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RECOMMENDATION ON THE IMPROVEMENT OF THE BARIT RIVER IRRIGATION SYSTEM

BICOL RIVER BASIN

PHILIPPINES

VOLUME I

METHODOLOGY FOR IRRIGATED RICE PRODUCTION SYSTEM PLANNING AND DESIGN

Final Report Submitted to the

UNITED STATES AGENCY FOR INTERNATIONAL DEVELOPMENT

In Partial Fulfillment of

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RECOMMENDATION ON THE IMPROVEMENT

OF THE

BARIT RIVER IRRIGATION SYSTEM

BICOL RIVER BASIN

PHILIPPINES

This report consists of the following volumes:

Volume I	Methodology For Irrigated Rice Production System Planning and Design		
Volume II	Summary Report		
	Annex A	Production and Incomes of Farm Households in the Barit River Irrigation System	
	Annex B	Procedures For Economic Analysis	
	Annex C	Design Data and System Layout	
Volume III	Computer Programs		

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INTRODUCTION

Irrigation water supply is only one input in an agricultural production system. However, the introduction of improved technology in one component of an overall production system often has implications for the overall system planning and management. Therefore, the development of successful irrigated rice production systems will hinge upon the perception and optimal design of all interacting components of a project.

One of the most important system planning components is the government's overall agricultural development strategy. For example, if the government should insist that all public-sector projects be financially viable and self-liquidating, then, it is conceivable that the farmers' increased income (resulting from the infrastructural investments) may be offset by the increased costs they have to bear, either directly from repaying the amortized loan for the infrastructure, or indirectly by the increase in irrigation water assessment, or the increase in rent from the increase in the capitalized land value. Conversely, the increased output may also be offset by a decrease in the output price, caused by external market forces, even if the farmers' production costs do not increase (12).

Farmer education is another component which has far reaching effects on the final production system. A certain level of farmer education is a prerequisite for many of the physical and infrastructural components to operate efficiently. However, the rehabilitation or changing of institutions does not guarantee the changing of habits in a short lapse of time. Innovations in the form of new credit institutions, organized irrigation and marketing facilities, and other innovations will always face the immediate problem of existing habitual behavior. For example, the sense of urgency in completing farm operations within irrigation schedules will not immediately take root among farmers who are used to prolonged slack periods, especially after harvest.

In setting up goals for development, a clear sequence of intermediate goals leading to the ultimate goal has to be listed. A subsistence farmer cannot be changed overnight into a fully commercialized one. His transformation follows sequential phases, the commencement of one can only begin after the completion of the preceding one. There must be minimal incongruencies between the intermediate societal goals and the values and objectives of the farmers at any particular stage of development. This is necessary in order to keep the farmers in pace with development (38).

In the design of irrigation systems it must be remembered that the development of socio-infrastructures may take a much longer time than the development of physical infrastructures. Therefore, it is important to design the physical infrastructure so it will have the flexibility to operate efficiently while the rural socio-infrastructure is undergoing the necessary developmental stages (36).

The sophistication of irrigation systems varies from continuous field-to-field irrigation, where water is available all the time and little or no control is exercised over supply schedule to the type of rotational system practiced in Taiwan, where supply schedule is based on 15 minutes intervals. That is to say the supply schedule may call for the initiation of water supply at 8:15 a.m. and termination of water supply at 2:45 a.m. the following day. This type of control has two implications. Firstly, there must be suitable organization to carry out the precise control. Secondly, the water distribution and measuring devices must be sufficiently accurate so that water delivery does not vary more than a few percentage from the design level (35).

Obviously, only a well maintained farm ditch can be expected to deliver water within a few percentage of the design level, and there is obviously little reason for engineers to design and construct a water delivery system with an accuracy far beyond expected maintenance capability. In other words, the irrigation water distribution systems, and the irrigated rice production system as a whole, will be a success only if the management sophistication required coincides with the existing social constraints.

Innovative design concepts and improved design formulae are presented in the chapters to follow. The application of these concepts and formulae, however, is an art which can be learnt only through experience.

CHAPTER I. A PHILOSOPHY OF DESIGN

Design procedures and formulae are tools important to successful engineering design and implementation. But excellent tools alone does not guarantee excellent results. The application of tools is, and probably always will be, an art, and one must always remember that the basic ingredient in a masterpiece is the understanding and control by the artist of the complete picture.

