The levee designed for the average condition in the combined reaches could be adopted in each reach. This would save the computational time without much distortion of the real situation and without appreciable loss of accuracy. On the basis of the above observation the six reaches were combined into three groups. Reaches 2, 3 and 4 formed Group 2, and reaches 5 and 6 formed Group 3. In subsequent analysis and tabulation of results these new grouped reaches were used.

Spacing and height of levee could tentatively be designed with the data available in Tables 9 and 10. However, for cost computation some specifications regarding side slopes, crown width, freeboard and berms should be established. Requirements of side slopes and crown width had been discussed in Chapter III. For general purpose a crown width of 6 feet, and side slopes of 3(H):1(V) on river side and 4(H):1(V) on land side were considered typical values. Pickels (49) recommended that the amount of freeboard for a levee should be about 3 feet and the berm should be at least 10 feet. He suggested to provide wider berm for levees with steep side slopes. A berm is a clear space that is kept between the edges of an enbankment and borrow pits or the boundary line. A freeboard of 3 feet and a berm of about 1.5 times the height of the enbankment have been used in this investigation.

Using the design flood peaks, levee heights were determined at varying distances from the river banks. Cost of embankment was obtained from Fig. 3 (p. 32). The area of land required per mile of levee was computed based on the specifications described in the preceeding paragraph. The unit cost of land was the same as used in the reservoir project except that the price was updated to the January 1973 level. Total cost of the levee is the sum of embankment cost and land cost. The cost per mile of the levee is then plotted against respective height in Fig. 9. The height corresponding to the minimum cost was determined and this was the optimum height of the levee. Details of necessary computations are given in Appendix B. A summary of relevant quantities is shown in Table 11.

The cost per mile of levee (for optimum height) was computed from Figs. 3 and 4 presented in Chapter III. Total cost of a levee project along the two banks of the lower Brazos River to give protection against a 100-year-frequency flood generated in the Navasota River was calculated and equals $66,700,000, as shown in Table 12.

A graphical relationship between distance of levee from river bank and cost per mile of levee, and height of levee is developed in Fig. B-2 in Appendix B. The cost per mile of levee used in this figure does not include the cost of internal drainage of the protected area.

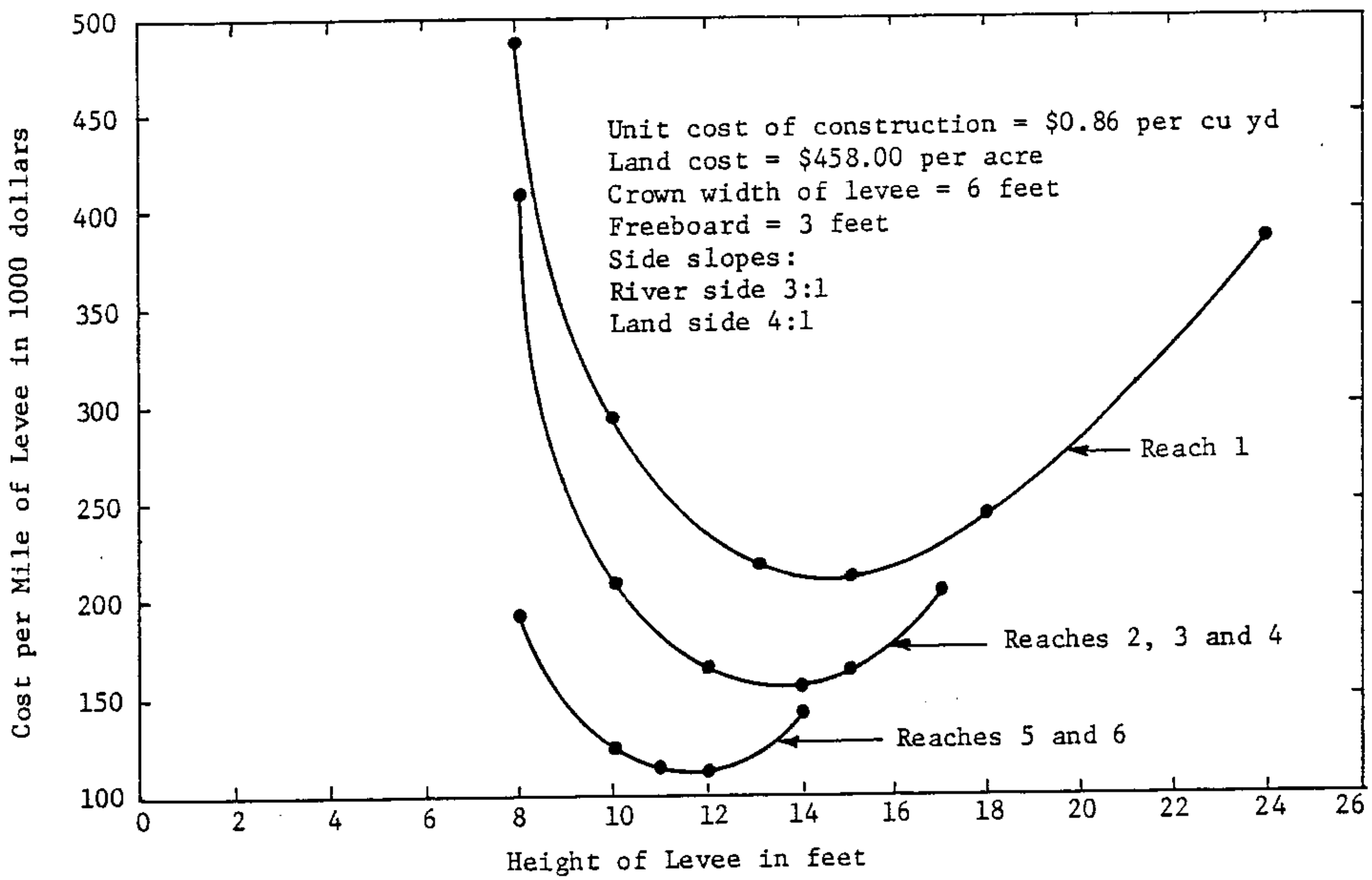


FIG. 9.-RELATIONSHIP BETWEEN HEIGHT AND COST OF LEVEE FOR THE LOWER BRAZOS RIVER

(January 1973 price level)

TABLE 11.- SUMMARY OF MINIMUM COST LEVEE SECTION

Crown width of levee = 6 feet, Freeboard = 3 feet, side slopes: 3(H):1(V) on river side
4(H):1(V) on land side

Reach	River Mile		Length of Reach, miles	Flood Peak, cfs	Capacity of Leveed Section, cfs	Distance of Levee from River Bank, ft	Total Height of Levee ft	Berm Width, ft	Width of Land Required, ft
	From	To							
1	232.00	197.14	34.86	131,390	132,200	1,000	15	25	1,136
2,3,4	197.14	79.93	117.21	112,443	112,600	405	14	21	530
5,6	79.93	16.86	63.07	99,300	99,370	140	12	18	248

TABLE 12.- COST OF LEVEE PROJECT ALONG THE LOWER BRAZOS RIVER

(JANUARY, 1973 PRICE LEVEL)

Reach	Reach Length, miles	Height of Levee, ft	Cost per Mile of Levee, dollar				Project Cost, million dollars
			Embankment (from Fig. 3)	Embankment, Drainage, etc. (from Fig. 4)	Land (Computed)	Total	
1	34.86	15	148,000	162,000	63,200	225,200	15.70
2,3,4	117.21	14	129,200	140,000	29,300	169,300	36.40
5,6	63.07	12	96,800	102,000	13,700	115,700	14.60
						Total	66.70

Basin Conservation Reservoir

Flood prevention by basin conservation is one of the oldest measures adopted by man to mitigate losses due to floods. These are relatively small reservoirs constructed at suitable sites on a watershed to store and withhold flood water temporarily. A preliminary cost estimate of such a project to prevent flood hazards in the Navasota River watershed and the lower Brazos River basin is presented in this section.

The methodology developed in Chapter III was employed in the determination of typical values for:

(a) average area protected by a basin conservation reservoir,

(b) average storage capacity of one structure, and

(c) average cost of installation and annual charges for each structure.

It was mentioned in Chapter III that necessary computations were given in Appendix C. Tables C-1 and C-2 showed the average quantities determined on the basis of the eight river basins as a single unit. Table C-3 showed the average values considering each river basin as individual unit and the average values of the eight basins. It was observed that the values computed by these two approaches were similar. However, for further smoothing of the differences, the means of the average values were considered to be more typical. The means are computed in Table 13.

TABLE 13.-TYPICAL VALUES FOR BASIN CONSERVATION RESERVOIRS

	Storage Capacity of One Structure, acre-feet	Area Controlled by One Structure, sq mi	Installation Cost of One Structure at 1961 price level, dollars	Annual Charge for One Structure at 1961 price level, dollars
Average of All Potential Structure	1,480	4.47	79,800	3,300
Average of Eight River Basins	1,720	5.58	88,436	3,550
Mean	1,600	5.02	84,118	3,425

Cost of flood prevention measures is estimated for the drainage area up to the Millican Reservoir dam site. This was done for comparing the result with that of the Corps' estimate for the Millican Reservoir project. Volume of flood water required to be detained in the reservoir project is 784,800 acre-feet (page 6 of reference 74). The ratio of the ENR construction cost indexes of January 1973 to the annual average of 1961 is 2.139.

Drainage area (Table 3, Chapter IV) = 2,100 square miles

Number of flood prevention structures = 2,100 ÷ 5.02 = 419

Installation cost of project = \$84,118 x 419 x 2.139 = \$75,500,000

Annual charge of project = \$3,425 x 419 x 2.139 = \$3,070,000

Flood storage capacity = 1600 acre-feet x 419 = 670,400 acre-feet

Difference between flood storage provided in the Millican and this project = 118,400 acre-feet.

Desalting

Desalination of sea water as an alternative means of water supply to the areas that were designated to be served by the Millican Reservoir is discussed below. Emphasis was given mainly to the cost analysis of the desalting project with a view to compare this solution when combined with other promising alternatives. For a better comprehension of the potentiality of desalting as a source of water supply and for an easy understanding of the estimating procedure used, the reader is referred to Appendix E. However, the guidelines for the selection of a particular desalting process and the formulation of the problem analyzed are discussed briefly in the following paragraphs.

The Office of Saline Water (OSW) showed in the Desalting Handbook for Planners (44) that the Multistage Flash (MSF) and Vertical Tube Evaporator-Multistage Flash (VTE-MSF) processes were most suitable for large scale commercial supply of water by desalination. These two processes can use feed water of as high as 50,000 ppm salinities to produce fresh water. In general, distillation processes are relatively insensitive to the amount of total dissolved solids (TDS) of the feed water. The OSW (44) also summarized the plant investment and fresh water production costs from the extensive cost analyses of six desalting processes made by the Stanford Research Institute (61). It was found that the VTE-MSF was the cheapest process for desalination of sea water as feed water. The

MSF process was second to it. The analysis further revealed that the cost of water production decreased with the scale-up of plant size.

Guided by the above findings, a VTE-MSF plant is selected for analysis in this investigation. It is a hybrid distillation process which incorporates both VTE and MSF. The MSF portion of the plant acts as a preheater for the feed to the VTE plant. In this part, a series of vessels or stages (usually two to three) are arranged such that the pressure relationship $p_3<p_2<p_1$ is maintained. As preheated water is introduced in each stage in succession a portion of the water vaporizes or flashes. The brine from this stage is admitted to a VTE effect. Instead of being transferred to a subsequent VTE effect, the brine is allowed to go through another group of MSF stages where again a portion flashes to form some product and preheat the feed for the next VTE effect. Individual pumps circulate the brine from the MSF stages to the VTE effect, where a portion evaporates, and the balance returns by gravity to a subsequent MSF stage. Product flows by gravity to a cooler MSF stage product tray where it too flashes, cools down and reduces some of its heat to the counterflowing feed water stream in tubes. Such configurations lead to lower product water cost due to improved thermodynamic efficiency, lower brine pumping costs, and common structural elements and containment walls.

Design problem. The following is an estimate of the cost of a VTE-MSF desalting plant constructed in the State of Texas in a coastal area, say Galveston, for supplying water to the areas that will be served by the Millican Reservoir project. The capacity of the plant is 193.9 mgd. It is assumed that the site location requires brine disposal and feed water pipes to extend one mile from shore. The average pump head at intake pumping location is 100 feet. Product water will be delivered directly to the distribution system. A natural gas-fired boiler will operate the desalting plant. Dr. W. D. Harris of the Chemical Engineering Department of the Texas A&M University suggests $0.40 per MBtu as the reasonable price of natural gas at the 1971 price level. Electric power will be available at an average price of $0.01 per kwhr. The product water quality is governed by drinking water standards. Feed water is sea water. Wenk (94) reports that average total dissolved solids (TDS) and calcium (Ca) concentration in sea water are 35,000 ppm and 400 ppm, respectively. Assume

(a) plant factor = 90 percent,

(b) interest rate on investment = 3.5 percent,

(c) insurance and taxes = 0.5 percent and

(d) plant life = 30 years (as suggested in reference 44).

Solution. Cost estimating procedures contained in the Desalting Handbook for Planners have been utilized to calculate the cost of different items. Relevant graphs were reproduced and included in Appendix E. A summary of important cost items and references to appropriate figures is presented here:

(a) construction cost of desalting plant including feed water and product water treatment costs - Fig. E-4,

(b) land costs - Figs. E-5 and E-5a,

(c) general site development costs - Fig. E-6,

(d) water conveyance costs - Fig. E-7,

(e) feed water supply and brine disposal costs - Fig. E-8,

(f) steam generator costs - Figs. E-9, E-10 and E-16,

(g) operation and maintenance labor, supplies and maintenance material costs - E-11, E-12 and E-13,

(h) chemical costs - $0.02 per 1000 gallons of product water, and

(i) electric power costs - Figs. E-14 and E-15.

Detailed cost computations are performed in Appendix E. The cost summary given in Table 14 includes cost of desalting plant, sea water intake and outfall and steam generator.

Ground Water

The utilization of ground water and its potentiality to fulfill the future need in and around the study area have been investigated by various agencies and organizations which are reviewed. Based on the information available in the literature it has been considered

TABLE 14.-COST SUMMARY OF DESALTING PLANT
(July 1971 price level)

Capital Costs:			
1.	Desalting plant	$135,000,000	
2.	Steam generator plant	17,000,000	
3.	General site development	3,200,000	
	Subtotal	$155,200,000	
4.	Interest during construction	$ 8,148,000	
5.	Start-up costs	1,862,400	
6.	Owner's general expense	10,864,000	
	Subtotal	$ 20,874,400	
	Total depreciating capital		$176,074,400
7.	Land cost	$ 75,000	
8.	Working capital	4,564,800	
	Total nondepreciating capital		$ 4,639,800
	Total capital cost		$180,714,200
Annual Costs:			
1.	Operation and maintenance labor, supplies, and maintenance materials	$ 2,380,000	
2.	Chemicals	1,412,550	
3.	Fuel	19,710,000	
4.	Electric power	3,643,650	
	Total of operation and maintenance cost		$ 27,386,200
5.	Annual cost of depreciating capital	$ 10,139,000	
6.	Annual cost of nondepreciating capital	162,000	
	Total annual capital charge		$ 10,301,000
	Total annual capital charge		$ 37,687,200
Unit cost of water = 53.4 cents/1000 gallons			

that ground water alone cannot satisfy the projected future water need in the Houston area. Therefore, a detailed estimation of ground water resources was not attempted. However, a short review of a few important publications has been presented to ascertain the adequacy of ground water reserves and the consequences of further withdrawal.

In 1944, White, et al. (99) published a paper reporting the rate of ground water withdrawal and its effect on the underlying water levels in the Houston District. Large quantities of ground water were pumped from the Houston, Pasadena and Katy areas. The rate of pumping in million gallons per day is shown in the following tabulation:

Area	Year				
	1930	1935	1937	1939	1940
Houston	39.8	38.5	41.2	43.2	45.8
Pasadena	10	10	29	29	33
Katy	18	14	30	40	45
Total	67.8	62.5	100.2	112.2	123.8

They stated that water levels remained practically constant during the period 1930 to 1937. This indicated that essential equilibrium in water levels has been reached for the amount of water pumped. It was concluded that if the 1940 rate of pumping continued, the artesian pressure would continue to decline. If it is desired to maintain water levels, pumping should be reduced and alternative sources of water supply be developed.

Petitt and Winslow (48) presented an account of ground water use and its effects on water levels and land subsidence. The amount of water derived from wells for all public uses and nearly all industrial uses was: 1938-6 mgd, 1940-17.8 mgd, 1945-34 mgd and 1945 to 1948-no change. Water levels declined in wells as pumping increased during the years prior to 1948. After diversion of water from the Brazos River in 1948, the water level in many wells became constant. Ground water withdrawal was reduced to 30 mgd. Evidence of land subsidence in Galveston County was first noticed in 1938. They observed that subsidence had occurred throughout the County during the period 1943 to 1951 and ranged from 0.207 foot at Hitchcock to 2.641 feet at La Marque. In addition, salt water encroachment was also a problem in the area.

Cronin and Follett (12) stated that the Fredericksburg and Washita Groups of Cretaceous age in the region from Eastland County to the coast in the Brazos River basin were not important sources of ground water. The amount of ground water in storage in the Quarternary alluvium was estimated to be 1,800,000 acre-feet. Cronin and Wilson (13) investigated the ground water resources of the flood plain of the Brazos River between Whitney Dam and Richmond. The thickness of the unconfined aquifer ranged from 0 to about 100 feet averaging about 45 feet. A conservative estimate of the average specific yield would be about 15 percent. The water table was observed to occur at depths ranging from less than 10 feet to almost 50 feet below the surface. The estimated recharge was about 155,000 acre-feet (138.5 mgd) per year during the period 1957-61. During the year 1964 about 49,000 acre-feet (43.7 mgd) was pumped. On the basis of the saturated thickness of the flood plain alluvium and an assumed coefficient of storage at 15 percent, approximately 2,760,000 acre-feet of water was in storage in the report area as of spring of 1963.

Gabrysch (16) found that the total withdrawal of ground water in the Houston District increased from about 311 mgd in 1960 to about 421 mgd in 1965. The water level significantly declined and in the Pasadena area the rate of decline was 9.5 feet annually. Chemical quality of ground water was changed in some areas, particularly, chloride content was increased in wells of southern parts of the district. Moreover, land subsidence continued as water levels declined. Between 1943 and 1964, the maximum subsidence had been in the Pasadena area where as much as 5 feet occurred just north of the Houston Ship Channel. It appeared likely that subsidence will continue for some time after any stabilization of water level. He concluded that to avoid future dangers due to subsidence further withdrawal of ground water appeared to be unwise and undesirable even from a water quality point of view.

In another paper published in 1972, Gabrysch (17) pointed out that pumpage of ground water in the Houston District increased from 412 mgd in 1966 to 507 mgd in 1969. The establishment of a new well field in the north western part of Houston area resulted in as much as 170 feet of water level decline.

Sandeen and Wesselman (54) reported that ground water pumpage for all uses in 1967 in Brazoria County was about 43 mgd. The ground water potential of Brazosport area was fully developed or overdeveloped. Land surface subsidence of more than 1.5 feet, attributed mostly to ground water removal, had taken place in northeast Brazoria County. Subsidence of as much as 1.6 feet had occurred in the Freeport area. It had been anticipated that no appreciable ground water potential was left over.

For comparison purposes, costs of the different projects analyzed in this Chapter were brought to the same price level, January 1973. A summary of the cost analyses is presented in Table 15.

TABLE 15 - SUMMARY OF COST ANALYSES

Project	Estimating Procedure	Estimated Price Level	Investment Cost, million dollars	Multiplying Factor for Cost Updating	Investment Cost at January, 1973 price level, Million dollars	Remarks
Reservoir	Developed in this re-research, Chapter III, Fig. 1	January, 1971	111.0	1.235	137.1	Flood control and water supply
	Developed in this research, Chapter III, Fig. 1 and Table 1	January, 1971	104.165	1.235	128.9	Flood control and water supply
	Detail estimate by the Corps of Engineers	January, 1971	104.809	1.235	129.2	Flood control, water supply and recreation
Levee	Developed in this research, Chapter III, Figs. 4, 12 and B-2 in appendix B	January, 1973	66.7	1.0	66.7	Flood control
Basin Conservation Reservoir	Developed in this research, Chapter III, Table 2, appendix C, Table C-3	January, 1973	75.5	1.0	75.5	Flood control
Desalting	Desalting Handbook for Planners by OSW	July, 1971	108.714	1.135	205.0	Water supply
Ground Water	Review of studies made for this purpose					Source seems inadequate to meet future need

CHAPTER VI

DISCUSSION OF RESULTS

The general approach toward the development of methodologies utilized in this research effort was philosophical in nature. Nonetheless, the concepts were based on purpose oriented and objectful activities of man. Some relevant discussions were made in the verification of concepts in Chapter III. The methodologies and their applications in the case study warranted more explanations to ascertain their adequacies and the future refinement needed for the techniques.

Methodology

Reservoir. The data utilized in the first technique, Fig. 1 (p. 24), were estimates for proposed reservoirs in Texas. It was not known from the report (66) how elaborate these estimates were and what type of dam would be used for each reservoir. The source also did not mention the time of estimate. The report was published in May, 1966. It was assumed that the estimates were made in 1965 and earthfill dams would be used. Although the report was published by a dependable organization, the data were associated with the above uncertainties. However, it was seen from Fig. 1 that 67 percent of the points fell witnin a band of $\pm$ 30 percent of the prediction line (graphically drawn relationship). It was further observed that the strength of prediction increased with an increase in the storage capacity of a reservoir.

The data for the second technique, Fig. 2 (p. 25), were collected from either project reports or final design memorandums that were available in the Office of the Corps of Engineers, Fort Worth, Texas. Data for thirteen reservoir projects with earthfill dams were gathered within the limited time and financial resources available. The figure revealed that the storage capacity and the land area submerged were correlated. Land cost as a percentage of total investment cost, shown in Table 1 (p. 27), was determined based on data for fifteen reservoirs only. It was realized that if more information could be collected, the strength of prediction could be asserted with more confidence.