A. System Design and Management

Engineering systems, when implemented, never perform exactly according to design specifications. This statement is true for all engineering systems but perhaps more so for agricultural engineering systems. The lack of data, both in quantity and in accuracy, is an important contributing factor to this fact. It is impractical to demand that engineers be supplied with all necessary data before engineering design starts. This is so not only because the collection of needed field data costs money and requires time, but also because some of the data cannot be easily obtained before the system is implemented and in operation.

Therefore, it is extremely important for the engineers who are designing an irrigation system to consider and evaluate how these inaccuracies may cause the system performance to deviate from design specifications. Whenever necessary, the design engineer should consider these possible deviations and what management strategies are needed to deal with these problems and design sufficient flexibility into the system so the management strategies can be employed. In other words, both "design information" and "system flexibility" are cost items and suitable compromise between these two items are necessary.

Since design is essentially an estimation about the "future," systems design models (calculations) are rarely suitable to be used as management tools.

After implementation, experiences (and data) gained from operation of the actual system helps in the establishment of management models that more closely simulate the actual performance of the irrigation system and therefore are capable of suggesting optimum management strategies.

B. Social and Engineering Efficiencies

Engineers are trained to design irrigation systems which are efficient in the physical sense. However, irrigation systems, or for that matter, agricultural production systems in general, are also social institutions. It is therefore important for the engineer to consult and interact with not only agriculturalists, but with people who can make some judgment on the difficulties which may be encountered during the implementation of alternative systems. In other words, in calculating the implementation costs, one should not only include the cost of construction but also the cost of rural extension work as well. The establishment of highly structured rural infrastructure can be very costly and time consuming. Looking at system design from this point of view, one may often find that a less physically efficient irrigation system can give better overall efficiency in terms of total resources needed for the successful completion of a project.

CHAPTER II. IRRIGATION SYSTEMS

There are three major methods of rice irrigation in use throughout the world: continuous flow, continuous submergence (flooding), and intermittent (of which rotational is the most important).

A. Continuous Flow

In continuous flow irrigation, water is constantly moving across the paddy field and is maintained at a fixed level by the height of the field drainage spillways. It has the advantage of supplying the soil with oxygen and diluting hydrogen sulphide and other harmful substances due to poor drainage. There is also a limited adjustment of soil temperature and a saving of water management labor. Soil nutrients carried away by the flowing water and the wastage of water are disadvantages (13).

B. Continuous Submergence

For continuous submergence irrigation the water in the paddy field is maintained at a fixed level (stagnant) and enough water is supplied daily to replace water lost through evapotranspiration and seepage. It gives better response to timely application of fertilizer, compared with other methods of irrigation, and its high water level helps control weeds. It reduces the labor requirement for water management. Also, it requires less water than the continuous flow practice to irrigate the same production area. This is the traditional method used in most of Asia (6, 13).

C. Intermittent

In the practice of intermittent irrigation, water is supplied either at irregular intervals or on a regular schedule. The latter practice is called rotational irrigation. On a macro scale, it could mean that a region receives irrigation water for one out of three or five years. However, in recent times "rotational irrigation" has come to mean a highly structured irrigation scheme that supplies irrigation water to small farm units, on fixed schedules. Rotational irrigation is used increasingly throughout the world since it has many advantages over the other methods.

Types of Rotational Irrigation:

1. Rotation by main canal sections
2. Rotation by lateral or sub-lateral sections
3. Rotation by farm ditches

In case 1, irrigation water will be supplied in turn to different section of the main canal. For example, if the area covered by the main canal is divided into three sections, then water will be supplied to each section in turn, while other sections will be dry. In case 2, water flow in the main canal will be continuous while water supply to the laterals or sub-laterals will be intermittent according to predetermined schedules.

In case 3, the whole area to be irrigated will be divided into rotational areas. There may be any number of rotational areas and each of the rotational areas is further subdivided into rotational units, Figure 1. Irrigation water will be then supplied to each rotational unit within a rotational area according to schedule on an intermittent basis. For example, a certain rotational area is subdivided into four units, and the rotational interval is six days, then each unit will get its share of the irrigation application time in proportion to its area. The sum of all application periods for rotational units within the rotational area will be six days.

The implementation of rotational irrigation quite frequently leads to increased rice production. With continuous irrigation the rice plant root system distributes in the horizontal direction near the surface of the soil and becomes dark, short, coarse and weak. Under rotational irrigation the root system penetrates vertically about 20 percent deeper into the soil and is uniformly distributed with long and slender but strong gray-white branches.