Generalized statistical regression equations for all prediction lines were not developed. One of the basic assumptions of statistical regression analysis is that the values of the independent variables should be exact, otherwise, the outcome of the regression is biased. This implied that completed projects data should be used to develop definite relationships. It was found, in course of data collection, that the owners did not have completion reports for many projects. What they had were the final design memorandums or definite project reports. As a result, estimated values were utilized in this investigation. In addition, the number of data points were insufficient for the second technique to develop a general statistical relation. Under these circumstances, derivation of any statistical regression equations based on the data used would not be fully unbiased. Consequently, the relationships developed in Chapter III showed the general trend and approximate relations but not the exact equation.

Levees. The estimated costs used in the preparation of Fig. 4 (p. 34) were old, but they were published by a dependable source. The costs included interior drainage, new channels, sluice gates, etc., as well as levees. Nothing was mentioned about the inclusion of land cost. In addition, it was not clearly stated whether all estimates were made at the same time or not. This investigation assumed the time of cost estimate as 1929 and land cost was not included in the estimated cost. Moreover, some distortion in the cost might be incurred due to updating by cost indexes. These were the possible shortcomings of the data. In spite of the above facts, the distribution of the plotted points in Fig. 4 (p. 34) in Chapter III positively demonstrated a relationship between the height and cost of the levee. The figure showed that more than 80 percent of the points fell within $\pm$ 30 percent of the predicted curve. Because of the anticipated discrepancies, it was not attempted to describe the prediction curve by a regression equation.

A theoretical curve, Fig. 3 (p. 32) in Chapter III, for the cost of embankment with typical section in the State of Texas was developed using the January 1973 prevailing unit cost of embankment. Figure 10 shows a comparison of the prediction curve and the theoretical curve. This figure exhibited consistent results when the height of levee was more than 10 feet. The two curves become unrealistic with a levee height less than 10 feet. This disagreement might be due to the constant side slopes and crown width used in the computations. It would be more realistic to increase the side slopes and to reduce the crown width as the height of levee decreased. In that event, the curves should be in better agreement.

Basin conservation reservoir. Data utilized in the development of the methodology were taken from a dependable source, the Report of the U.S. Study Commission - Texas. Table C-1 in Appendix C showed that the average area protected by a structure ranged from 3.07 to 10.86 square miles. The number of structures in different river basins varied widely. As a result, the mean of the average area from the eight basins was different from the mean of all structures considered together. It should be noted that cost data for the 386 existing structures were not available. The means computed in Table 13 (p. 67) were based on the data of the potential structures only.

Case Study

Reservoir. The costs determined by the first method (Fig. 1)

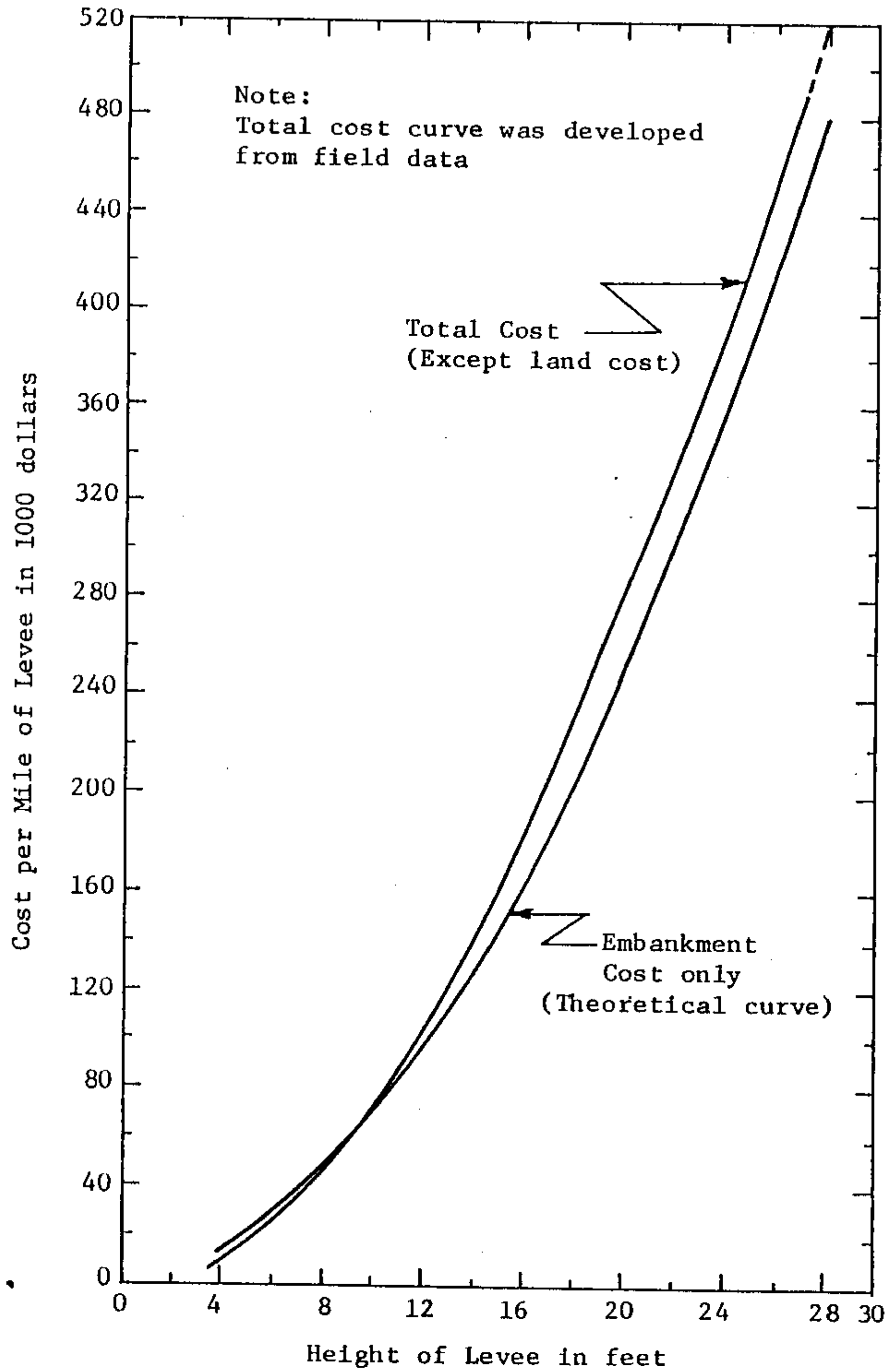


FIG. 10.-COMPARISON OF TOTAL COST AND EMBANKMENT COST OF LEVEE
(January 1973 price level)

and the second method (Fig. 2 and Table 1) were off by 5.92 and 0.62 percent, respectively, from the Corps' estimate. These differences were considered insignificant. This demonstrated the excellent practical usability of the methodology developed in Chapter III.

It must be noted here that the Corps of Engineers estimated the project cost for triple purposes, flood control, water supply and recreation and wildlife enhancement. The breakdown of cost for different objectives was made in page I-26 of Appendix I in the Corps' report (75). The allocation of cost has been shown in Table 16.

TABLE 16. - BREAKDOWN OF RESERVOIR PROJECT COST

Purpose	Cost, dollar	Percentage of Total	Ratio of Cost to Flood Control Cost
Flood Control	42,781,000	40.8	1.00
Water Supply	43,201,000	41.3	1.01
Recreation	18,827,000	17.9	0.44
Total	104,809,000	100.0	2.45

It was not known for sure, whether the data used in the development of Fig. 1 (p. 24) incorporated cost for recreation or not. The worst case would be to assume the data did not include the cost for such facilities. Then the omission of recreational cost would increase the error of prediction. Still the percentage of error would not be beyond the acceptable limit for order-of-magnitude comparison. The techniques predicted the cost of reservoir project with reasonable accuracy.

Levees. In using the Muskingum method for flood routing, the storage constant, K, was approximated by the travel time of flow in a reach at bankful stage. The constant, x, which expresses the relative importance of inflow and ourflow in storage was assumed to be 0.20 for all reaches. Each reach was about 40 miles and the routing period was taken to be 6 hours. The justification for using the above assumed values has been stated in Appendix B. Beside that, the objective of this investigation was to study the order-of-magnitude of various solutions. Therefore, the assumed values were considered reasonable although techniques were available to determine the exact values of K and x. It was realized that the accuracy of the magnitude of peak flow depended on K and x.

The elements of Table 9 (p. 61) were determined from drawings (prepared in 1939) obtained from the Office of the Corps of Engineers, Fort Worth, Texas. Present conditions of the stream most likely deviated from the 1930's situation. However, more recent data were not available and, as a result, the old measurements were utilized in this research.

Neither the Corps of Engineers nor any other agency made an estimate of a levee project in the reaches under consideration.

Therefore, the result of this investigation could not be compared and the relative accuracy of the estimate could not be computed.

Basin conservation reservoir. In the early 1960's, the Soil Conservation Service (59) conducted an investigation for basin conservation reservoirs in the Brazos River basin. It was shown in the Upstream Flood Prevention and Water Resources Development Map that this type of project was not favorable in the Navasota River watershed. The Service noted that the observation was made on the basis of a preliminary field examination which was carried out by the technicians of the organization. They did not show a detailed estimate in the report.

It was stated in Chapter IV that the Navasota has a large number of tributaries, illustrated by Table 5 (p. 44). The existence of tributaries suggests that there might be enough sites for basin conservation structures. However, without a detailed field survey definite conclusion should not be drawn. This research was intended to make a prefeasibility estimate of such a project on the basin.

The cost of a basin conservation project consisting of 419 structures was estimated in Chapter V. The total flood storage capacity of the project was about 87 percent of the flood storage provided in the Millican Reservoir project. Thus, this project is esteemed to be a promising substitute for the Millican project as far as flood control is concerned. The estimated cost of this project could not be compared with a similar project in the area because no such estimate was available from other sources.

Desalting. The procedure followed in computing the cost of a single purpose desalination plant in Chapter V was recommended by a competent agency, the Office of Saline Water (44). It was stated in page 7-1 of the Handbook, "Cost estimates so prepared should only be used for comparison studies, preliminary economic analyses, and to assist in selection of one or more water supply or augmentation plans for feasibility studies." The objectives of this investigation fully satisfied the above limitation, viz., the water resources development alternatives were evaluated for comparison.

Ground water. The summary of various investigations presented in Chapter V revealed that the withdrawal of ground water in the Houston area and its vicinity should be discouraged. Further withdrawal would accelerate the rate of subsidence of the land surface. In this context, it was the general concensus that the ground water reserves in the Houston area and its vicinity were almost fully developed. For future needs alternative sources must be found.

In a report published by Turner, Collie and Braden, Inc. (67) in October, 1973, it was observed that the ground water available in the Navasota River watershed and its adjoining areas would be sufficient to provide a suitable water supply through 2020. It was further anticipated that this source would likely be used because of economy. The relevant findings of the report are quoted in Appendix H.

Comparative Appraisal

The cost of a reservoir project estimated by the techniques developed in this investigation and the Corps' estimate were found in excellent agreement, as shown in Table 15 (p. 77). For comparison with other alternatives the Corps' estimate would be used. Table 16 (p. 83) showed that flood control, water supply and recreation costs were 40.8, 41.3 and 17.9 percent, respectively, of the total project cost. The allocations of cost were updated using ENR construction cost indexes. The updated costs are shown in Table 17.

TABLE 17.-COST SUMMARY OF ALTERNATIVES
(January 1973 price level)

Project	Cost in million dollars			
	Flood Control	Water Supply	Recreation	Total
Mutiple Purpose Reservoir	52.6	53.4	23.2	129.2
Desalting	none	205.0	none	205.0
Levee	66.7	none	none	66.7
Basin Conservation Reservoir	75.5	none	none	75.5

Flood control and water supply were the primary objectives of the Millican project. These objectives could be achieved by the alternative dual purpose projects, as described in Table 18. The costs of the alternatives were obtained by a combination of values shown in Table 17.

TABLE 18.-ORDER-OF-MAGNITUDE OF ALTERNATIVE PROJECTS

Project No.	Description	Project Cost, January 1973 price level, million dollars	Order-of-Magnitude	Rank
1	Dual Purpose Reservoir	106.0	1.00	1
2	Levee and Desalting Plant	271.7	2.56	5
3	Basin Conservation Reservoir and Desalting Plant	282.5	2.67	6
4	Reservoir for Flood Control and Desalting plant for Water Supply	257.6	2.43	4
5	Levee for Flood Protection and Reservoir for Water Supply	120.1	1.13	2
6	Basin Conservation Reservoir for Flood Prevention and Reservoir for Water Supply	130.9	1.24	3

Project no. 1 (the Millican project) and project no. 2 ranked first and fifth, respectively, among the six alternative plans shown in Table 18. However, project no. 2 has the following special features.

1. It does not include any kind of reservoir in combination.
2. This is located in the region where the benefits of the project will be accrued. It was revealed in Chapter IV that most of the benefits of the Millican project would be accrued outside the Navasota River watershed. Consequently, project no. 2 may appeal more to the local interests although it is more costly.
3. It requires 28,400 acres of land for levee and 75 acres for desalting plant whereas project no. 1 needs more than 83,000 acres of land. If this project is substituted for project no. 1, 66 percent of land area will be saved for future use. Beside that project no. 2 will utilize mostly unused river-valley land.
4. This project involves an evolving technology, desalting, whereas project no. 1 reflects a stabilized one. It is likely that desalting plant costs will come down with time whereas conventional reservoir project costs will go up.
5. The water resources of the Navasota River watershed will remain and will be available for more potential use in the future. From this view point, project no. 2 seems to be more prospective.

Environmental Consideration

A preliminary environmental study of the Navasota River watershed was made by a group of researchers at Texas A&M Universitv (64). The study team used the general methodology developed by them for execution of an environmental investigation for any water resource study. The investigation was based on available information and general knowledge of the watershed. The relevant and important findings of the study are summarized here:

1. The Millican Reservoir site is a heavily wooded bottomland area and primary recreational use of the reservoir site is for deer hunting.
2. The construction of the reservoir will help in generating an aquatic organism and perhaps waterfowl resource. At the same time it will flood the central habitat for the large white-tailed deer, grey squirrel and white turkey population. No mitigation of this loss is considered possible. It is observed that this loss is most important as a part of a general dwindling of bottomland area to support these species in Eastern Texas as a result of reservoir development at many sites.
3. The study team recommended delay of action until environmental data are collected and evaluated. The team did not suggest any specific alternative.

4. It was generally indicated that the new environmental changes which would be caused by the Millican Reservoir may be relatively minor when compared with such projects as the Trinity River System in general and the Wallisville Reservoir in particular which may drastically affect Galveston Bay.
5. Finally, it was concluded that the study team did not attempt to make any specific recommendation relative to the construction of the reservoir.

Seelig and Sorensen (56) concluded that the decline in predicted sand load of the Brazos River in the 1940's was apparently due to completion of a series of dams beginning with Possum Kingdom and improved soil conservation measures. One of the figures showed that the rate of sand input through the Brazos River to the Gulf after 1940 was one-third of the rate of sand input before 1940. They attributed this as one of the causes of shoreline retreat.

No specific environmental investigations were carried out for the six alternative plans discussed in the previous section. However, the following comment is made based upon general knowledge of the case study area. It appears that flood protection by levee and water supply by desalting will create minimum imbalance to the existing environmental setting.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

This research was directed mainly toward the development of techniques to evaluate approximate capital investments necessary in different alternative solutions to achieve some pre-set water resources development objectives. Three methodologies for determining the estimated cost of reservoir, levee and basin conservation reservoir projects were developed. It was not attempted to develop statistical equations for the relationships in want of enough trustworthy data covering all sizes of projects. The paucity of authentic data was felt immensely during the period of this study.

A tremendous amount of manpower and financial resources are needed to collect and to compile the scattered information in more useable form for research purposes. It is suggested that a governmental agency, as for example the Corps of Engineers, may maintain a division that will keep records of all water resources development activities. If necessary, the division will collect data and prepare completion reports not only of its own projects but also of other water resources development projects.

The reliability and dependability of the data used in this investigation were discussed in Chapter VI. The extent of this study was limited in comparison to the complexity and scope of the problem. The following specific conclusions are drawn from the results of this investigation:

1. This study accomplished the development of techniques for estimating the approximate investment costs of reservoir, levee and basin conservation reservoir projects. The methodologies are simple and based on actual field data from similar projects.
2. The investment costs determined by the techniques are considered most useful for prefeasibility or order-of-magnitude studies of alternatives. It is believed this will help the project evaluation teams to compare the alternatives and to decide on the most viable solutions with minimum efforts.
3. The findings of this investigation will be extremely useful in that they will contribute important elements in filling the gaps in the development of more complete methods for comparative study of water resources development alternatives.
4. Cost of a multipurpose reservoir project at Millican in the Navasota River watershed was estimated by the techniques developed in this study. The results were in excellent agreement with the cost estimates made by the Corps of Engineers using conventional procedure of estimating (this should

have been expected since the primary data were derived from the Corps of Engineers' sources).

5. A multipurpose reservoir at Millican was found to be the least costly project as shown in Table 18 (environmental costs not considered).

6. Levee for flood protection in the lower Brazos River basin and desalting for water supply appeared to possess more intangible benefits although the estimated cost of this alternative was 2.56 times that for a multipurpose reservoir project at Millican. Additional merits of this project were discussed in Chapter VI.

7. Other studies indicated that the ground water resources in the Houston area were fully utilized. Further withdrawal of ground water will accelerate land surface subsidence.

Recommendations

This investigation was not directed to delve into all aspects of water resources development problems. It has been recognized by all that a single study is not sufficient to deal with all facets of a very involved problem like the evaluation of all water resources development alternatives. This study concentrated on the development of methodology for rapid evaluation of investment costs of reservoir, levee and basin conservation reservoir projects.

Continued and concerted research efforts are necessary to find a complete solution of the problem. More studies are needed also for the refinement of methodologies. A few of the necessary studies are proposed:

1. Research should be undertaken to collect definite project completion data for reservoirs and levees in the State of Texas with a view to develop statistical equations for the relations developed in Fig. 1 (p. 24), Fig. 2 (p. 25) and Fig. 4 (p. 34).
2. Studies should be conducted to gather and to compile data from different economic regions of the United States to develop relations, as suggested in 1. Finally, a generalized model that represents the relation may be derived. All relationships and the generalized model should be verified by case study.
3. Investigations should be undertaken to determine the environmental impact of levees and desalting plants in the case study area used in this research. Relative effects of this alternative with respect to the multiple purpose reservoir at Millican should be evaluated.
4. Investigations should be carried out to study the feasibility of channel improvement as a measure of flood relief, and recycling and reuse of waste water for water supply in the case study area.

5. Effects of zoning and flood insurance for mitigation of flood losses in the affected areas, and limiting the water use and industrial growth in the most rapidly growing locality should also be considered for evaluation.
6. A method of transposition of costs was suggested in Chapter III. Research efforts should be directed to develop refined techniques based on practical data for transfering costs to similar regions where information on existing structures is not available.