Agronomists feel that one advantage of rotational irrigation is that it permits periodic aeration of the root zone and a decrease in the reduction process of the soil.

Rotational irrigation can eliminate waterlogging in depression areas where drainage is insufficient, thereby improving the soil environment. It will not appreciably affect the fertility of the soil as compared with continuous submergence irrigation. And, the soil moisture remains for the greater part of the rice growing period at or near the maximum water holding capacity, though not always under flooding (1, 2, 3, 5, 6, 13, 14, 15, 17, 20, 26, 27, 28).

From the water management point of view, rotational irrigation has many advantages. Improved distribution networks make possible an improvement in system manageability. There is an increase in the ability to utilize rainfall, since there is a storage capacity in the paddy most of the time. The increased water control and management capability under rotational irrigation will lead to increasing farmer confidence in the system.

The main difficulty (and therefore disadvantage) in the successful implementation of rotational irrigation is its requirement of well designed and relatively efficient farm level institutional and physical infrastructures. However, the planning of a simple rotational irrigation scheme is certainly, from an engineering point of view, not difficult. The overall water use efficiency of a rotational irrigation system generally depends upon the availability and the accuracy of information on climate (effective rainfall), soil (seepage, percolative loss) and agronomy (consumptive water requirement), etc.

From an engineering point of view, the chief advantages of rotational irrigation are two:

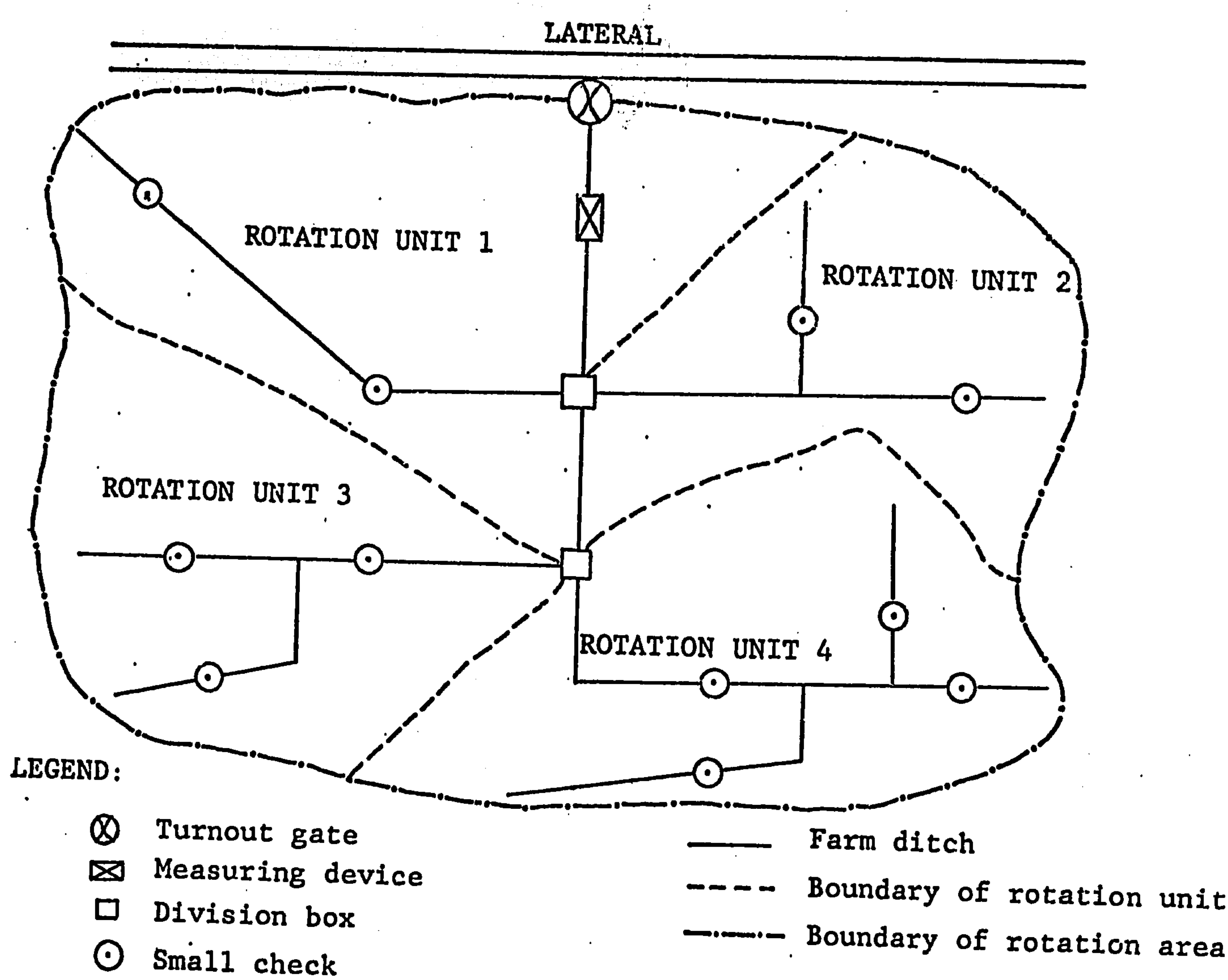


Figure 1. Rotation by farm ditch.

1. In a well designed rotational irrigation system the water carrying capacity of canals can often be reduced, therefore, both the seepage loss and the engineering cost can be reduced. However, rotational irrigation systems do require about 100 meters of farm ditch for each hectare of farm land, a terminal network, additional cost for the construction of farm ditches may be required.

2. The application of rotational irrigation can lead to substantial water saving. Quoting work done by the Chinese-American Joint Commission on Rural Reconstruction (JCRR, Taiwan), Chow (5, 6) indicated that rotational irrigation can save from 15.8% to 38.7% of the water needed by conventional continuous irrigation schemes.

There are many reasons for this water saving. Reduced seepage and percolative loss is one, and the ability to efficiently manage the irrigation supply can also lead to reduction of water waste (5, 6, 17, 19, 20, 30).

D. Terminal Facilities

A teminal network reduces a large irrigation block into smaller independent units. These small units would be self-contained and any irrigation problem could be easily isolated and overcome within such small units. The smaller units will also improve farmer cooperation and simplify irrigation extension. A large block may have four or five villages, each having its own leaders and social characteristics, and it would be a mammoth task to get them working together. However, a small unit may only contain a village or part thereof which could be more easily managed. It would be possible to stagger irrigation and transplanting dates within these units so that the utilization of local labor and machines could also be staggered within a small locality. Staggering activities between large blocks would call for the deployment of labor and machines over a very large area which would require a great deal of organization (31).

Finally, although the terminal systems in each agricultural unit are assumed to be independent, self-contained, they are in fact structurally and functionally linked with the secondary and primary infrastructural systems of the total project. For exmaple, in designing the block drainage system it is important to consider the removal problem in its aggregate rather than the units in isolation. Otherwise negative feedback within the aggregative secondary drainage system can be such that the drainage canal with a limited flow capacity would backup and reflood some of the units (12).

For agricultural administrators, the requirement for a highly structured rural infrastructure to implement rotational irrigation can be both a headache and a blessing in disguise. The ability to better guarantee a timely delivery of irrigation water may just give the farmers sufficient additional incentive to participate in the establishment of rural infrastructure needed by modern

Furthermore, fixed irrigation schedules will encourage farmers to closely follow production schedules, such as fertilization, weeding, etc. The ability to implement such schedules is essential to the success of modern agriculture.

Table 1. Comparison of Irrigation Method Characteristics (32)

Terms of Requirement	Rotational Irrigation	Flooding, Standing Water				Continuous Flooding Flowing Water
		Intermittent Irregular	Continuous Shallow (2.5 cm)	Medium (7.5 cm)	Deep (15 cm)	
1. Water Requirement	Moderate to low. Seepage loss relatively small. Total on-farm requirement about 600-700 mm.	About 800-900 mm with good in-field management. Higher amount will be needed if management is less satisfactory.	Moderate. Seepage loss relatively small. Total on-farm requirement amount to 600-800 mm.	Moderage same as shallow water.	Moderate to relatively high. Seepage loss high. Total requirement amounts to 700-1000 mm.	High to very high. Total water requirement around 1200-1500 mm.
2. Water Management Requirement	Requires high ability to deliver measured amount of water at specific time. Total water use will be 1000-1300 mm with a moderate level of irrigation management.	Requires high ability and proper control facilities.	Water management is minimum with adequate water supply. Field surface should be well prepared, dikes should be maintained to use as much rainfall as possible.	Same as shallow water but land leveling is less necessary. Dike must be maintained to reduce seepage.	If water supply is adequate, same as shallow water. If water supply is limited, dikes should be built and maintained well. For the use of herbicides, drain should be controlled.	Amount of water supply is large, water management is limited to shut-off. Drainage ability should be good, otherwise causes water logging at downstream depressions.
3. Weed Control	Ineffective, manual or mechanical weeding is necessary. Distribution and stability of herbicides are poor unless special water management provided.	Ineffective, distribution and stability of herbicides are likely to be poor. Manual or mechanical weeding is necessary.	Some control of grasses, sedges and broad leaf weeds is achieved but the effect is less than in greater depth of standing water.	Effectively controlled. Sedges and broad leaf weeds moderately controlled.	Effective for grasses and sedges. For the control of broad leaf weeds water level should be lowered for direct spray of hormones.	Depending on the depth of flow in the paddy. Degree of control will be the same as standing water.
4. Yield	Optimum.	Variable depending on the ability of maintaining soil moisture content at least field capacity condition.	Optimum.	Optimum.	Optimum to less than optimum.	Optimum.