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APPENDICES

APPENDIX A

RESERVOIR DATA

TABLE A-1 - STORAGE CAPACITY AND CONSTRUCTION COST OF SOME RESERVOIRS IN TEXAS (Reference: 66)

River Basin	Name of Reservoir	Storage Capacity, million acre-feet	Estimated Construction Cost, (July 1965 price level), million dollars
Sulphur	Naples I	2.0570	74.9
Brazos	Navasota No. 2	1.9356	61.1
San Antonio	Goliad	1.7020	50.5
Brazos	Millican	1.5557	58.6
Neches	Ponta	1.4846	51.8
Colorado	Stacy	1.3593	26.4
Brazos	Cameron	1.1280	32.5
Trinity	Richland Creek	1.1355	30.0
Colorado	Columbus Bend	0.9650	44.2
Trinity	Aubrey	0.8999	34.1
Sabine	Lake Fork	0.8753	45.9
Sabin	Mineola	0.8469	49.5
Cypress	Marshal	0.7823	25.1
Nueces	Choke Canyon	0.7000	31.9
Sulphur	Sluphur Bluff I	0.6354	31.2
Brazos	Breckenridge	0.6170	19.6
Trinity	Bedias	0.5047	25.2

TABLE A-1 (Cont'd)

River Basin	Name of Reservoir	Storage Capacity, million acre-feet	Estimated Construction Cost, (July 1965 price level), million dollars
Colorado	Robert Lee	0.4888	12.8
Trinity	Lakeview	0.4887	31.8
San Jacinto	Cleveland	0.4840	18.7
Cypress	Titus County	0.4227	12.0
San Antonio	Cibolo	0.4180	26.0
Sulphur	Cooper	0.4098	18.0
Trinity	Tehuacana Creek	0.4075	19.7
Red	Pecan Bayou	0.3833	16.6
San Jacinto	Lower East Fork	0.3380	35.1
Guadalupe	Cloptin Crossing	0.257	14.5
Brazos	Laneport	0.2442	32.2
Colorado	Upper Pecan Bayou	0.2063	10.5
Brazos	Aquilla Creek	0.1993	23.5
Red	Bois d'Arc	0.1795	13.9
Brazos	De Cordova Bend	0.1500	15.0
Guadalupe	Ingram	0.0904	8.5

TABLE A-2 - STORAGE CAPACITY AND POOL AREA OF RESERVOIRS
(References: 73, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85)

Sl. No.	Name of Reservoir	Storage Capacity at Maximum Design Water Surface, million acre-feet	Pool Area at Maximum Design Water Surface, thousand acres
1	Bardwell	0.2684	9.480
2	Navarro Mills	0.3358	15.950
3	Proctor	0.4330	15.410
4	San Angelo	0.728941	19.128
5	Waco	0.8322	21.350
6	Lavon	0.9212	33.500
7	Stillhouse Hollow	1.0087	16.310
8	Somerville	1.0288	39.800
9	Kaw	1.2850	38.900
10	Garza-Little-Elm	2.0512	66.100
11	Ferguson No. 3	2.1420	71.150
12	Millican	2.1991	83.300
13	Navasota No. 2	2.2776	69.440

TABLE A-3 - SUMMARY OF COST ANALYSIS OF SOME RESERVOIRS WITH ROCKFILL AND EARTHFILL DAMS IN TEXAS
(References: 73, 75, 76, 77, 78, 80, 81, 82, 83, 84, 86, 87, 88, 89)

Name of Reservoir	Time of Cost Estimate	Total Estimated Cost, million dollars	Component Costs, million dollars						
			Land	Relocation	Reservoir and Dam	Recreation	Engineering Design	Supervision & Admn.	Misc.
South Fork	July, 1966	12.400	2.858	0.267	7.332	0.458	0.825	0.480	0.180
Aquilla	Jan., 1965	23.300	5.187	3.063	11.526	1.154	1.180	0.996	0.194
North Fork	July, 1966	15.200	2.821	1.068	8.190	0.885	1.100	0.820	0.316
Bardwell	Oct., 1962	12.800	2.770	3.009	4.812	0.600	0.754	0.558	0.297
Laneport	July, 1962	32.900	9.829	0.933	17.938	0.605	1.983	1.260	0.352
Navarro Mills	July, 1959	10.390	2.328	1.935	4.522	0.225	0.560	0.595	0.225
Proctor	Dec., 1959	15.470	2.778	1.383	8.653	0.500	0.860	0.950	0.346
Waco	Feb., 1958	39.100	11.937	5.509	17.243	0.425	1.566	2.202	0.218
Lavon	Jan., 1966	31.400	10.348	9.717	7.287	1.277	1.465	1.284	0.022
Stillhouse Hollow	Feb., 1966	19.400	2.815	0.463	13.115	0.505	1.053	1.110	0.339
Somerville	July, 1961	16.900	5.660	0.725	8.254	0.549	0.600	0.757	0.355
Kaw	Spring, 1961	83.300	22.940	28.654	23.777	0.761	2.973	3.705	0.490
Ferguson	July, 1961	47.700	14.266	7.970	19.935	1.650	1.450	2.024	0.405
Millican	Jan., 1971	104.809	39.524	26.119	21.056	10.098	4.220	3.177	0.615
Navasota No. 2	Jan., 1971	121.357	31.036	41.910	30.257	7.124	5.652	4.733	0.645

TABLE A-4 - PERCENTAGE OF COMPONENT COST OF SOME RESERVOIR PROJECTS IN TEXAS
(References: 73, 75, 76, 77, 78, 80, 81, 82, 83, 84, 86, 87, 88, 89)

Name of Reservoir	Storage Capacity, million acre-feet	Percentage of Total Cost						
		Land	Relocation	Reservoir and Dam	Recreation	Engi-neering Design	Super-vision & Admn.	Misc.
South Fork	0.1440	23.0	2.3	59.0	3.7	6.6	4.0	1.4
Aquilla	0.1993	22.3	13.2	49.4	4.8	5.5	4.0	0.8
North Fork	0.2342	18.7	7.0	53.8	5.8	7.3	5.4	2.0
Bardwell	0.2684	21.7	23.4	37.6	4.7	5.9	4.4	2.3
Laneport	0.3165	30.0	2.8	54.6	1.8	6.0	3.8	1.0
Navarro Mills	0.3358	22.3	18.5	43.3	2.0	5.2	6.5	2.2
Proctor	0.4330	18.0	8.9	56.0	3.2	5.5	6.1	2.3
Waco	0.8322	30.5	14.1	44.0	1.1	4.0	5.6	0.7
Lavon	0.9212	33.0	31.0	23.0	4.0	4.3	4.0	0.7
Stillhous Hollow	1.0087	14.5	2.4	67.3	2.6	5.5	5.8	1.9
Somerville	1.0288	33.5	4.3	48.7	3.3	3.6	4.5	2.1
Kaw	1.2850	27.5	34.4	28.6	0.9	3.6	4.4	0.6
Ferguson No. 3	2.1420	30.0	16.7	41.7	3.3	3.2	4.3	0.8
Millican	2.1990	37.5	25.0	19.7	10.2	4.0	3.0	0.6
Navasota No. 2	2.2770	26.0	34.5	24.8	5.8	4.6	3.8	0.5
	TOTAL	388.5	238.5	651.5	57.2	74.8	69.6	19.9
	AVERAGE	26.0	16.0	43.4	3.8	5.0	4.6	1.2

APPENDIX B

LEVEE DATA AND DESIGN

TABLE B-1.-COMPUTATION OF COST DATA FOR FIG. 3 (p. 32)
(January, 1973 Price Level)

Height of Levee, ft	Cross-sectional Area of Levee		Volume, cu yd/ft	Cost of Construction	
	sq ft	sq yd		dollars/ft	dollars/mile
28	2856	317.3	105.8	90.6	478,000
26	2522	280.2	93.4	80.3	424,000
24	2160	240.0	80.0	68.7	362,000
22	1862	206.9	67.6	58.0	306,000
20	1520	168.9	56.4	48.4	255,400
18	1242	138.0	46.0	39.8	210,000
16	992	110.2	36.7	31.6	166,700
14	770	85.6	28.5	24.5	129,200
13	670	74.4	24.8	21.3	112,500
12	576	64.0	21.3	18.4	96,800
11	490	54.4	18.1	15.6	82,400
10	410	45.6	15.2	13.1	69.00
8	272	30.2	10.1	8.7	45.600
6	162	18.0	6.0	5.2	27,200
4	80	8.9	3.0	2.6	13,500

NOTE: Crown width of levee: 6 feet

Unit cost of construction: $0.86 per cu yd (65)

Side slopes of levee: River side 3(H):1(V)

Land side 4(H):1(V)

TABLE B-2

DATA ON LEVEE CONSTRUCTION IN TEXAS
(Reference: 100)

Average height of levee feet	Length of levee miles	Estimated Cost at 1929 Price Level: Total Cost of project, dollars	Estimated Cost at 1929 Price Level: Cost per mile of levee, dollars	Cost per mile of levee at January, 1973 price level, dollars
6	8.86	50,000	5,640	49,300
7	13.03	59,000	4,540	39,700
10	27.07	285,320	10,520	92,000
18	20.13	355,000	17,650	154,400
3.5	7.90	6,000	760	6,650
9	5.50	18,500	3,360	29,400
6.5	16.00	57,000	3,565	31,200
9	5.53	62,500	11,300	98,800
18	6.90	200,000	29,000	253,900
14	9.03	220,000	24,350	213,000
15	9.58	222,000	23,200	203,000
15.5	13.24	342,000	25,800	225,900
20	16.08	664,700	41,250	361,000
14	12.82	288,000	22,450	196,500
12	4.65	71,000	15,600	136,500
10	7.86	55,000	7,000	61,250
10	2.40	20,000	8,350	73,000
14	8.05	120,000	14,900	130,300
12	2.33	32,600	14,000	122,500
16	3.30	50,000	15,150	132,400
12	6.44	77,000	11,980	104,800
9	5.45	25,000	4,600	40,200
8	2.89	10,000	3,460	30,250
12.5	1.58	17,500	11,100	97,100
10	9.43	62,500	6,630	58,600
13	2.96	55,000	18,980	162,800
18	2.74	52,000	18,900	166,000
12	10.00	89,000	8,900	77,800
20.5	10.12	235,000	23,200	203,000
17	27.82	700,000	25,150	220,000
9	4.40	50,000	11,380	99,500
14	9.92	193,000	19,450	170,000
16	7.76	125,000	16,100	140,900
12	10.03	150,000	15,000	131,250

TABLE B-2 (Cont'd)

Average height of levee feet	Length of levee miles	Estimated Cost at 1929 Price Level		Cost per mile of levee at January, 1973 price level, dollars
		Total Cost of project, dollars	Cost per mile of levee, dollars	
16	6.63	165,000	24,900	218,000
15	3.79	60,000	15,800	138,200
9.5	4.42	24,561	5,550	48,600
12	4.22	30,000	7,120	62,300
12	3.50	47,000	13,400	117,200
12	6.38	67,000	10,500	91,800
14.5	5.16	46,000	9,000	78,700
9	26.76	152,073	5,680	49,700
10	3.88	22,000	5,670	49,600
10.5	8.69	113,000	13,000	113,800
18.5	6.55	187,000	28,600	250,000
16	1.63	30,000	18,400	161,000
15.5	13.98	262,000	18,750	164,000
13.5	5.79	82,000	14,200	124,000
18	6.03	150,000	24,950	218,500
11	5.17	47,500	9,170	80,200
14	8.73	135,000	15,500	135,600
12	3.06	50,000	16,350	143,000
16	2.60	50,000	19,250	168,500

TABLE B-3 - DETERMINATION OF 6-HOUR VALUES OF 100-YEAR-FREQUENCY FLOOD HYDROGRAPH IN BRAZOS RIVER AT THE MOUTH OF NAVASOTA RIVER (Reference: 74)

Time		Instantaneous Flow from Navasota River cfs	Base Flow in Brazos River, cfs	Total Flow, cfs
Day	Hour			
1	00	4,000	30,000	34,000
	06	4,750	"	34,750
	12	6,000	"	36,000
	18	21,000	"	51,000
2	00	36,000	"	66,000
	06	52,000	"	82,000
	12	64,000	"	94,000
	18	77,000	"	107,000
3	00	88,000	"	118,000
	06	105,000	"	135,000
	12	107,000	"	137,000
	18	103,000	"	133,000
4	00	90,000	"	123,000
	06	79,000	"	109,000
	12	72,000	"	102,000
	18	67,000	"	97,000
5	00	62,000	"	92,000
	06	58,000	"	88,000
	12	55,000	"	85,000
	18	51,500	"	81,500
6	00	48,500	"	78,500
	06	46,000	"	76,000
	12	44,000	"	74,000
	18	42,500	"	72,500
7	00	41,000	"	71,000
	06	39,000	"	69,000
	12	37,500	"	67,500
	18	35,000	"	65,000
8	00	32,000	"	62,000
	06	26,500	"	56,500
	12	18,000	"	48,000
	18	12,000	"	42,000

NOTE: Instantaneous flows from the Navasota River were read from the hydrograph of 100-year-frequency flood, Fig. 8 (p. 60) in Chapter V.

Determination of Coefficients for Muskingum Method of Flood Routing

Linsley, Kohler and Paulhus (40) stated that:

(a) the storage constant, K, is approximately equal to the time of travel through the reach and, in absence of better data, is sometimes estimated in this way;

(b) the constant x expresses the relative importance of inflow and ourflow in determining storage and, for most streams, x is between 0 and 0.3 with a mean value near 0.2; and

(c) the routing period, t, should never be greater than the time of travel through the reach and, generally, a routing period between one-half and one-third of the travel time will work quite well.

Reach 1.

Reach length = 232.00 - 197.14 = 34.86 miles

Mean velocity of flow in the reach at bankful stage of stream =

$$\frac{3.52 + 3.17 + 2.98}{3} = 3.22 \text{ ft/sec (Reference: Table 6)}$$

Average time of travel through the reach = $\frac{34.86 \times 5280}{3.22 \times 3600}$ = 16 hrs.

Assumptions: K = 16 hrs. = 0.67 day,

x = 0.2, and

t = 6 hrs = 0.25 day

Coefficients: $C_o = \dfrac{Kx - 0.5t}{K-Kx + 0.5t} = -0.014$

$$C_1 = \frac{Kx + 0.5t}{K-Kx + 0.55} = 0.392$$

$$C_2 = \frac{K-Kx - 0.5t}{K-Kx + 0.5t} = 0.622$$

$$C_o + C_1 + C_2 = 1, \text{ checked}$$

Likewise, the coefficients for other reaches were computed and are given in the following tabulation:

Reach	K, day	t, day	x	C_o	C_1	C_2
1	0.67	0.25	0.2	-0.014	0.392	0.622
2	0.75	0.25	0.2	-0.035	0.380	0.655
3	0.92	0.25	0.2	-0.068	0.358	0.710
4	0.75	0.25	0.2	-0.035	0.380	0.655
5	0.71	0.25	0.2	-0.025	0.385	0.640
6	Not completed					

Definition of reach:

Reach 2 - From river mile 197.14 to 161.27 = 35.87 miles

Reach 3 - From river mile 161.27 to 118.82 = 42.45 miles

Reach 4 - From river mile 118.82 to 79.93 = 38.89 miles

Reach 5 - From river mile 79.93 to 38.75 = 41.18 miles

Reach 6 - From river mile 38.75 to 16.86 = 21.89 miles

TABLE B-4 - FLOOD ROUTING BY MUSKINGUM METHOD THROUGH LOWER BRAZOS RIVER

(a) Reach 1: From river mile 232 to 197.14

Time		Inflow, I, cfs	C_oI_2, cfs	C_1I_1, cfs	C_2O_1, cfs	Outflow, O, cfs
Day	Hour					
1	00	34,000				30,000
	06	34,750	- 485	13,330	18,700	31,545
	12	36,000	- 505	13,600	19,620	32,715
	18	51,000	- 715	14,100	20,350	33,735
2	00	66,000	- 925	20,000	21,000	40,075
	06	82,000	- 1150	25,820	24,950	49,620
	12	94,000	- 1320	32,150	30,900	61,730
	18	107,000	- 1500	36,800	38,400	73,700
3	00	118,000	- 1650	42,000	45,800	86,150
	06	135,000	- 1890	46,300	53,600	97,010
	12	137,000*	- 1920	53,000	60,400	111,480
	18	133,000	- 1860	53,700	69,400	121,240
4	00	123,000	- 1720	52,200	75,300	125,780*
	06	109,000	- 1525	48,300	78,200	124,975
	12	102,000	- 1425	42,700	77,600	118,875
	18	97,000	- 1360	40,000	73,800	112,440
5	00	92,000	- 1290	38,000	70,000	106,710
	06	88,000	- 1230	36,000	66,400	101,170
	12	85,000	- 1190	34,500	63,000	96,310
	18	81,500	- 1140	34,300	60,000	93,160
6	00	78,500	- 1090	31,950	58,000	88,860
	06	76,000	- 1060	30,800	55,200	84,940
	12	74,000	- 1035	29,800	52,800	81,565
	18	72,500	- 1015	29,000	50,700	78,685
7	00	71,000	- 990	28,400	49,000	76,410
	06	69,000	- 965	27,800	47,600	75,430
	12	67,500	- 945	27,100	46,800	72,955
	18	65,000	- 910	26,450	45,400	70,940
8	00	62,000	- 870	25,500	44,100	68,730
	06	56,500	- 790	24,300	42,800	66,310
	12	48,000	- 670	22,200	41,200	62,730
	18	42,000	- 590	18,800	39,000	57,210

Inflow is taken from Table B-3

* Peak flows

TABLE B-4 (con't)

Similarly, flood routing through the remaining reaches were performed using ourflows of the upstream reach as the inflows for the immediate downstream reach, e.g., outflows of reach 1 were the inflows of reach 2 and so on. The outflows from different reaches are shown below.

Time		Outflows, cfs				
Day	Hour	Reach 1	Reach 2	Reach 3	Reach 4	Reach 5
1	00	30,000	30,000	30,000	30,000	30,000
	06	31,545	30,350	30,000	30,000	30,000
	12	32,715	30,650	30,070	30,000	30,000
	18	33,735	31,320	30,190	30,015	30,000
2	00	40,075	31,900	30,430	30,060	30,000
	06	49,620	34,360	30,590	30,130	30,022
	12	61,730	39,190	31,340	30,290	30,060
	18	73,700	46,420	33,050	30,720	30,155
3	00	86,150	55,330	36,290	31,380	30,220
	06	97,010	65,300	41,140	32,860	30,620
	12	111,480	75,890	47,490	35,440	31,315
	18	121,240	87,760	54,840	39,280	32,670
4	00	125,780*	99,100	63,560	44,280	34,900
	06	124,975	108,240	73,250	50,640	38,090
	12	118,875	114,250	82,940	58,150	42,250
	18	112,440	116,070*	91,700	66,400	48,850
5	00	106,710	114,970	98,900	74,890	54,930
	06	101,170	112,260	103,700	82,980	61,830
	12	96,310	108,530	106,340	89,880	69,260
	18	93,160	104,340	107,220*	95,650	76,400
6	00	88,860	100,690	103,560	99,770	83,100
	06	84,940	96,330	102,950	101,200	89,070
	12	81,565	92,440	101,230	101,760*	93,460
	18	78,685	88,730	98,770	100,650	96,290
7	00	76,410	85,330	96,000	99,540	97,810
	06	75,430	82,160	93,000	98,450	98,440
	12	72,955	79,750	90,070	96,750	98,480*
	18	70,940	77,520	87,230	94,550	97,840
8	00	68,730	75,290	84,580	92,240	96,700
	06	66,310	73,080	82,040	89,730	95,260
	12	62,730	70,710	79,600	87,110	93,320
	18	57,210	68,100	77,270	84,500	90,090

* Peak flows

TABLE B-4 (Cont'd)

(b) Reach 6:

It is observed from the foregoing computations that the rate of decrease in the peak flow has gradually become small as the routing continues from upper to lower reaches. Moreover, the length of this reach is about one-half of the other reaches. The change of peak will be much smaller. Therefore, it was assumed that the inflow peak in the reach was the average peak in the reach.

Levee Design

Definition of Symbols used:

A_r = area of river section, sq ft

A_f = area of floodway section, sq ft

d = distance of river side edge of levee from river bank, ft

n = Manning roughness coefficient

= 0.05 for river section and 0.07 for floodway

P_r = wetted perimeter of river section, ft

P_f = wetted perimeter of floodway section, ft

Q = discharge, cfs

Q_p = design peak discharge, cfs

Q_r = discharge through river section, cfs

Q_f = discharge through floodway section, cfs

R_r = hydraulic radius of river section, ft

R_f = hydraulic radius of floodway section, ft

s = slope of river bed and floodway = 0.000142

T = top width of leveed flow section, ft

T_r = top width of river section

U_r = mean velocity of flow through river section, ft/sec

U_f = mean velocity of flow through floodway section, ft/sec

y = depth of flow above bankful stage of river, ft

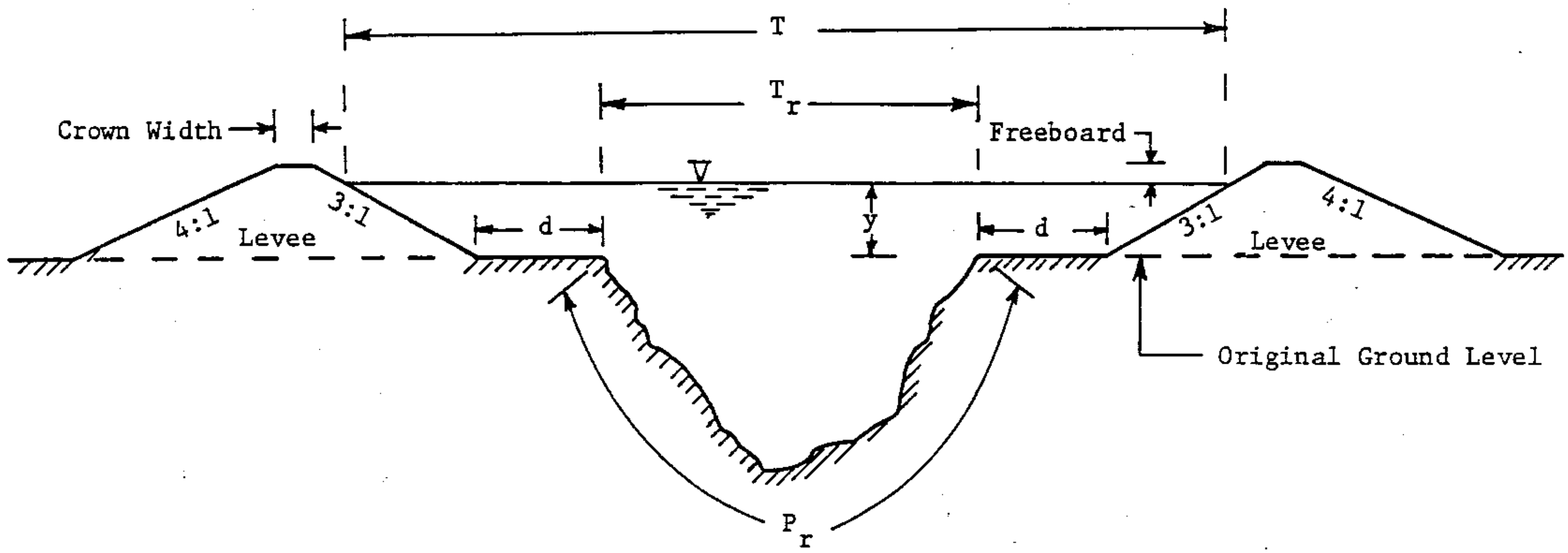

FIG. B-1.-DEFINITION SKETCH OF LEVEE DESIGN

Levee Design (Cont'd)

Reach 1: From river mile 232 to 197.14

$$Q_p = \frac{137{,}000 + 125{,}780}{2} = 131{,}390 \text{ cfs}$$

Side slope of levee = 3(H):1(V) on river side.

Spacing and height of levee is calculated by trial, that is, either the value of d is assumed and the value of y is computed or vice-versa. These two elements are determined such that the leveed section can accommodate and discharge the design flood peak without overflowing the levee. Two sample computations are presented below.

(1) $d = 0$ ft (assumed)

$y = 21$ ft

$T = 613 + 2 \times 63 = 739$ ft

$T_r = 613$ ft

$P_r = 638 + 2 \times 66.4 = 771$ ft

$A_r = 17{,}096 + 1/2(613 + 739) \times 21 = 31{,}296$ sq ft

$R_r = A_r \div P_r = 31{,}296 \div 771 = 40.6$ ft

$n = 0.05$

$s = 0.000142$

$$U_r = \frac{1.49}{n} s^{1/2} R_r^{2/3} = \frac{1.49}{0.05} (0.000142)^{1/2} R_r^{2/3} = 0.355 R_r^{2/3}$$

$= 4.24$ ft/sec

$Q_r = A_r U_r = 31{,}296 \times 4.24 = 132{,}600$ cfs

Levee Design (Cont'd)

(2) $y = 15$ ft (assumed)

$d = 410$ ft

$T = 613 + 2 \times 410 + 2 \times 45 = 1523$ ft

$T_r = 613$ ft

$P_r = 638$ ft

$A_r = 17{,}096 + 613 \times 15 = 26{,}291$ sq ft

$R_r = A_r \div P_r = 26{,}291 \div 638 = 41.2$ ft

$n = 0.05$

$s = 0.000142$

$U_r = 0.355\ R_r^{2/3} = 4.24$ ft/sec

$Q_r = A_r\ U_r = 26{,}291 \times 4.24 = 112{,}000$ cfs

$A_f = (410 \times 15) \times 2 + 2 \times \frac{1}{2} \times 45 \times 15 = 12{,}975$ sq ft

$P_f = 2 \times 410 + 2 \times 47.5 = 915$ ft

$R_f = A_f \div P_f = 12{,}975 \div 915 = 14.2$ ft

$$U_f = \frac{1.49}{n} s^{1/2} R_f^{2/3} = \frac{1.49}{0.07} \times (0.000142)^{1/2} R_f^{2/3} = 0.254 R_f^{2/3}$$

$= 1.5$ ft/sec

$Q_f = A_f\ U_f = 12{,}975 \times 1.5 = 19{,}500$ cfs

$Q = Q_r + Q_f = 112{,}000 + 19{,}500 = 131{,}500$ cfs

$Q_p = 131{,}390$ cfs

TABLE B-5.-SUMMARY OF LEVEE DESIGN

$s = 0.000142$, $n = 0.05$ for river section, 0.07 for floodway

(a) Reach 1:

	y = 21	y = 15	y = 12	y = 10	y = 7	y = 5
d	0	410	1,000	1,700	3,965	7,920
T	739	1,523	2,685	4,073	8,585	16,483
T_r	613	613	613	613	613	613
P_r	771	638	638	638	638	638
A_r	31,296	26,291	24,452	23,226	21,387	20,161
R_r	40.6	41.2	38.4	36.5	33.5	31.6
U_r	4.24	4.24	4.08	3.94	3.73	3.58
Q_r	132,600	112,000	99,800	91,500	79,500	72,200
A_f		12,975	24,432	34,300	55,657	79,200
P_f		915	2,076	3,463	7,974	15,872
R_f		14.2	11.8	9.92	6.98	5
U_f		1.50	1.33	1.18	0.93	0.75
Q_f		19,500	32,400	40,500	52,000	59,400
Q	132,600	131,500	132,200	132,000	131,500	131,600
Q_p	131,390	131,390	131,390	131,390	131,390	131,390

(b) Reaches 2, 3 and 4:

	y = 21	y = 12	y = 11	y = 9	y = 7	y = 5
d	0	190	405	1,100	2,365	5,280
T	884	1,252	1,676	3,054	5,572	11,390
T_r	800	800	800	800	800	800
P_r	904	816	816	816	816	816
A_r	30,179	27,991	27,191	25,591	23,991	22,391
R_r	33.4	34.3	33.3	31.3	29.4	27.4
U_r	3.73	3.80	3.73	3.55	3.40	3.27
Q_r	112,500	106,200	101,200	90,600	81,600	73,000
A_f		4,992	9,273	20,043	33,100	52,800
P_f		456	880	2,257	4,774	10,312
R_f		10.95	10.55	8.90	6.94	5
U_f		1.26	1.23	1.10	0.94	0.75
Q_f		6,300	11,400	22,000	31,100	39,600
Q	112,500	112,500	112,600	112,600	112,700	112,600
Q_p	112,443	112,443	112,443	112,443	112,443	112,443

TABLE B-5 (Cont'd)

(c) Reaches 5 and 6:

	y = 11	y = 9	y = 8	y = 7	y = 5
d	0	140	450	870	2,550
T	689	957	1,571	2,405	5,753
T_r	623	623	623	623	623
P_r	711	641	641	641	641
A_r	25,682	24,053	23,430	22,807	21,561
R_r	36.1	37.5	36.6	35.5	33.6
U_r	3.90	4.03	3.94	3.86	3.73
Q_r	100,000	96,500	92,200	88,000	80,300
A_f		2,763	7,392	12,327	25,555
P_f		337	951	1,784	5,132
R_f		8.20	7.77	6.90	4.98
U_f		1.04	1.00	0.93	0.75
Q_f		2,870	7,392	11,400	19,050
Q	100,000	99,370	99,592	99,400	99,350
Q_p	99,300	99,300	99,300	99,300	99,300

TABLE B-6 - ELEMENTS OF DESIGNED LEVEES

Crown width of levee = 6 ft, Freeboard = 3 ft,

side slopes: 3(H):1(V) on river side and 4(H):1(V) on land side

Reach	Length of Reach, mi	Design Flood Peak, cfs	Capacity of Leveed Section, cfs	Distance of Levee Edge from River Bank, ft	Depth of Water on Floodway, ft	Height of Levee, ft	Width of Berm, ft	Width of Land Required, ft
1	34.86	131,390	132,600	0	21	24	36	420
			131,500	410	15	18	30	572
			132,200	1,000	12	15	25	1,136
			132,000	1,700	10	13	20	1,817
			131,500	3,965	7	10	15	4,056
			131,400	7,920	5	8	12	7,994
2, 3, and 4	117.21	112,443	112,500	0	14	17	25	300
			112,500	190	12	15	25	326
			112,600	405	11	14	21	530
			112,600	1,100	9	12	18	1,208
			112,500	2,365	7	10	15	2,456
			112,500	6,500	5	8	12	6,574
5 and 6	63.07	99,300	100,000	0	11	14	21	250
			99,370	140	9	12	18	248
			99,592	450	8	11	17	550
			99,400	870	7	10	15	961
			99,350	2,550	5	8	12	2,624

TABLE B-7 - COMPUTATION OF COST FOR DESIGNED LEVEE

Reach	Length of Reach mi	Height of Levee ft	Land Area per Mile of Levee, acres	Cost per Mile of Levee, January 1973 price level			
				Land @ $458 per acre, dollars	Cost (from Fig. 3), dollars	Total Cost, dollars	Minimum Cost, dollars
1	34.86	24	51	23,000	362,000	385,300	211,200
		18	70	32,000	210,000	243,000	
		15	138	63,200	148,000	211,200	
		13	220	100,900	112,500	213,400	
		10	492	224,800	69,000	293,800	
		8	970	444,000	45,600	489,600	
2, 3, and 4	117.21	17	36	16,500	188,000	204,500	158,500
		15	39.4	18,050	144,000	162,050	
		14	64	29,300	129,200	158,500	
		12	146	67,000	96,800	163,800	
		10	297	136,000	69,000	205,000	
		8	795	364,000	45,600	409,600	
5 and 6	63.07	14	30.2	13,800	129,200	143,000	110,500
		12	30	13,700	96,800	110,500	
		11	66.5	30,400	82,400	112,800	
		10	116	53,400	69,000	122,200	
		8	317.5	145,000	45,600	190,600	

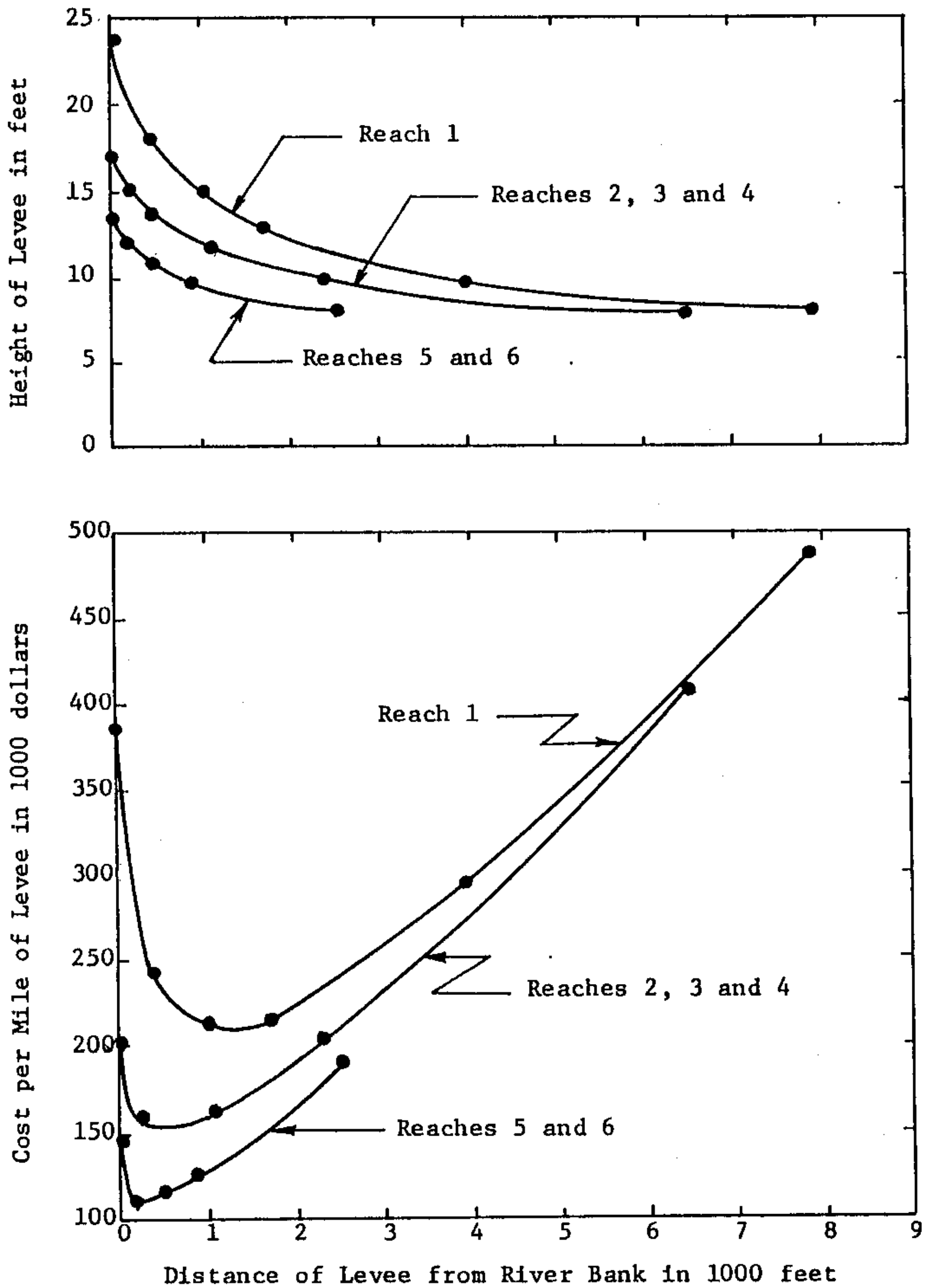

FIG. B-2.-RELATIONSHIP BETWEEN DISTANCE FROM RIVER BANK, COST PER MILE, AND HEIGHT OF LEVEE

APPENDIX C

BASIN CONSERVATION RESERVOIR DATA

TABLE C-1 - FLOOD PREVENTION BY BASIN CONSERVATION RESERVOIR - DATA FOR TEXAS
(Reference: 91)

Name of the River Basin and Project Status	Drainage Area Controlled, sq mi	Number of Structure	Storage Capacity, acre-feet	Average Storage Capacity of Each Structure, acre-feet	Average Area Controlled by Each Structure, sq mi
Neches River					
a. Project Complete	0	0	0	0	0
b. Potential Project	562	103	216,976	2,106	5.45
Total of (a) and (b)	562	103	216,976	2,106	5.45
Trinity River					
a. Project Complete	574	223	204,671	917	2.54
b. Potential Project	3,105	977	1,097,295	1,120	3.17
Total of (a) and (b)	3,679	1,200	1,301,966	1,085	3.07
San Jacinto River					
a. Project Complete	0	0	0	0	0
b. Potential Project	45	14	18,221	1,300	3.21
Total of (a) and (b)	45	14	18,221	1,300	3.21
Brazos River					
a. Project Complete	166	34	58,586	1,725	4.89
b. Potential Project	4,067	805	1,395,650	1,733	5.05
Total of (a) and (b)	4,233	839	1,454,236	1,730	5.04

TABLE C-1 (Cont'd)

Name of the River Basin and Project Status	Drainage Area Controlled, sq mi	Number of Structure	Storage Capacity, acre-feet	Average Storage Capacity of Each Structure, acre-feet	Average Area Controlled by Each Structure, sq mi
Colorado River					
a. Project Complete	846	109	258,521	2,370	7.76
b. Potential Project	1,496	269	396,382	1,472	5.56
Total of (a) and (b)	2,342	378	654,903	1,730	6.19
Guadalupe River					
a. Project Complete	0	0	0	0	0
b. Potential Project	482	70	147,234	2,100	6.88
Total of (a) and (b)	482	70	147,234	2,100	6.88
San Antonio River					
a. Project Complete	84	20	29,466	1,473	4.20
b. Potential Project	382	85	123,990	1,460	4.50
Total of (a) and (b)	466	105	153,456	1,465	4.43
Nueces River					
a. Project Complete	0	0	0	0	0
b. Potential Project	391	36	89,298	2,480	10.86
Total of (a) and (b)	391	36	89,298	2,480	10.86
Project Complete	1,670	386	551,244	1,427	4.33
Potential Projects	10,530	2,359	3,485,049	1,480	4.47
Combined	12,200	2,745	4,036,293	1,470	4.45

TABLE C-2 - COST OF POTENTIAL BASIN CONSERVATION RESERVOIR PROJECTS IN TEXAS
(Reference: 91)

Name of the River Basin	Number of Structure	Cost at 1961 Price Level			
		Total of All Structures		Average for One Structure	
		Installation, dollars	Annual Charge, dollars	Installation, dollars	Annual Charge, dollars
Neches River	103	7,002,800	314,610	67,988	3,054
Trinity River	977	75,079,400	2,907,870	77,870	2,976
San Jacinto River	14	712,700	28,500	50,907	2,035
Brazos River	805	65,513,000	2,753,280	81,382	3,420
Colorado River	269	18,283,600	735,540	67,968	2,734
Guadalupe River	70	9,942,000	389,669	142,028	5,566
San Antonio River	85	6,166,800	243,120	72,550	2,860
Nueces River	36	5,284,200	207,300	146,783	5,758
All Structures	2,359	187,984,500	7,779,889	79,800	3,300

TABLE C-3 - COMPUTATION OF AVERAGE QUANTITIES FOR POTENTIAL BASIN CONSERVATION PROJECTS IN TEXAS
(Reference: 91)

Name of the River Basin	Average Drainage Area Controlled by One Structure, sq mi	Average Storage Capacity of One Structure, acre-feet	Average Cost for One Structure at 1961 Price Level	
			Installation Cost, dollars	Annual Charge, dollars
Neches River	5.45	2,106	67,988	3,054
Trinity River	3.17	1,120	77,870	2,976
San Jacinto River	3.21	1,300	50,907	2,035
Brazos River	5.05	1,733	81,382	3,420
Colorado River	5.56	1,472	67,968	2,734
Guadalupe River	6.88	2,100	142,028	5,566
San Antonio River	4.50	1,460	72,550	2,860
Nueces River	10.86	2,480	146,783	5,758
Total	44.68	13,771	707,476	28,403
Average	5.58	1,720	88,436	3,550

APPENDIX D

CROSS-SECTIONS OF THE BRAZOS RIVER

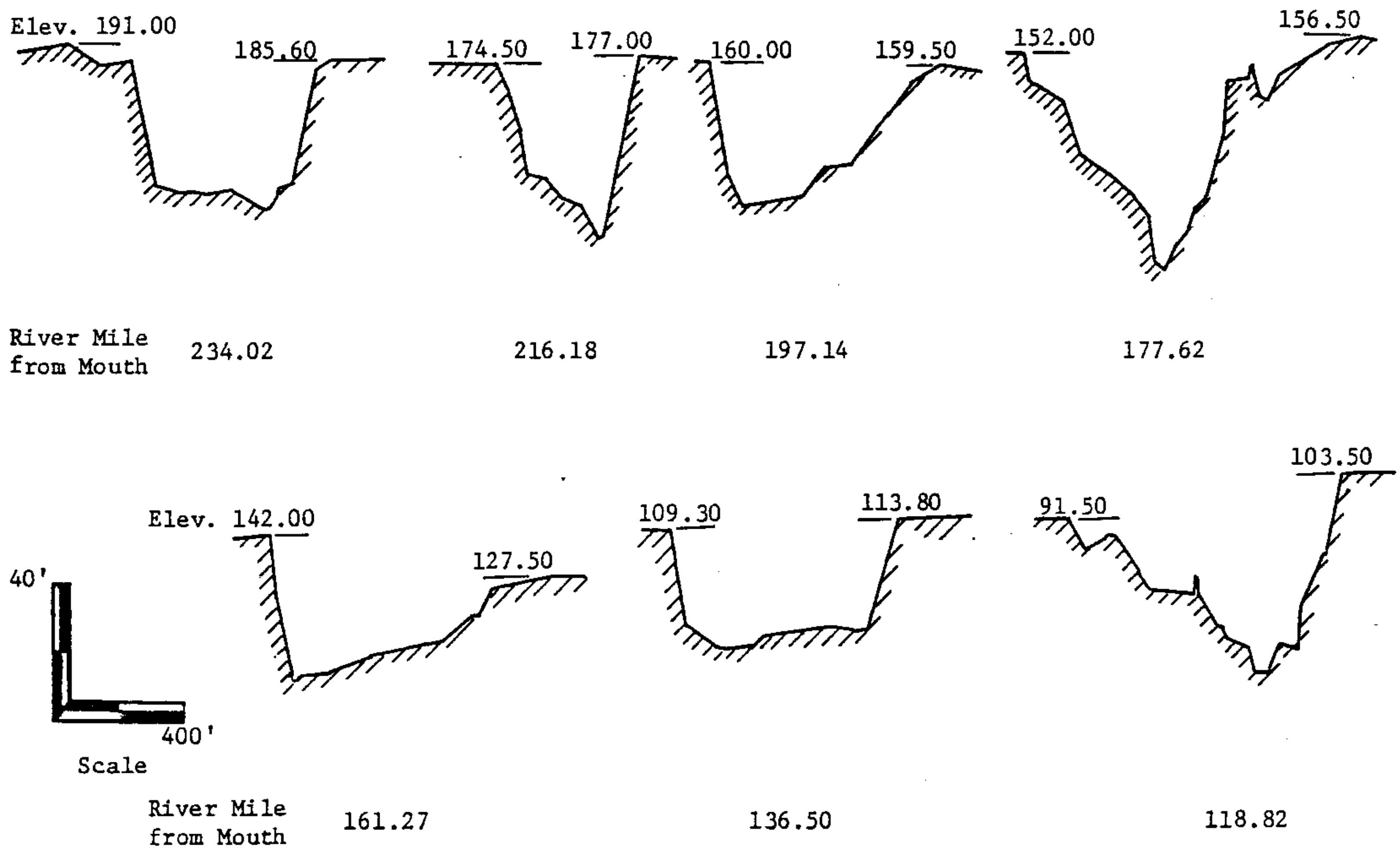

FIG. D-1.-CROSS-SECTIONS OF BRAZOS RIVER

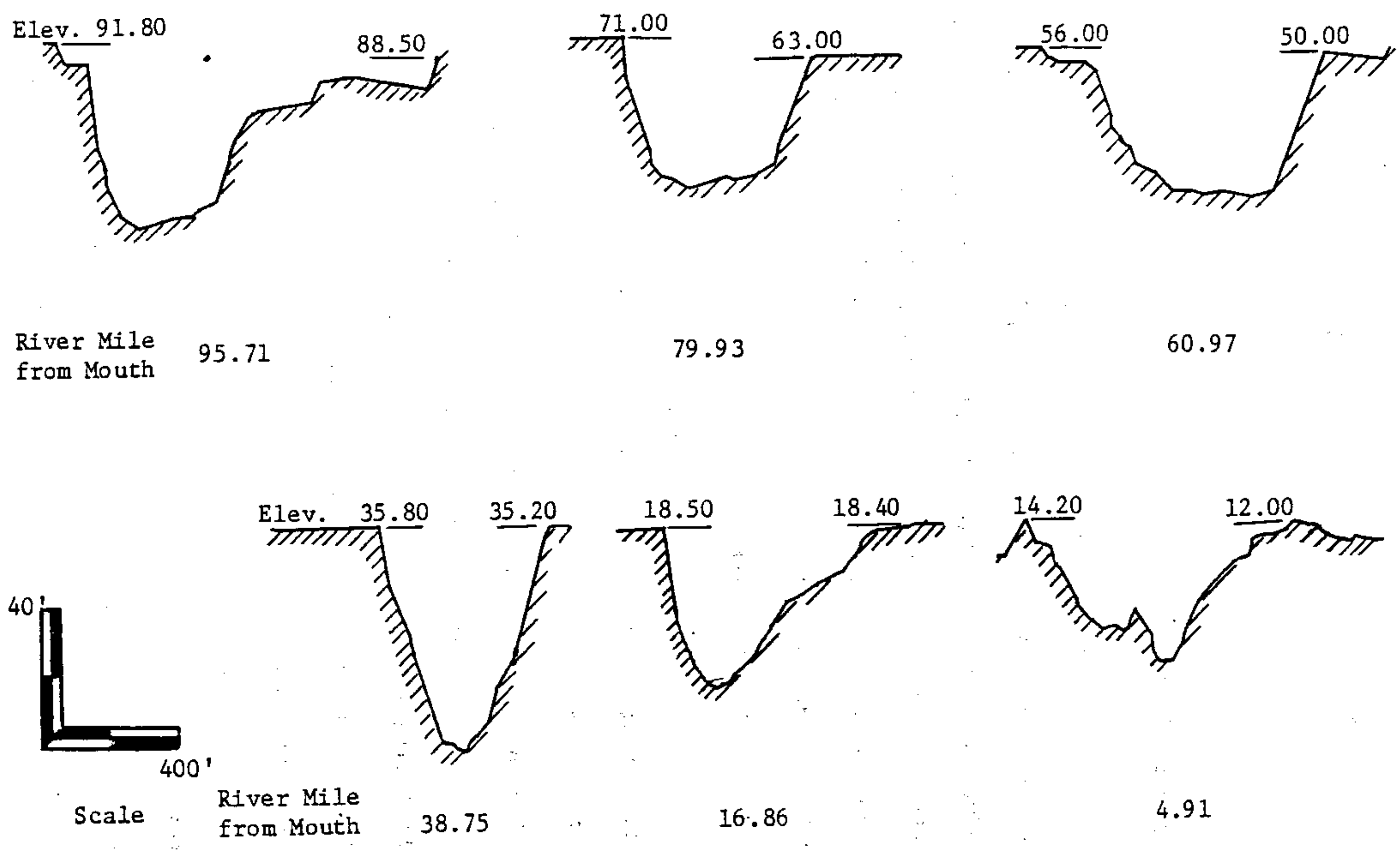

FIG. D-1.- Continued

APPENDIX E

DESALTING

General Notes

It has been increasingly recognized that desalting is a potential source of water supply for current and future needs. The cost of desalted water has been coming down over time. Figure E-1 presents an overall view of desalting costs based upon the study of a planning model by the Office of Saline Water (44). It is seen from this figure that in the near future desalted water will be a comparable competitor to conventional methods of water supply.

Research and development activities by research organizations, universities, industrial interests and the Office of Saline Water have brought desalting technology to a point where desalting plants in sizes on the order of 50 to 500 mgd are becoming a reality. Figure E-2 shows an overall view of desalting plant capacity projected into the future based upon an OSW planning model. Efforts are continuing to develop improved desalting processes and technology to meet low cost desalting goals.

The most developed process to date for sea water conversion is distillation. This may be accomplished by various methods, e.g., multistage flash (MSF), vertical tube evaporation (VTE), vapor compression (VC), and combination of these. The multistage flash process

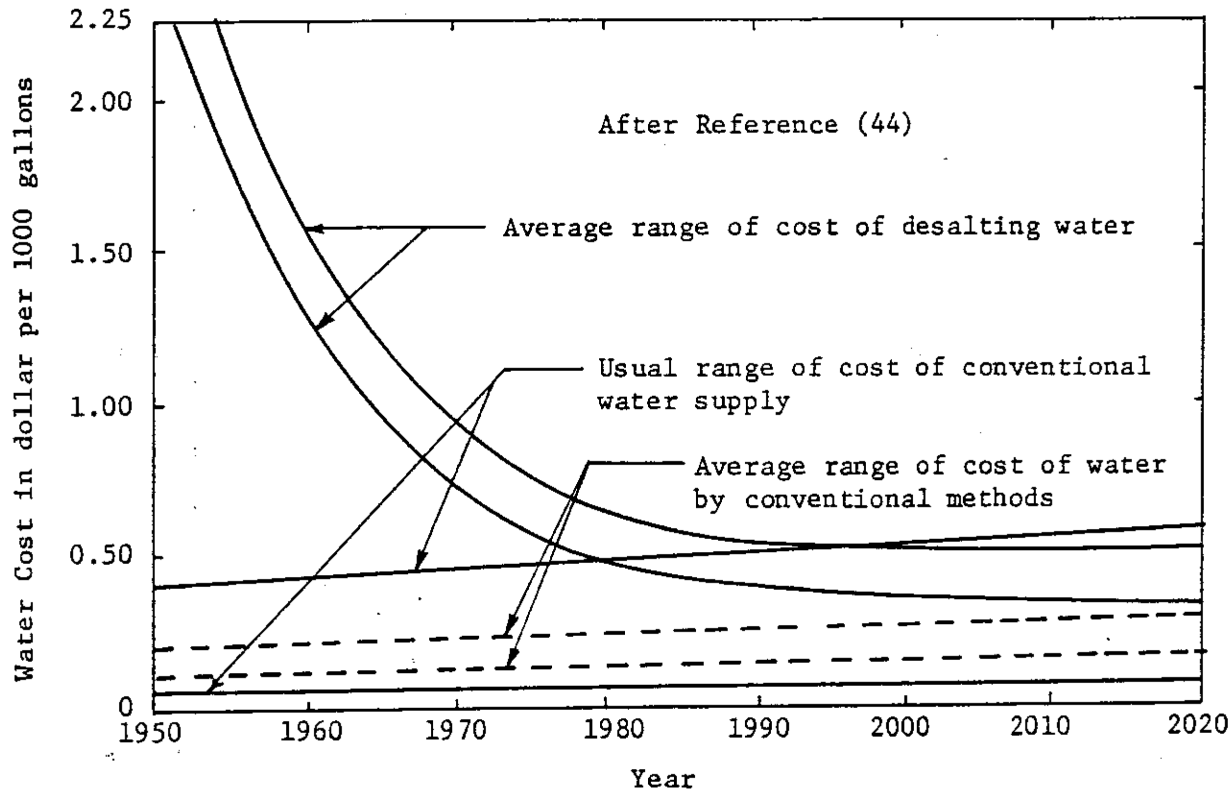

FIG. E-1.-COMPARATIVE APPRAISAL OF COSTS OF WATER BY CONVENTIONAL METHOD AND BY DESALTING (July 1971 price level)

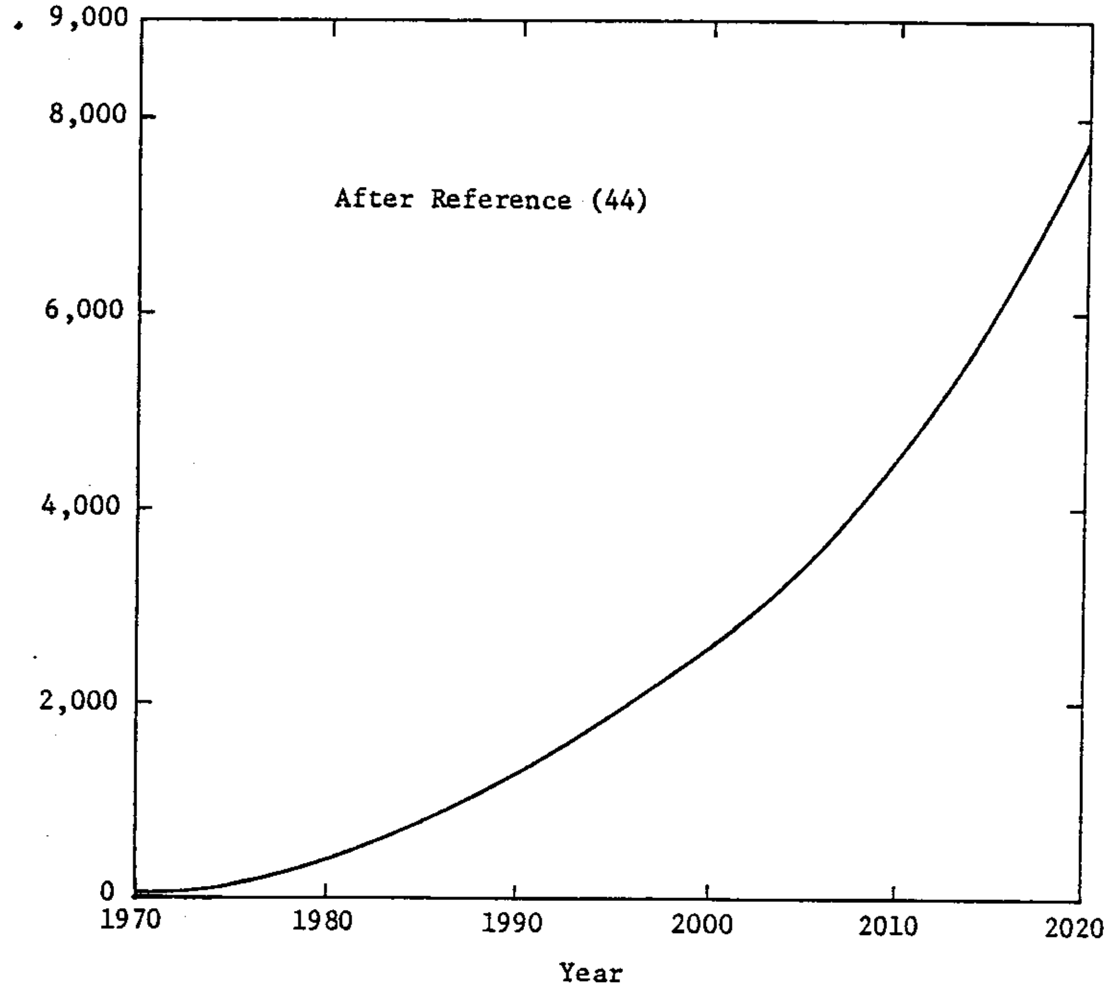

FIG. E-2.-PROJECTION OF DESALTING PLANT CAPACITY BASED UPON OFFICE OF SALINE WATER PLANNING MODEL is the current production leader in sea water desalting. This is true because production plants are available in desired sizes, and its design, operation, maintenance and other process characteristics are well developed.

Some studies indicated more economy in a VTE-MSF desalting system. A VTE-MSF plant at Freeport, Texas, demonstrated good stability and operational characteristics. Skelton (58) quoted Newton, the manager of the Freeport plant, "A plant with a capacity 50 to 100 mgd might be as low as 35 cents per 1000 gallons, but 65 to 70 cents possible would be more realistic." Newton further stated that the commercial feasibility of desalination had been proven.

With the advancement of desalting technology various processes have been developed to suit different conditions. Therefore, depending on circumstances, a particular process would be more advantageous than other. Selection of a process or processes is guided by many considerations. The important factors are: (1) quantity of the desalted water needed, (2) salt concentration and composition of the feed water, (3) temperature of the feed water, (4) product water quality, (5) availability of energy, (6) dependability of feed water source, (7) opportunity of brine disposal, (8) site location of plant, and (9) environmental factors.

Design and Construction Times

The design and construction times of a desalting plant depend on the size and purpose of the plant, that is, whether it is a multi-purpose or single purpose project. Expected construction times for various plants were determined by the Office of Saline Water. Table E-1 shows the times required for single purpose MSF and VTE-MSF plants.

TABLE E-1.-DESIGN AND CONSTRUCTION TIMES FOR SINGLE PURPOSE PLANT

MSF			VTE-MSF		
Plant Capacity, mgd	Design Time, month	Construction Time, month	Plant Capacity, mgd	Design Time, month	Construction Time, month
0.1-0.5	6	8	0.2	6	8
2.5	8	20	12	6	24
25	8	36	50	9	40
50	12	36	250	16	48
100	12	42			

Cost Estimating

Prehn and McGaugh (50) described desalting cost calculating procedures. The Office of Saline Water (44) condensed the procedures to make them more useable. The general formats presented in the Desalting Handbook for Planners were followed in this investigation. It was mentioned in the book that the cost estimate might not be adequate enough to use for repayment negotiations or similar purposes.

Therefore, cost estimates should only be used for comparison in preliminary economic analysis for feasibility studies.

In the general formats the project cost was divided into two major cost factors: (a) capital costs, and (b) annual costs. Capital costs consisted of cost for construction, interest during construction, startup cost, land cost, working capital, investigations, design and specifications, construction supervision, and other associated general expenses. Annual costs included operation and maintenance cost of plant, interest, insurance, replacement reserves, energy and fuel, chemicals, and other associated costs. Computational procedures for these items are given in the next section.

Interest during construction was determined on the basis that the owner will borrow money as needed. Glen and Barbour (18) presented details for computing interest during construction. The owner of a project makes some indirect construction costs. These costs include preliminary expenses for project investigation, land acquisition, contract administration, and other general overhead and administrative expenses. In the absence of specific information regarding these items, Fig. E-3 has been recommended for use as an approximate estimate. The values of the plotted points were taken from Desalting Handbook for Planners.

The life of a project has a very important effect on the unit price of the product of the project. The Office of Saline Water considered 30 years to be the useful life of a desalting plant.

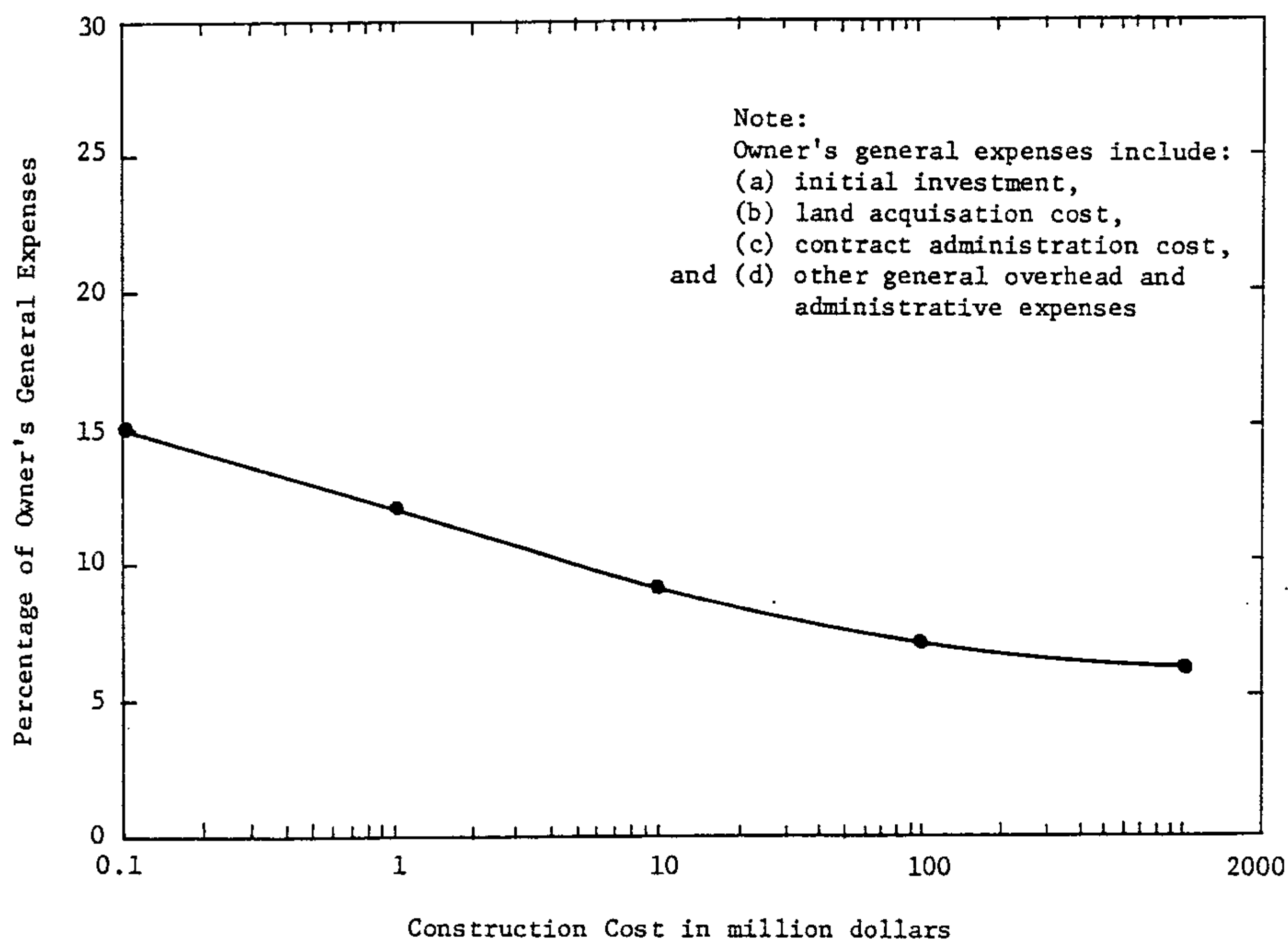

FIG. E-3.-CONSTRUCTION COST AND OWNER'S GENERAL EXPENSES
(Plotted points are taken from reference 44)

Estimating Procedure

Itemized outline of the procedure followed is given below:

a. Construction cost: It is obtained from Fig. E-4.

b. Land costs: The area of land required for MSF and VTE-MSF plants depending on the plant capacity is determined from Figs. E-5 and E-5a. Prevailing unit cost of land in the plant site should be assessed from local source. The area of land times the unit price gives the cost of land.

c. General site development costs: These costs are estimated from Fig. E-6.

d. Water storage and conveyance costs: Conveyance pipeline cost is computed from Fig. E-7. Storage required in this problem under investigation was assumed to be zero. However, for inclusion of storage cost in analysis Fig. 7-22 of reference 44 might be used.

e. Feed water and product water treatment costs: These costs are included in construction costs (Fig. E-4).

f. Feed water supply and brine disposal costs: An installation might be with or without cooling towers. For a plant without a cooling tower the capital costs for feed water supply and brine disposal facilities are estimated from Fig. E-8. The intake flow stream volume has been obtained from the following equations:

1. MSF plant:

(i) based on maximum allowable concentration of calcium in the brine:

$$BPR = \frac{1 - (50 \div TDS_i)}{(900 \div Ca_i) - 1} \qquad \text{- - - - - - - - - - - - (E-1)}$$

(ii) based on maximum allowable total dissolved solids in the brine:

$$BPR = \frac{TDS_i - 50}{60{,}000 - TDS_i} \qquad \text{- - - - - - - - - - - - (E-2)}$$

where, BPR = brine-to-product ratio

50 = ppm assumed total dissolved solids in product

TDS_i = ppm total dissolved solids in feed water

900 = ppm assumed allowable calcium concentration in the brine; use other Ca concentration where conditions so indicate

Ca_i = ppm calcium concentration in feed water

60,000 = ppm allowable total dissolved solids in brine.

Select the larger of the two BPR obtained from Equations (E-1) and (E-2), and calculate the cooling water rate:

$$Cw = Cp\ (2.5 - BPR) \qquad \text{- - - - - - - - - - - (E-3)}$$

where, Cw = cooling water rate, mgd

Cp = product water rate, mgd

Now compute the brine discharge rate using the larger BPR obtained in Equations (E-1) and (E-2) from the following:

$$C_b = C_p \times BPR \quad \text{- - - - - - - - - - - - -} \quad (E-4)$$

where, C_b = brine discharge rate, mgd

The intake volume of feed water is obtained by:

$$C_i = C_p + C_b + C_w \quad \text{- - - - - - - - - - -} \quad (E-5)$$

where, C_i = intake volume rate of feed water, mgd

2. VTE-MSF plant:

$$BPR = \frac{TDS_i - 50}{80,000 - TDS_i} \quad \text{- - - - - - - - - - -} \quad (E-6)$$

Select the larger of two BPR obtained from Equations (E-1) and (E-6), compute:

C_b from Equation (E-4),

$$C_w = C_p\ (2.0 - BPR) \quad \text{- - - - - - - - - -} \quad (E-7)$$

and C_i from Equation (E-5)

g. Steam generator costs: A steam generator for feed water heating is necessary for a single purpose desalting plant where there is no alternative source of steam supply.

(i) Obtain steam requirements from Fig. E-9.

(ii) Estimate fuel requirement from Fig. E-16.

(iii) The cost of steam generator is estimated from Fig. E-10.

h. Operation and maintenance labor, supplies and maintenance material costs:

(i) Use Fig. E-11 for desalting plant annual cost.

(ii) Use Fig. E-12 for intake and brine outfall system annual cost.

(iii) Use Fig. E-13 for steam generators annual cost.

i. Chemical costs: The cost of chemicals for distillation desalting processes is about $0.02 per 1000 gallons of product water.

j. Electric power costs: The electric power requirements of MSF and VTE-MSF desalting processes is obtained from Fig. E-14. The cost of electric power for feed water and cooling water supply, brine and cooling water disposal, and product water conveyance are computed on the basis of 0.004 Kwhr required per thousand gallons pumped per foot of dynamic head.

It was recommended that the total dynamic head be computed as follows:

(a) head pressure at destination

plus (b) destination elevation

plus (c) friction losses in pipeline

minus (d) origin elevation

minus (e) available head pressure at origin.

The following typical pressure heads may be used for the purpose of order-of-magnitude estimates (44):

(a) required desalting plant pressure head = 100 feet

(b) product water discharge pressure head = 250 feet

(c) cooling water discharge pressure head = 10 feet

(d) pipeline head loss is determined from Fig. E-15.

Unit Water Cost

The Office of Saline Water suggested two equations for computing the average cost per unit of water for baseloaded and variable production rate plants. In the first case the plant was expected to operate at its design capacity and in the later the production rate of desalted water was expected to vary from year to year due to growing demand. The equations are:

(a) Baseload:

$$c' = \frac{A'}{10\ Q'N'} \qquad \text{(E-8)}$$

where, c' = average unit cost of water, cents/1000 gals

A' = total annual costs, dollars/year

Q' = desalting plant design capacity, mgd

N' = number of stream days per year = 365 f

f = plant capacity factor = $\frac{N'}{365}$

(b) Variable plant factor:

$$c = \frac{K' + \sum_{i=1}^{N} (B.r + A_i)\left[\frac{1}{(1+r)}i\right]}{10 \sum_{i=1}^{N} P_i \left[\frac{1}{(1+r)}i\right]} \qquad \text{(E-9)}$$

where, c = levelized unit cost, cents/1000 gals

K' = depreciable capital costs, dollars

B = nondepreciable capital costs (land costs and working capital), dollars

A_i = operating and maintenance cost plus annual replacement costs during the i-th year, dollars/year

r = interest rate expressed in decimal form

$\frac{1}{(1+r)^i}$ = single payment worth factor for the i-th year

P_i = anticipated water production during the i-th year, millions of gallons per year

N = plant life, years

Cost Computations

Project Description: Supply of desalted water to the areas that would be served by the Millican Reservoir project.

Cost Includes: Costs of desalting plant, sea water intake and outfall, and steam generator.

Price Level: July, 1971

Plant Type: VTE-MSF, single purpose

Plant Life: 30 years, Plant Factor: 90%

Water Requirement: 193.9 mgd,

Annual Production: 70.6725×10^9 gallons.

General Computation:

Fixed charge rate = interest rate + insurance and tax rate + amortization rate

= 3.5% + 0.5% + 1.06% = 5.06% for 30 yrs.

= 3.5% + 0.5% + 0.01% = 4.01% for 100 yrs.

Production Capacity = 193.9 ÷ 0.90 (plant factor) = 215 mgd

$$\text{BPR} = \frac{1 - (50 \div 35{,}000)}{(900 \div 400) - 1} = 0.80, \text{ from Eqn. (E-1)}$$

$$\text{BPR} = \frac{35{,}000 - 50}{80{,}000 - 35{,}000} = 0.777, \text{ from Eqn. (E-6)}$$

C_b = 0.8 x 215 = 172 mgd, Eqn. (E-4)

$C_w = C_p$ (2 - BPR) = 215 x 1.2 = 258 mgd, Eqn. (E-7)

$C_i = C_p + C_b + C_w$ = 215 + 172 + 258

= 645 mgd, Eqn. (E-5)

<u>Capital Cost Computation</u>:

1.	Construction cost of desalting plant:	
	from Fig. E-4 for 215 mgd	\$ 135,000,000
2.	General site development:	
	from Fig. E-6 for 215 mgd	3,200,000
3.	Construction Cost of steam generator:	
	from Fig. E-9 for 215 mgd	
	steam requirement = 130,000 MBtu and	
	from Fig. E-10	17,000,000
	Subtotal	\$ 155,200,000

4. Interest during construction:

construction period is 48 month from Table E-1.

Interest rate is calculated as in pp. 7-3 of reference 44.

	Interest rate is 3.5 (3 ÷ 2) = 5.25%	$ 8,148,000
5.	Start-up cost:	
	1/12th of total operation and maintenance cost minus $420,000 [see 1(iii), p. 152]	1,862,400
6.	Owner's general expense from Fig. E-3, 7% of subtotal	10,864,000
	Total of depreciating cost	$ 176,074,400

7. Land Cost:

(a) Land requirement:

(i)	desalting plant, Fig. E-5	= 37 acres
(ii)	steam generator: from Fig. E-5a for 130,000 MBtu as in Item 3	= 24 acres
(iii)	sea water intake: pp. 7-25 of reference 44	= 10 acres
	Total Land required	= 71 acres
	Use 75 acres	

(b)	Land cost is $1,000 per acre (assumed) total land cost = $1,000 x 75 =	$ 75,000
8.	Working capital: 1/6th of total operation and maintenance	4,564,800
	Total of nondepreciating capital cost =	$ 4,639,800

Total capital cost

= depreciating capital + nondepreciating capital

= \$176,074,400 + \$4,639,800 = \$180,714,200

<u>Annual Cost Computation:</u>

1. Operation and maintenance labor, supplies, and maintenance materials:

(i)	desalting plant, Fig. E-11 for 215 mgd =	\$ 1,320,000
(ii)	sea water intake and outfall, Fig. E-12 for 645 mgd =	420,000
(iii)	steam generator, Fig. E-13 for 130,000 MBtu =	880,000
	Total =	\$ 2,620,000

2. Chemicals: \$0.02 per 1000 gallons of product water = \$0.02 x 70,627,500 = \$ 1,412,550

3. Annual fuel cost:

   from Fig. E-16, steam generator fuel requirements for 130,000 MBtu steam rating of boiler = 150,000 MBtu

   unit cost of fuel = \$0.40 per MBtu as assumed in problem

   Total fuel cost = \$0.40 x 150,000 x 365 x 0.9 = \$19,710,000

4. Electric power:

   (i) from Fig. E-14 for 215 mgd,

   desalting plant requirement = 850,000 Kwhr/day

   (ii) intake pumping unit for 645 mgd,

   discussed in article j of Cost

   Estimating Procedure, power requirement =

   0.004 x 645 x 1000 x 100 = 258,000 Kwhr/day

   Total power requirement = 1,108,000 Kwhr/day.

   Unit cost of power assumed in problem =

   $ 0.01 per Kwhr

   Total annual cost of power =

   1,108,000 x 365 x 0.9 x $ 0.01 = $ 3,643,650

   Total operation and maintenance = $ 27,386,200

5. Annual cost of depreciating capital =

   176,074,400 x 0.0506 + 1,239,000*

   * taken from cost computation for sea water intake and brine outfall (next page) $ 10,139,000

6. Annual cost of nondepreciating capital =

   4,639,800 x 0.035 $ 162,000

   Total annual capital charges = $ 10,301,000

Total annual costs =

$ 27,386,200 + $ 10,301,000 = $ 37,687,200

Unit cost of water =

(37,687,200 ÷ 70,627,500) 100 = 53.4 cents/1000 gals

<u>Cost Computation of Sea Water Intake and Brine Outfall</u>:

<u>Capital Cost</u>:

1. Construction cost:

   (i) from Fig. E-8 for 645 mgd = $ 26,000,000

   (ii) cost of sea water intake pipe from Fig. E-7 = $ 170 per foot

   cost of 1 mile of pipeline = $ 897,600

   (iii) cost of brine outfall pipe from Fig. 7 = $ 110 per foot

   cost of 1 mile of pipe line = $ 580,800

   Subtotal = $ 27,478,400

2. Interest during construction:

   For calculation of interest rate see pp. 7-3 of reference 44,

   interest rate = 3.5 x 3/2 = 5.25%, $ 1,442,000

3. Startup costs:

   1/12th of total operation and maintenance cost = 1/12 x $420,000 = $ 35,000

4.	Owner's general expense:	
	from Fig. E-3, 7%	$ 1,920,000
	Total depreciating capital =	$ 30,875,400

Annual Costs:

1.	Operation and maintenance labor, supplies, and maintenance material, from Fig. E-12 for 645 mgd =	$ 420,000
2.	Annual cost of depreciating capital = $ 30,875,400 x 0.0401 =	$ 1,239,000

Costs computed in this section were summarized and shown in Table 14 in Chapter V.

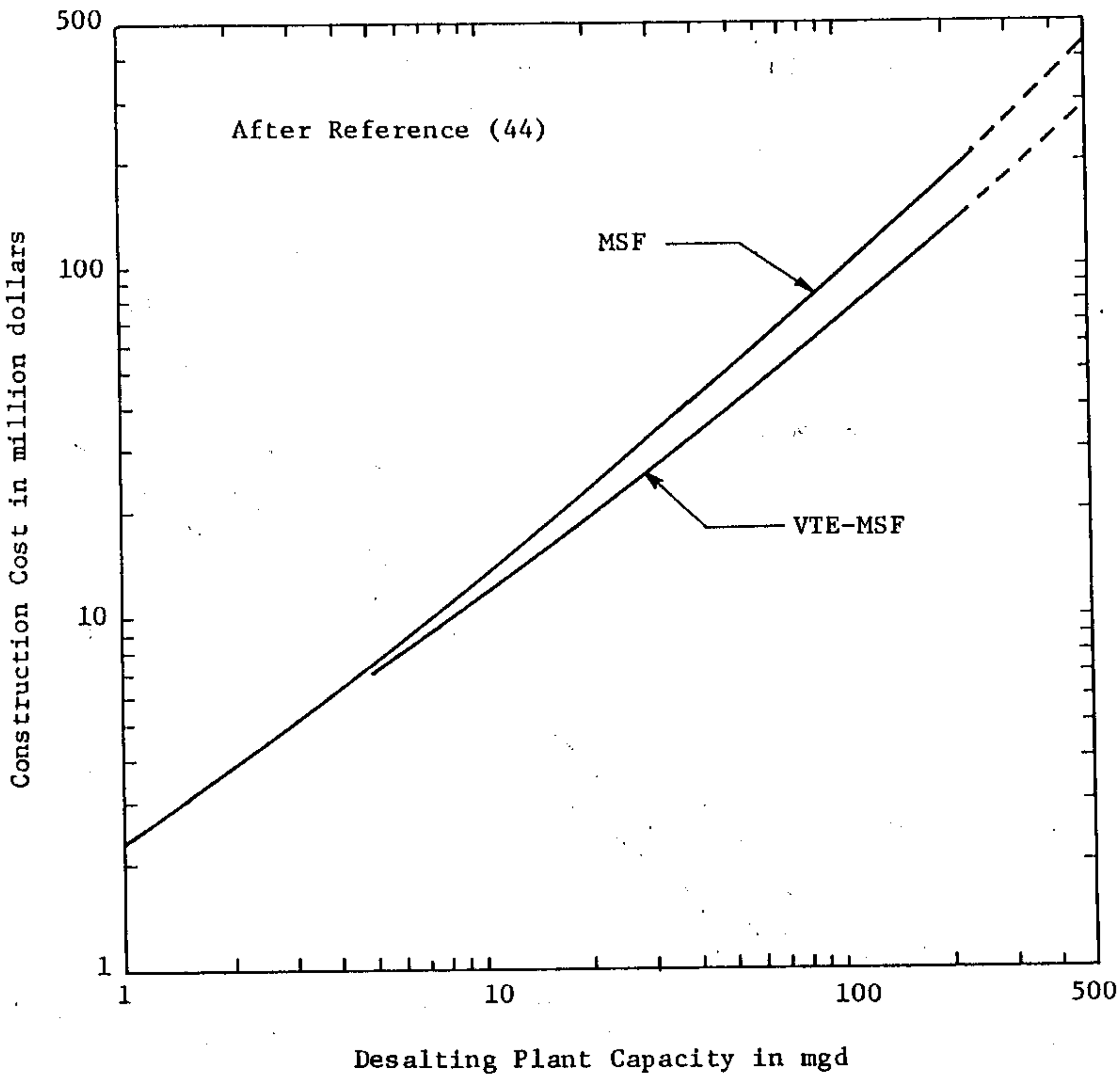

FIG. E-4.-CONSTRUCTION COST - SINGLE PURPOSE MSF AND VTE-MSF DESALTING PLANTS (July 1971 price level)

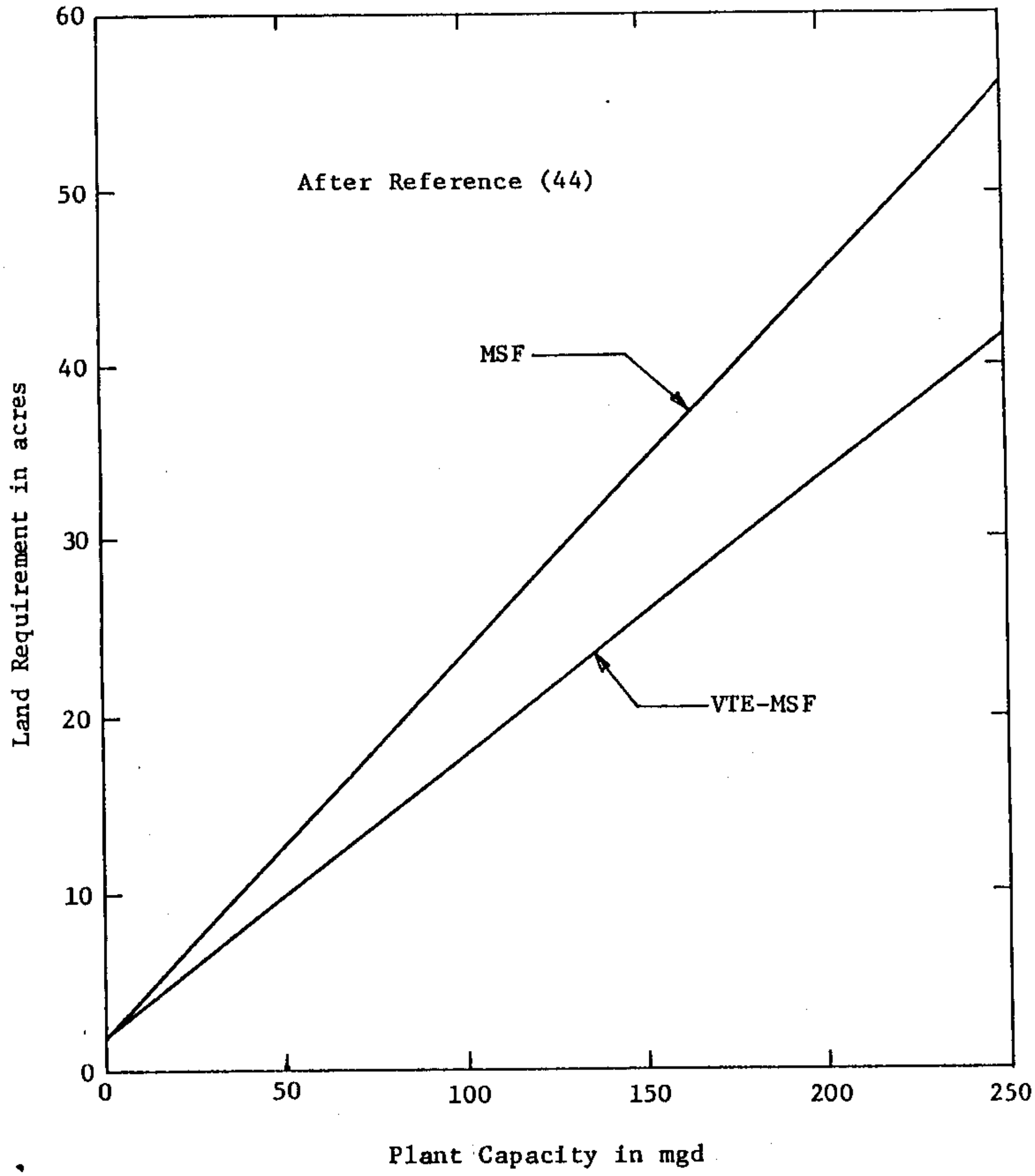

FIG. E-5.-LAND REQUIREMENT FOR SINGLE PURPOSE MSF AND VTE-MSF DESALTING PLANTS

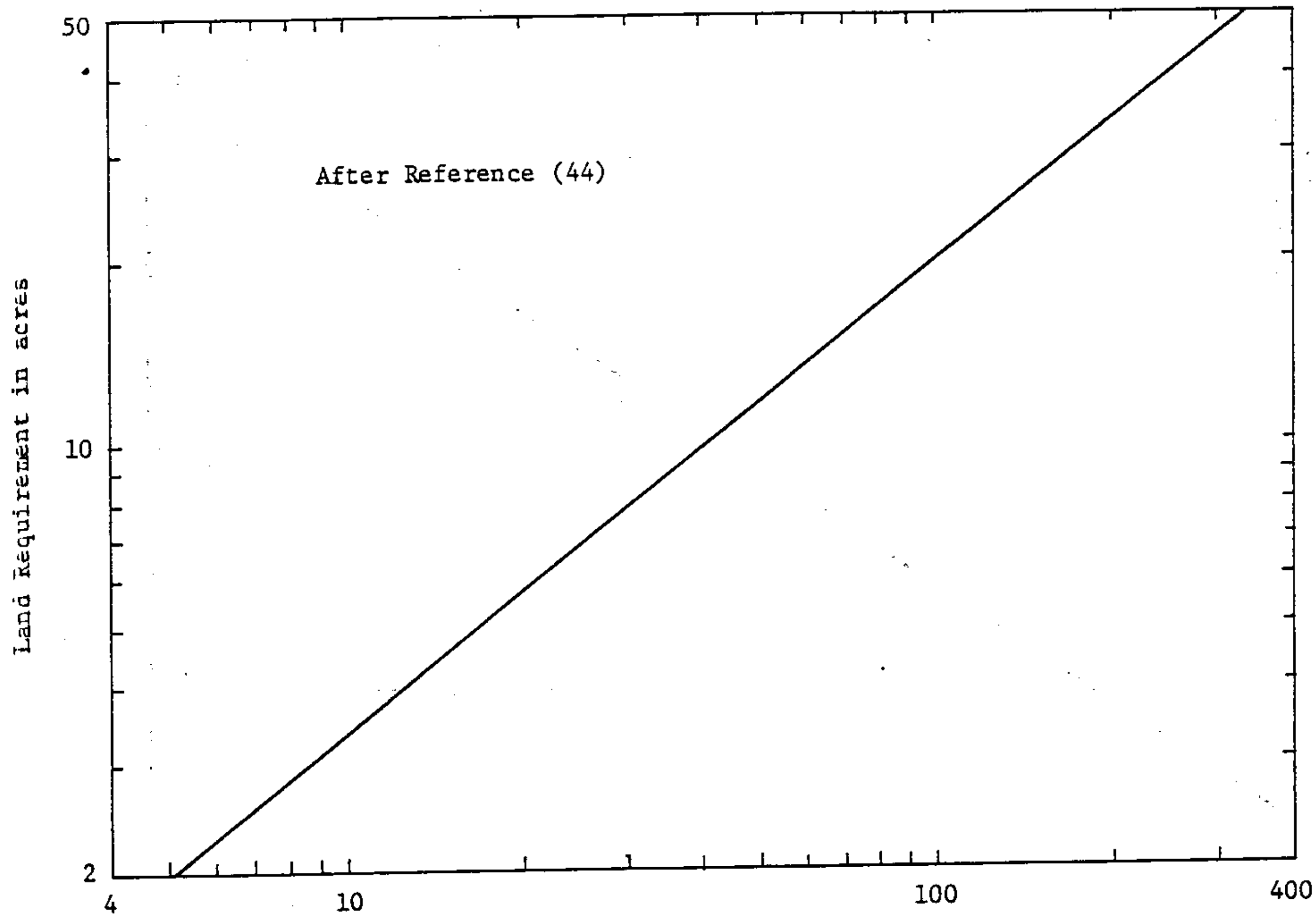

FIG. E-5a.-LAND REQUIREMENT FOR STEAM GENERATOR FOR FEED WATER HEATING

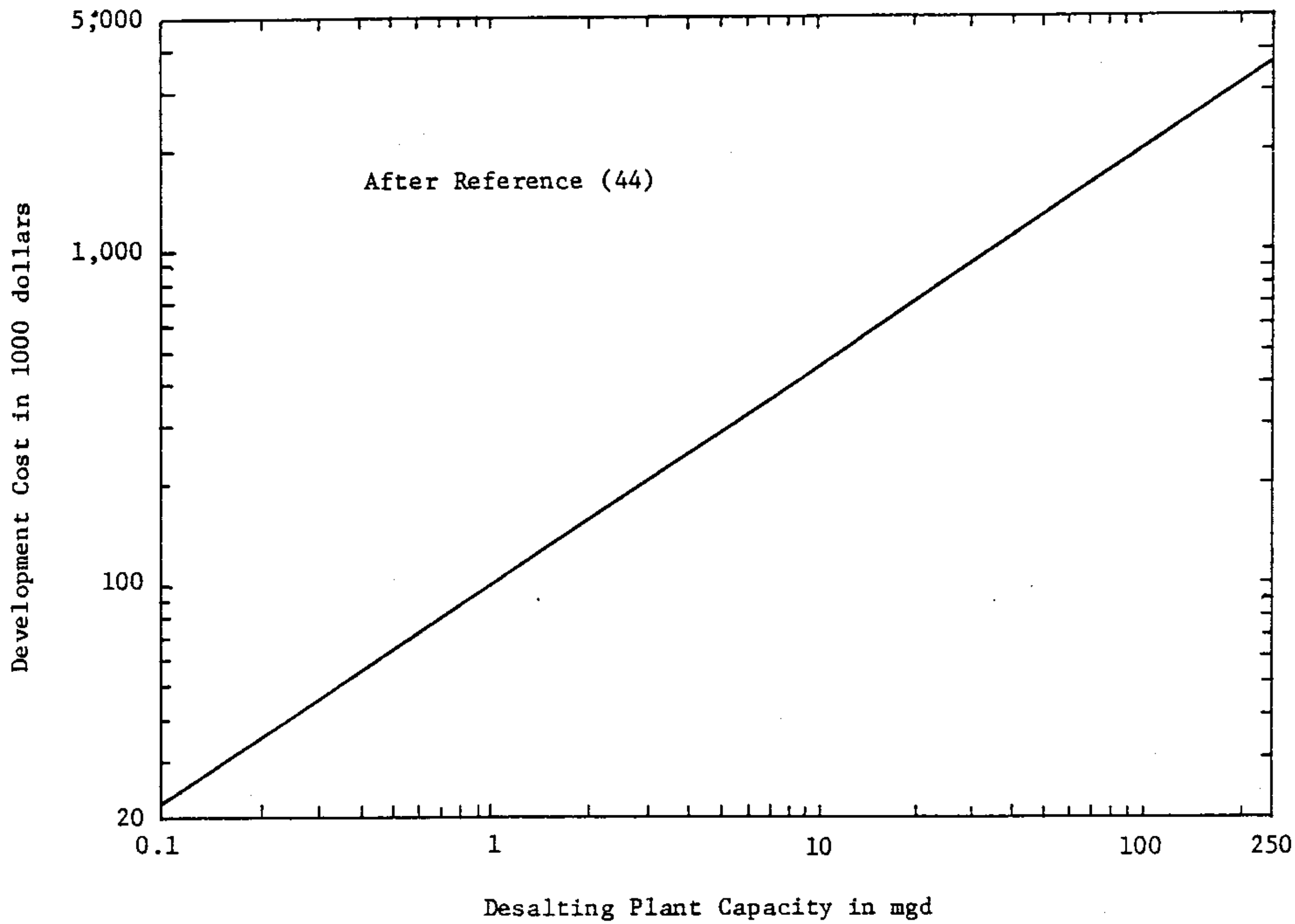

FIG. E-6.-GENERAL SITE DEVELOPMENT COST OF DESALTING PLANT (July 1971 price level)

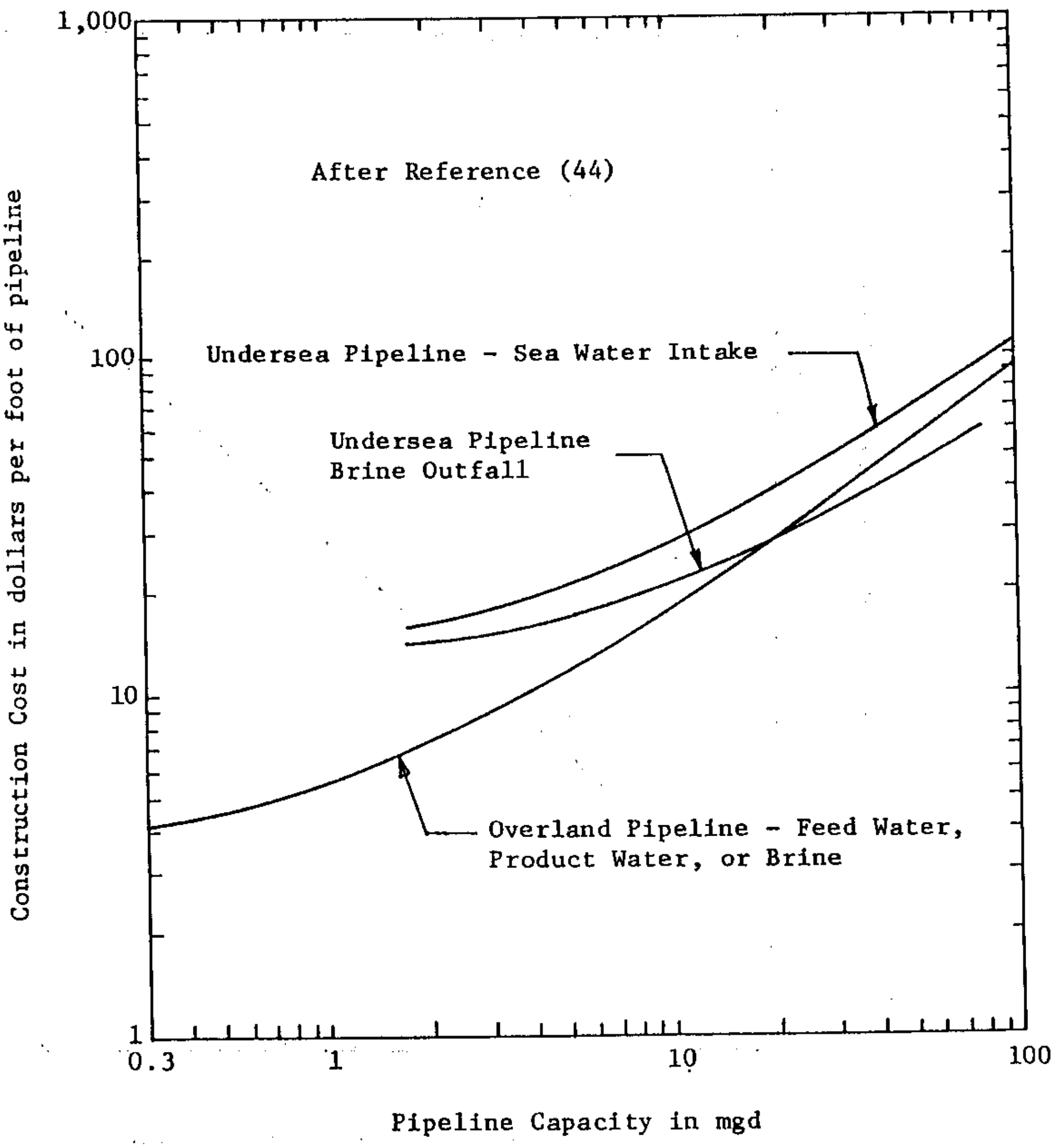

FIG. E-7.-CONSTRUCTION COSTS - PIPELINES (July 1971 price level)

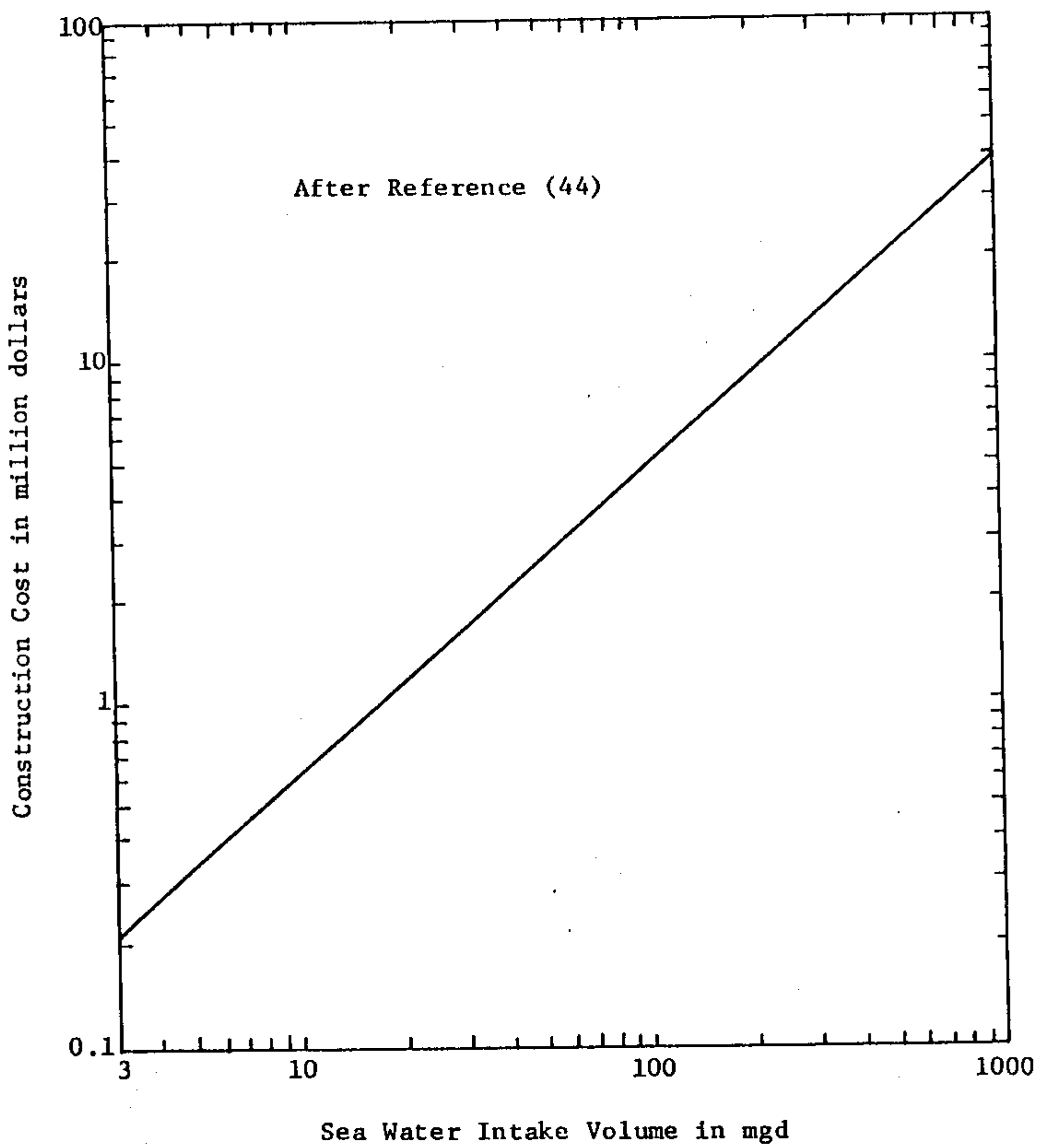

FIG. E-8.-CONSTRUCTION COST - SEA WATER INTAKE AND BRINE OUTFALL

(July 1971 price level)

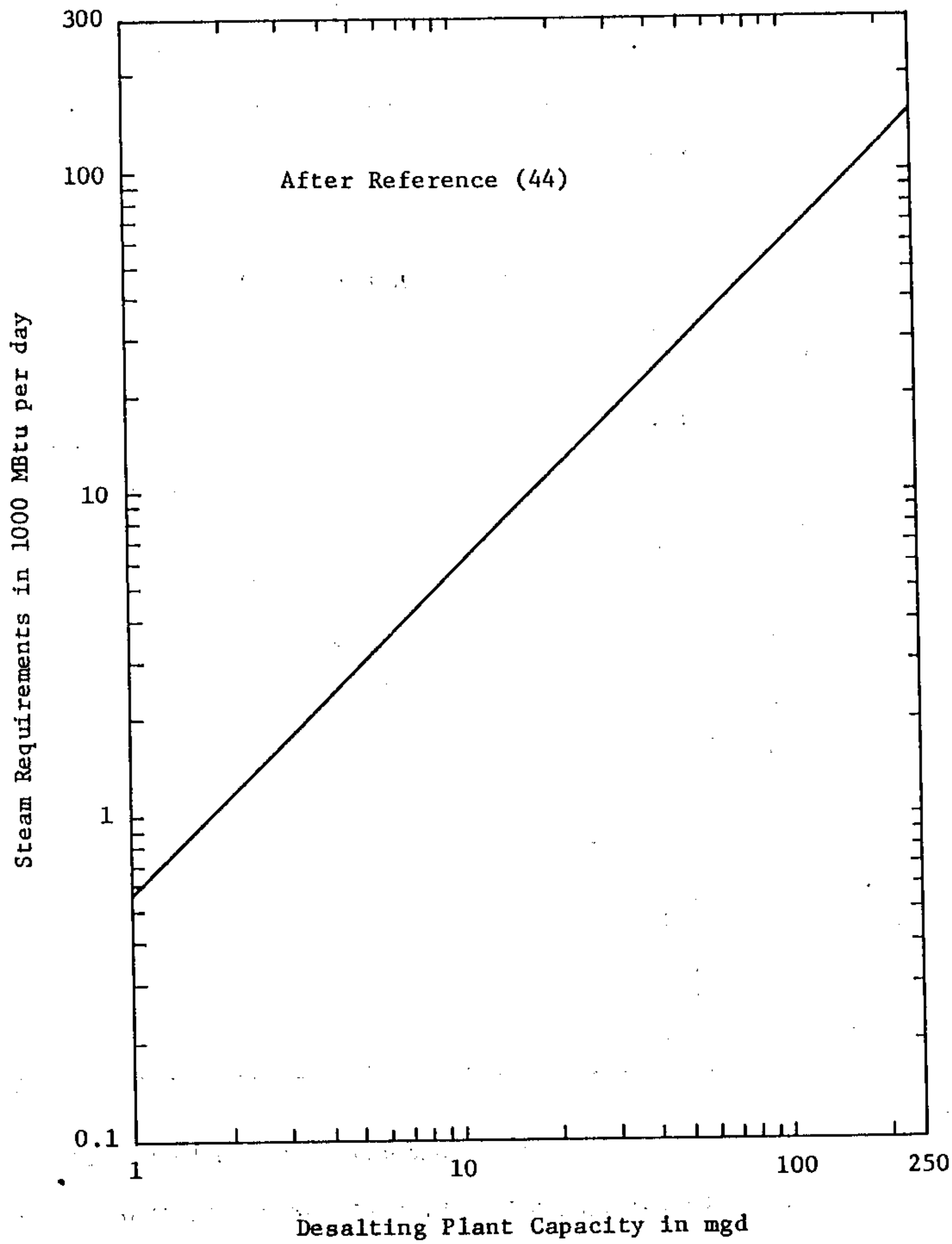

FIG. E-9.-STEAM REQUIREMENTS FOR SINGLE PURPOSE MSF AND VTE-MSF DESALTING PLANTS

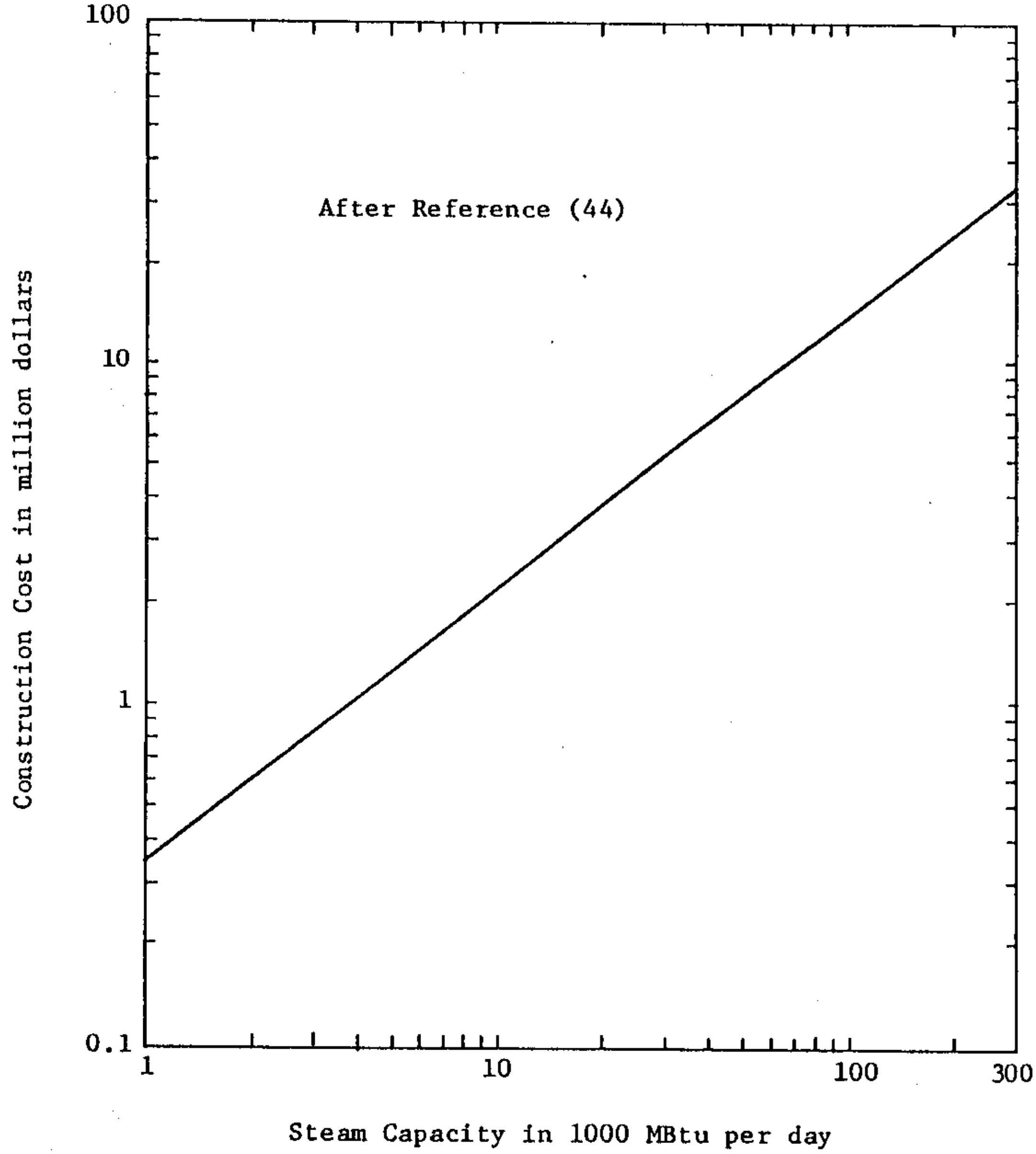

FIG. E-10.-CONSTRUCTION COST OF STEAM GENERATOR FOR FEED WATER HEATING (July 1971 price level)

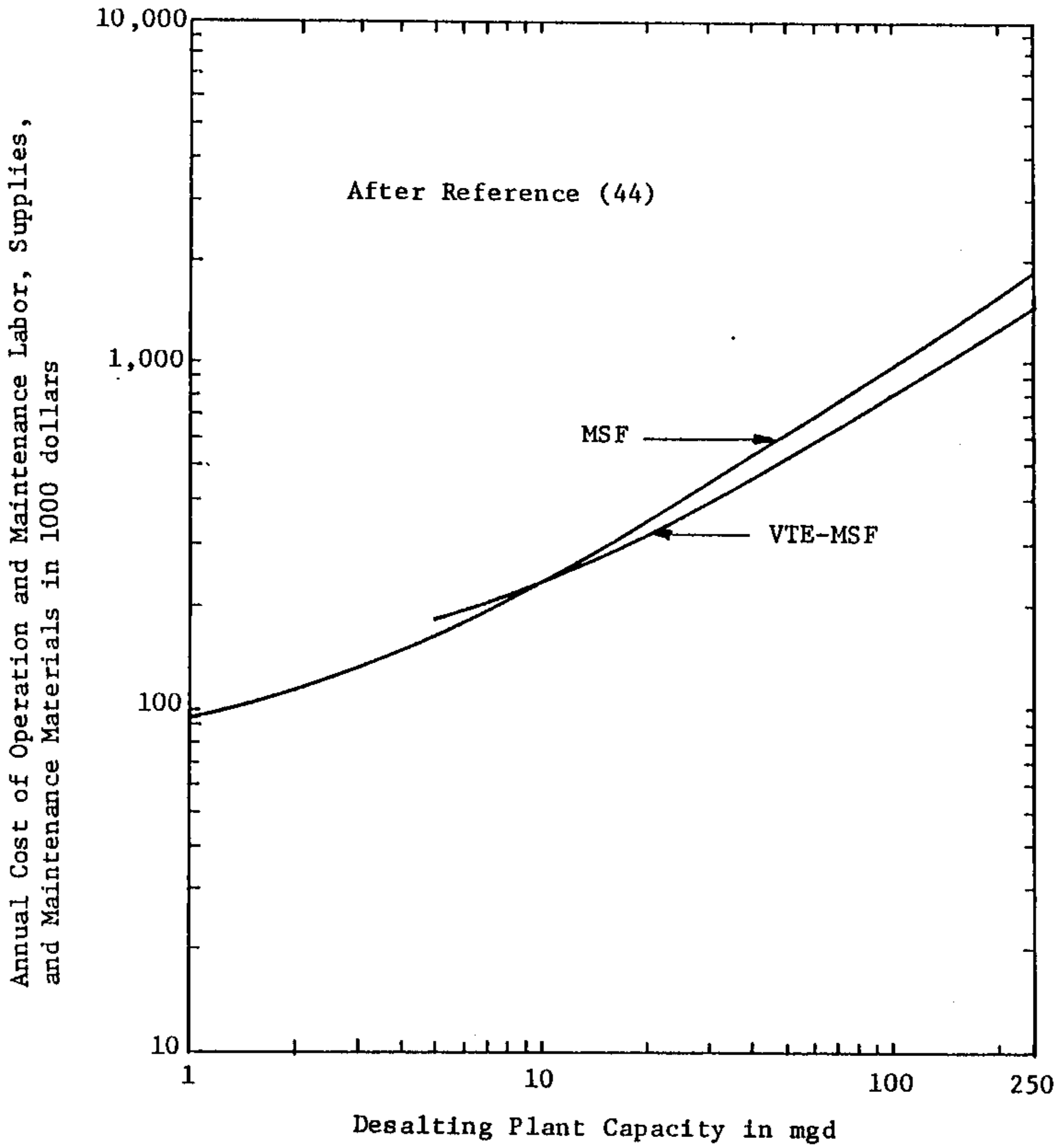

FIG. E-11.-ANNUAL COST OF O&M LABOR, SUPPLIES, AND MAINTENANCE MATERIALS FOR MSF AND VTE-MSF DESALTING PLANTS (July 1971 price level)

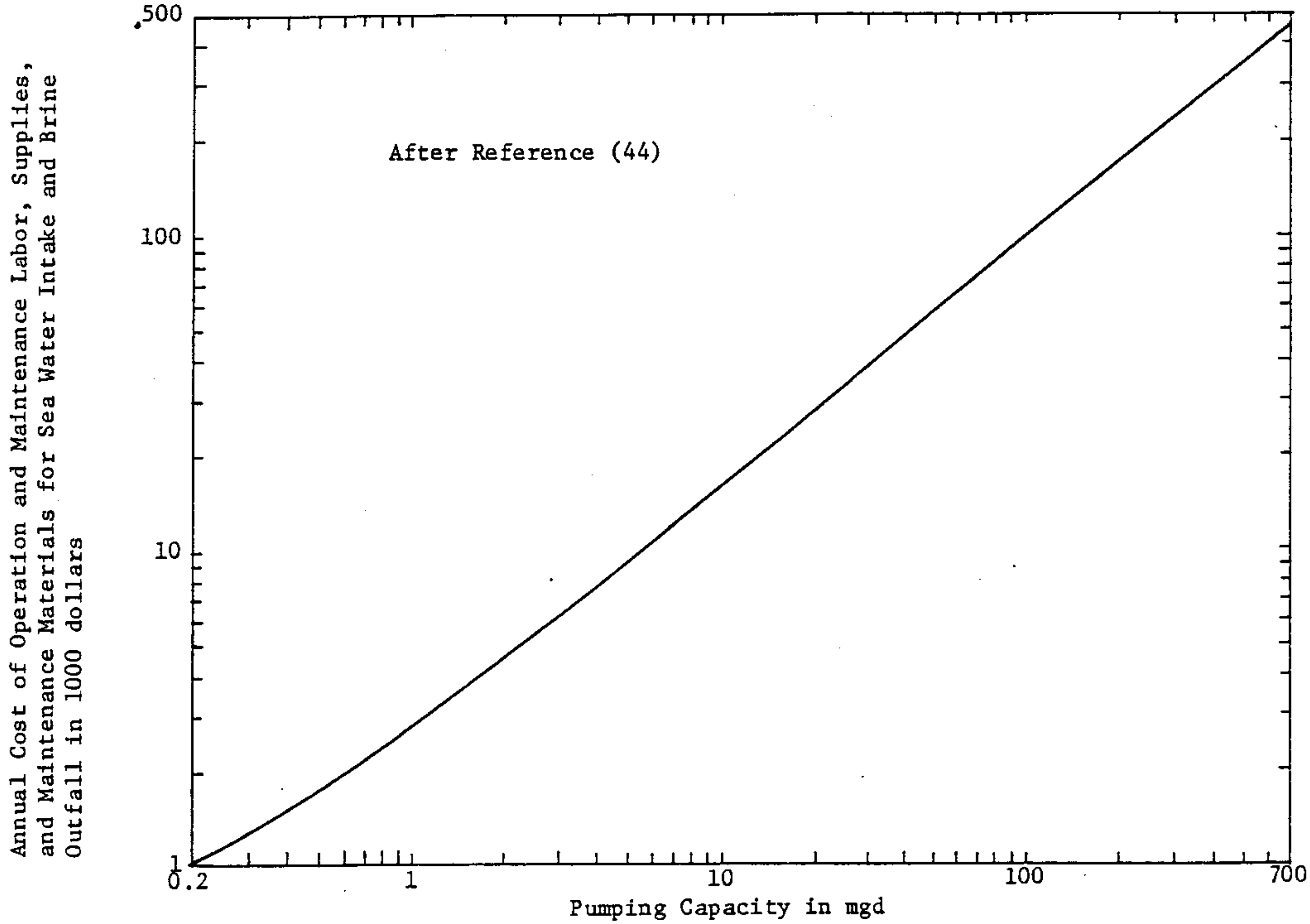

FIG. E-12.-ANNUAL COST OF SEA WATER INTAKE AND BRINE OUTFALL (July 1971 price level)

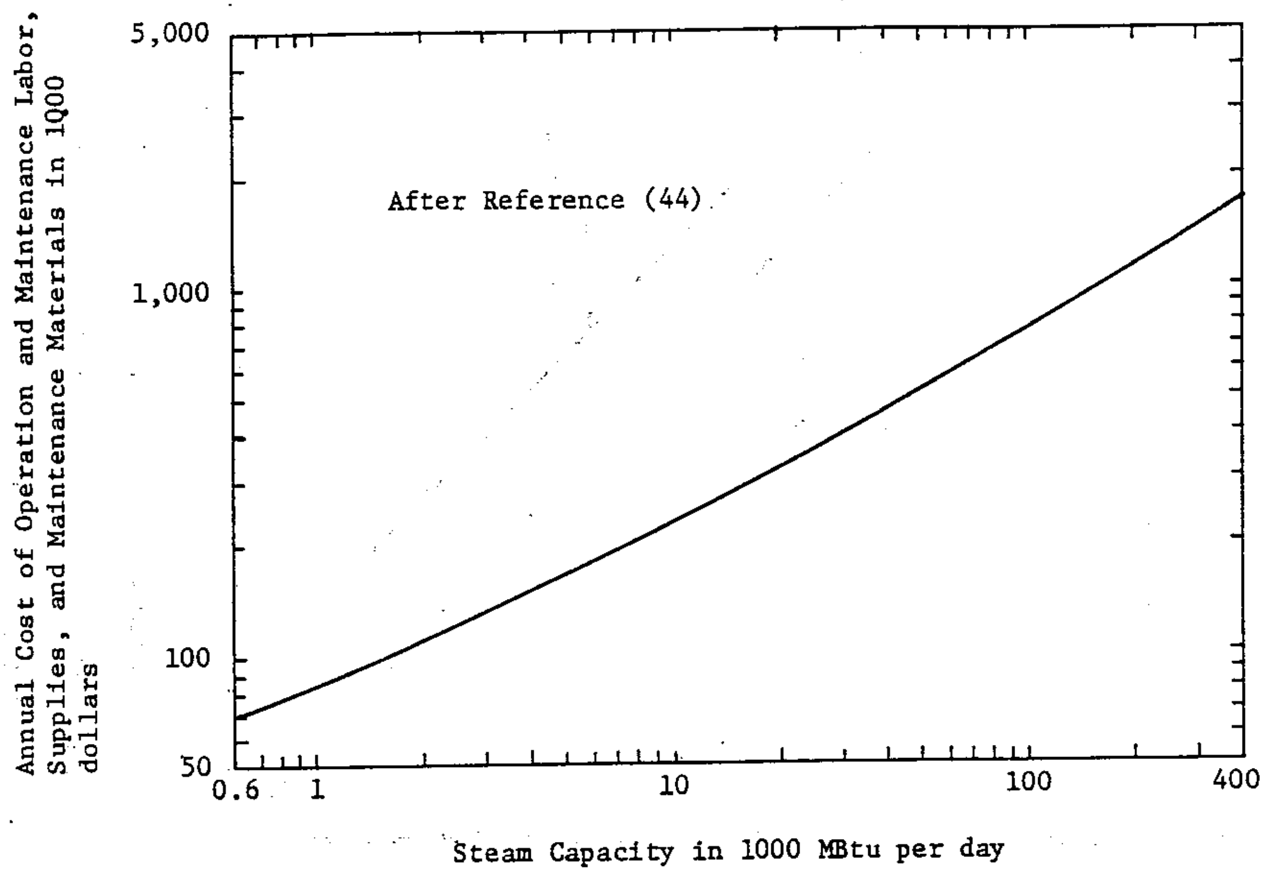

FIG. E-13.-ANNUAL COST OF STEAM GENERATOR FOR FEED WATER

(July 1971 price level)

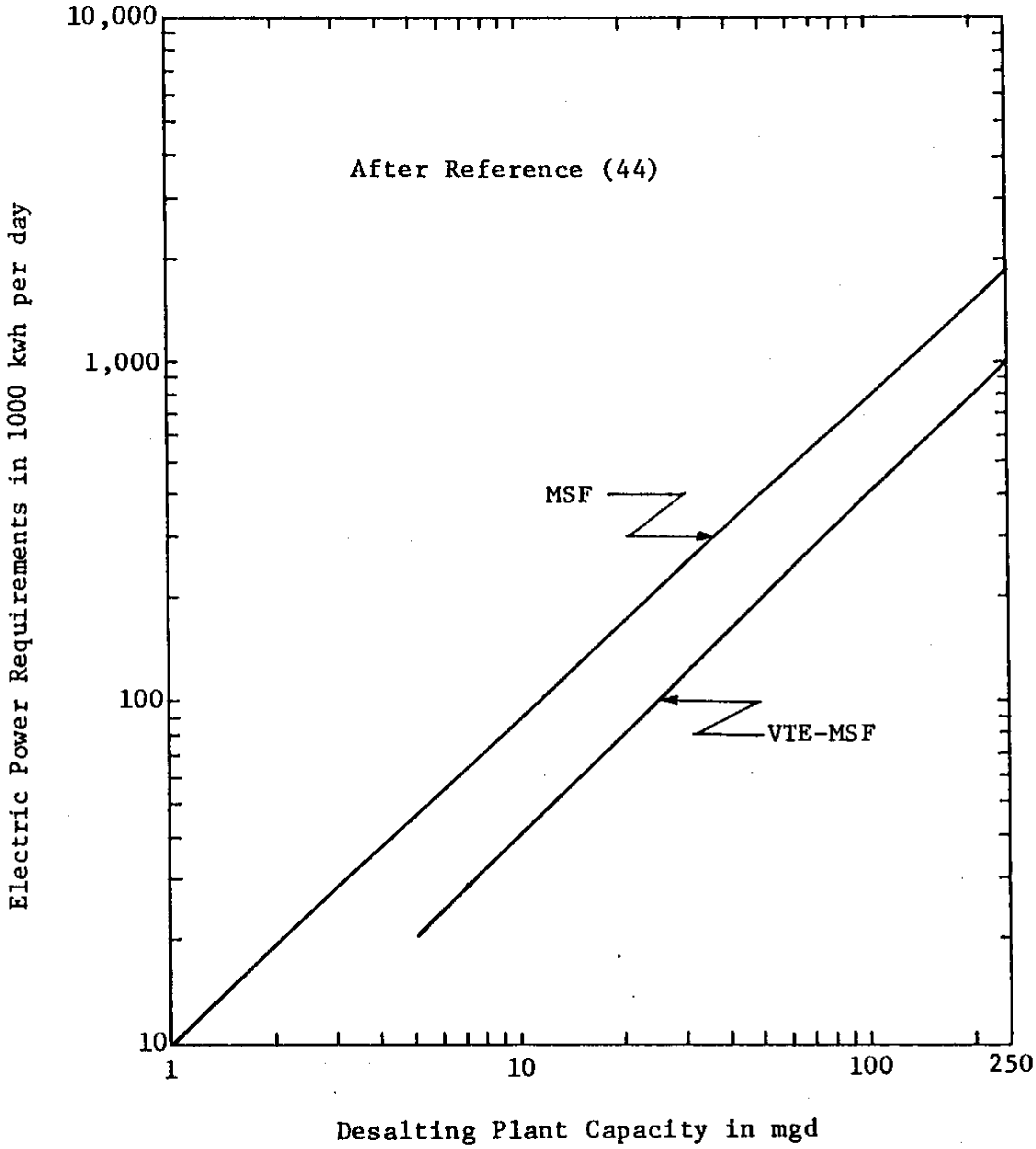

FIG. E-14.-ELECTRIC POWER REQUIREMENTS FOR SINGLE PURPOSE MSF AND VTE-MSF DESALTING PLANTS

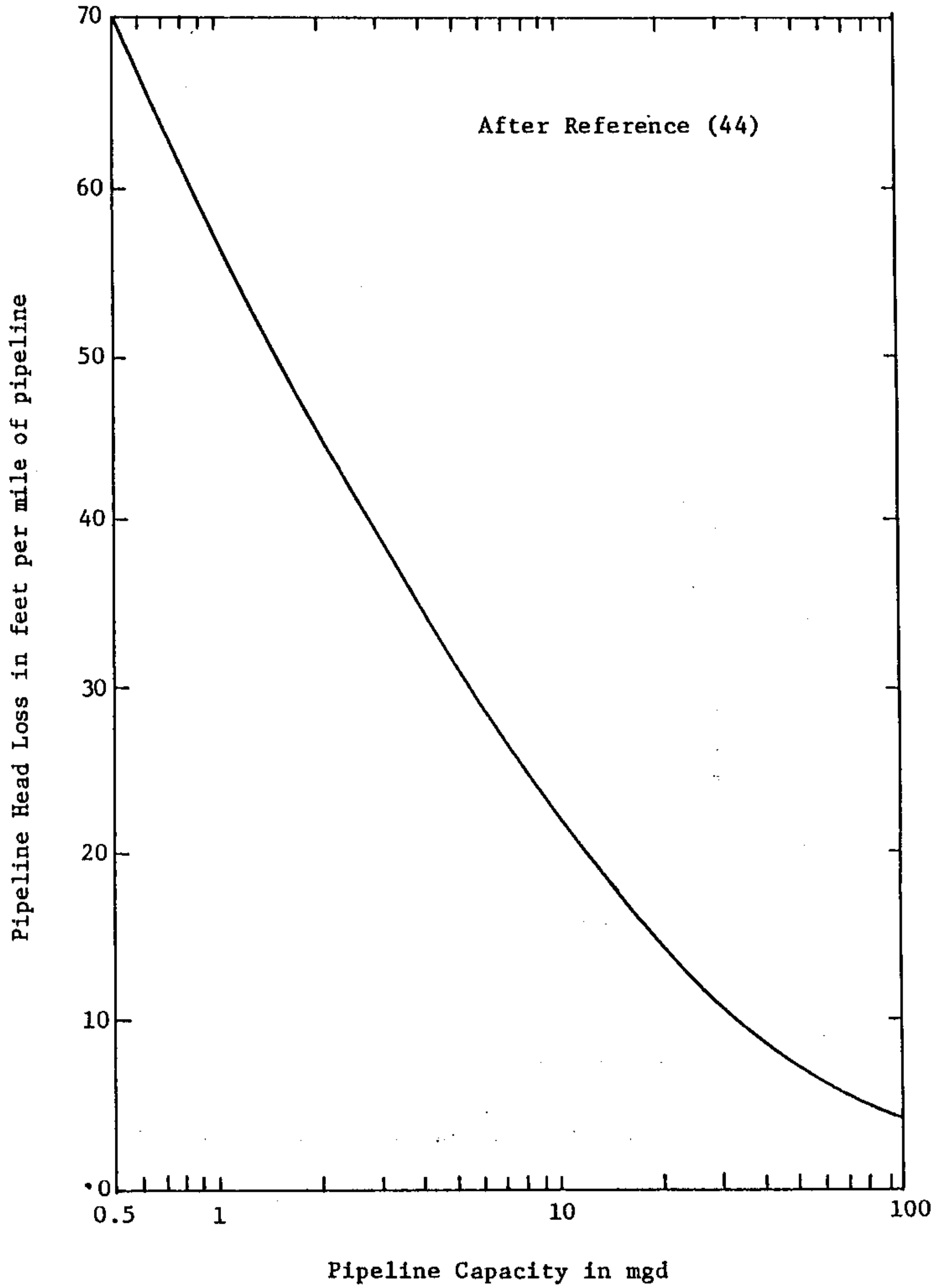

FIG. E-15.-ELECTRIC POWER REQUIREMENTS FOR HEAD LOSS IN PIPELINES

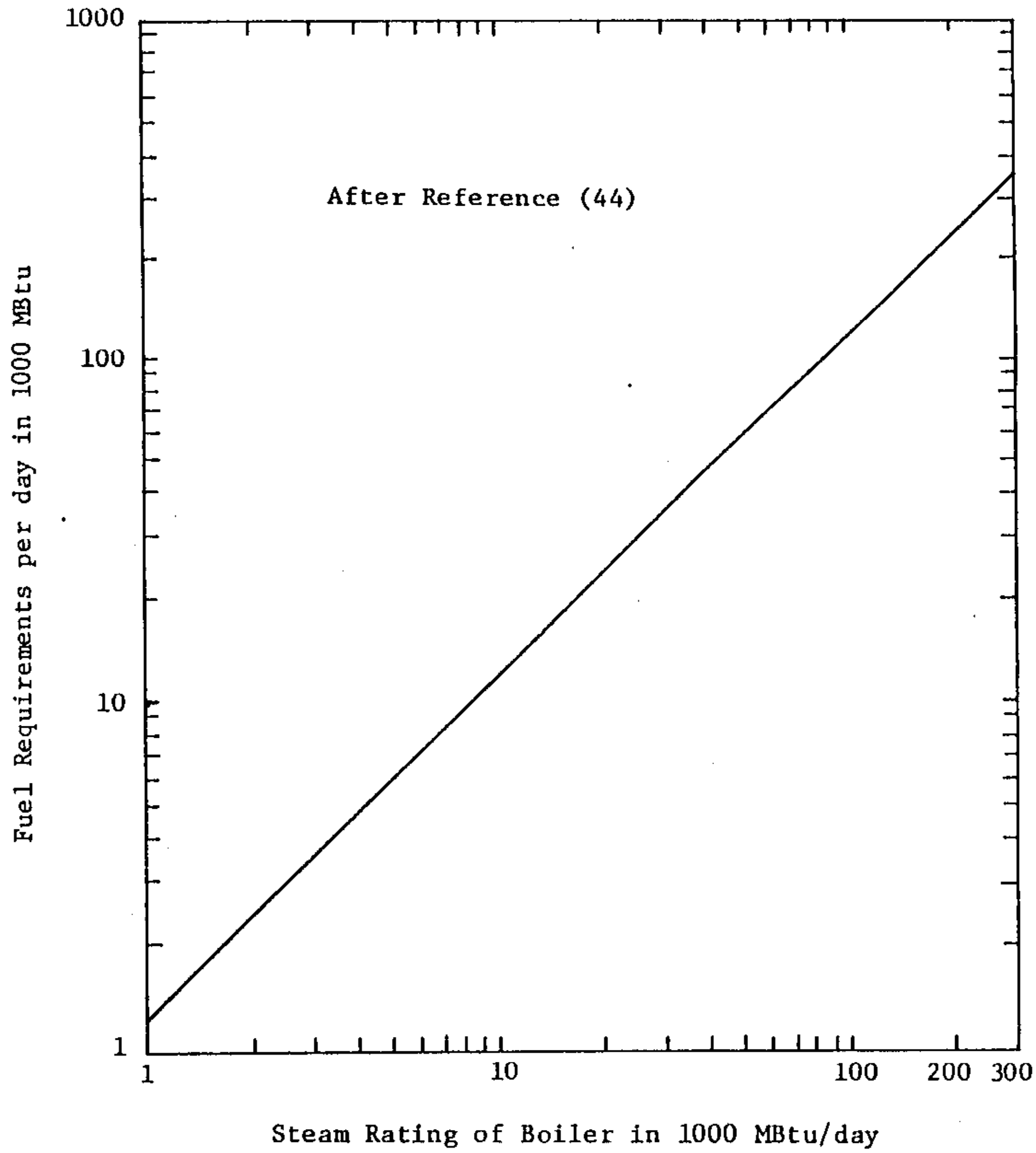

FIG. E-16.-FUEL REQUIREMENTS OF STEAM GENERATOR FOR FEED WATER HEATING

APPENDIX F

ENGINEERING NEWS RECORD (ENR) COST INDEXES

TABLE F-1.-AVERAGE YEARLY CONSTRUCTION COST INDEXES
(Base Year 1913 = 100)

Year	Cost Index	Year	Cost Index
1903	93.9	1940	242.0
1904	87.4	1941	257.8
1905	90.6	1942	276.3
1906	95.1	1943	290.0
1907	100.6	1944	298.7
1908	97.2	1945	307.8
1909	90.9	1946	346.0
1910	96.3	1947	413.2
1911	93.4	1948	460.7
1912	90.7	1949	477.0
1913	100.0	1950	509.6
1914	88.6	1951	542.7
1915	92.6	1952	569.4
1916	129.6	1953	600.0
1917	181.2	1954	628.0
1918	189.2	1955	659.7
1919	198.4	1956	692.4
1920	251.3	1957	723.8
1921	201.8	1958	759.2
1922	174.5	1959	796.9
1923	214.1	1960	823.6
1924	215.4	1961	847.1
1925	206.7	1962	871.8
1926	208.0	1963	900.7
1927	206.2	1964	936.5
1928	206.8	1965	971.2
1929	207.0	1966	1021.0
1930	202.9	1967	1070.4
1931	181.4	1968	1154.4
1932	157.0	1969	1270.5
1933	170.2	1970	1380.0
1934	198.1	1971	1570.6
1935	196.4	1972	1725.9
1936	206.4		
1937	234.7		
1938	235.8		
1939	235.5		

TABLE F-2 - MONTHLY CONSTRUCTION COST INDEXES
(Base Year 1913 = 100)

Year	Month											
	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1973	1812	1848	1857									
1972	1666	1679	1684	1687	1697	1712	1722	1739	1757	1768	1798	1807
1971	1466	1467	1494	1512	1543	1575	1598	1615	1640	1643	1644	1655
1970	1309	1311	1315	1329	1345	1369	1414	1418	1423	1434	1445	1445
1969	1216	1230	1238	1249	1258	1285	1283	1292	1285	1299	1305	1306
1968	1107	1114	1117	1122	1142	1154	1158	1171	1186	1191	1191	1201
1967	1043	1045	1048	1048	1063	1072	1083	1092	1097	1101	1101	1103
1966	988	997	998	1006	1014	1029	1034	1037	1037	1036	1037	1028
1965	948	957	958	957	958	969	977	984	986	986	986	988
1964	918	920	922	926	930	935	945	948	947	948	948	948
1963	883	883	884	885	894	899	909	914	914	916	914	915
1962	855	858	861	863	872	873	877	881	881	880	880	880

APPENDIX G

NOTATIONS

A, A_f, A_r = cross-sectional areas;

A_i = operation, maintenance and replacement cost in i-th year;

A' = total annual cost;

B = nondepreciating capital cost;

BPR = brine-to-product ratio;

C, C_F, C_L, C_P, C_S = total cost and component costs;

C_A, C_B = costs in regions A and B;

C_0, C_1, C_2 = coefficients of Muskingum method;

C_b = brine discharge rate;

C_i = feed water intake rate;

C_p = product water rate;

C_w = cooling water rate;

Ca_i = calcium concentration in feed water

c = levelized unit cost of water (cents per 1000 gallons);

c' = average unit cost of water (cents per 1000 gallons);

d = distance of levee from river bank;

f = plant capacity factor;

h = height of levee;

I_A, I_B = regional cost indexes;

I, I_1, I_2 = inflow rates;

K = storage constant;

K' = depreciable capital cost;

MSF = multistage flash;

N = plant life;

N' = stream days per year;

n = Manning roughness coefficient;

O, O_1 = outflow rates;

P_i = anticipated water production in i-th year;

P_f, P_r = wetted perimeters;

Q, Q_f, Q_p, Q_r = discharge rates;

Q' = desalting plant design capacity;

R_f, R_r = hydraulic radii;

r = interest rate in decimal form;

s = slope of streambed and floodway;

T, T_r = top widths;

TDS_i = total dissolved solids in feed water;

t = routing period;

U_f, U_r = mean velocity of flow;

V = volume content per mile of levee;

VTE-MSF = vertical tube evaporator - multistage flash;

x = constant used in Muskingum method;

y = depth of flow above bankful stage of river;

Z = cost of construction per mile of levee.

APPENDIX H

RELEVANT EXCERPTS

(Water Uses in the Navasota River Watershed and Its Vicinity)

Source: Turner, Collie, and Braden, Inc., Hydrology Report - Millican Lake, Navasota River, Texas, prepared for U. S. Army Corps of Engineers, Little Rock, Arkansas, October, 1973, pp. 3-5, 3-9.

(a) Population data for the basin, as projected by the Texas Water Development Board (TWDB) in December 1972, were used in combination with gallons-per-day-per-capita figures obtained from the Texas Water Plan, Brazos River basin, published by the TWDB in 1966. These projections were used to obtain the total municipal and industrial water requirements for the basin through the year 2020.

(b) The population projections revealed that none of the towns within the Navasota River basin are expected to increase in population. College Station and Bryan, which are adjacent to the basin and which may find need for a supplemental water supply, are projected to have continued growth.

(c) While it is anticipated that the construction of Millican Reservoir will result in some growth of Navasota and the area adjacent to the new lake, it is anticipated that the groundwater available in these areas will be sufficient to provide a suitable water supply and will likely be used because of economy.

(d) A study conducted for the City of Bryan by Spencer J. Buchanan, Consulting Engineer, concluded that the existing groundwater source of supply is adequate for the City's required water supply through 2020; therefore, no surface water requirement was projected for Bryan. The study of the Texas Water Development Board indicates that College Station can rely on groundwater supplies until 1990, but after that date, a supplemental supply of surface water will be required. On the basis of the foregoing, the municipal and industrial water use projections through 2020 were estimated as a constant use of the maximum reported water use in the basin for the past 10 years, plus the projected surface water requirement of College Station computed on an average per capita use of 200 gallons per day.

(e) The irrigation use for the Navasota basin for 1972 (the first year of the projection) was assumed to be equal to the highest usage recorded for the last 10 years. The irrigation surface water usage for 1990 was projected to be double that of 1964, in keeping with the projections for this general area in the Texas Water Plan, and thenceforth remain constant to 2020.

(f) The Texas Water Plan indicated that mining water use for this area should remain constant from 1960 to 1990 and then decrease to zero by 2020 as a result of depletion of the area's oil reserves. Therefore, the maximum recorded annual use was used for 1972 to 1990 and then was decreased linearly to zero in 2020.