CHAPTER III. EXISTING SYSTEM CAPACITY DESIGN FORMULAE

Maximum water demand for irrigated rice production generally occurs during the period of land soaking and preparation and transplanting if there is no rainfall or a large area is to be prepared in a short time period. This maximum demand determines the size of canals that are needed to carry the flow.

A. Conventional Formulas

The formula used by the Provincial Water Conservancy Bureau of Taiwan (37) for the determination of maximum canal or pump capacity can be written as:

$$Q = \left[\frac{AD_s}{N} + AD_t\right]\frac{1}{1-L} \qquad (1)$$

where:

Q = canal or pump capacity (maximum discharge), in m^3/day
A = area to be irrigated, in m^2
D_t = water requirement after transplanting field, hereafter called maintenance water, in m/day
D_s = $D_{ss} + D_{st}$, in m
N = time required to prepare area A, in days
L = conveyance loss, in decimals
D_{st} = standing water requirement, in m
D_{ss} = soil saturation water requirement, in m
= depth of soil saturation x porosity x (1 - degree of saturation)

Example: Depth of soil saturation = 0.5 m
Porosity = 40%
Degree of saturation = 30%
D_{ss} = 0.5 x 0.4 x 0.7 = 0.14 m

Chow (5) gave a revised form of equation (1) as follows:

$$Q = \frac{A}{8.64}\left[\frac{d_s}{P_s} + \frac{d_r}{P_r}\right]\frac{1}{1-L} \qquad (2)$$

where:

Q = required canal capacity, in m^3/sec
A = irrigated area, in hectares
d_s = standing water requirement + soil saturation water requirement
d_r = depth of water for each application to the transplanted field, in m
P_s = time period for field soaking, in days
P_r = rotation interval, in days
L = conveyance loss, in decimals

Equation (2) is equal to equation (1) since d_r equals D_t times P_r.

The formula used in Japan is (37):

$$R_{max} = \frac{10A}{n}\left[s + (n-1)d\right]\frac{1}{1-L} \qquad (3)$$

where:

R_{max} = maximum water required, in m^3/day
A = area to be puddled, in hectares
d = maintenance water, in mm/day
s = standing water requirement, depth of water required for field preparation = soil saturation water requirement
n = number of days in the land preparation period
L = conveyance loss, in decimals

The three equations are derived assuming the land preparation rate is a constant throughout the entire land preparation period and the effective rainfall is negligible in the short period of days. Equations (1) and (2) also assume maintenance water is supplied in addition to puddling water from the first day and daily thereafter until the end of land preparation. Equation (3) assumes maintenance water is not supplied the first day but is supplied daily thereafter until the end of land preparation.

Maintaining a constant rate of land preparation for the entire land preparation period causes the demand for puddling water to be constant, Figure 2a. Figure 2b shows the variable flow rate is the one supplying maintenance water. This flow rate will increase daily and reach maximum on the last day of land preparation. The sum of the two flow rates for the last day of land preparation determines the maximum water demand to be handled, Figure 2c.

Disadvantages of the conventional formulas are as follows. The linear increase of flow rate during the land preparation period is difficult to implement in canal operation. Full capacity of the canal is not used except for the last day of land preparation, resulting in an expensive canal system being under utilized except for 2 or 3 days of the year.

B. Goor-Zijlstra-Wen Formula

Van de Goor and Zijlstra developed the following formula (34) during an FAO assingment in Malaysia from October 1961 to April 1963. The formula was first published in 1968.

$$I = \frac{Me^{MT/s}}{e^{MT/s}-1} \qquad (4)$$

(a)

(b)

(c)

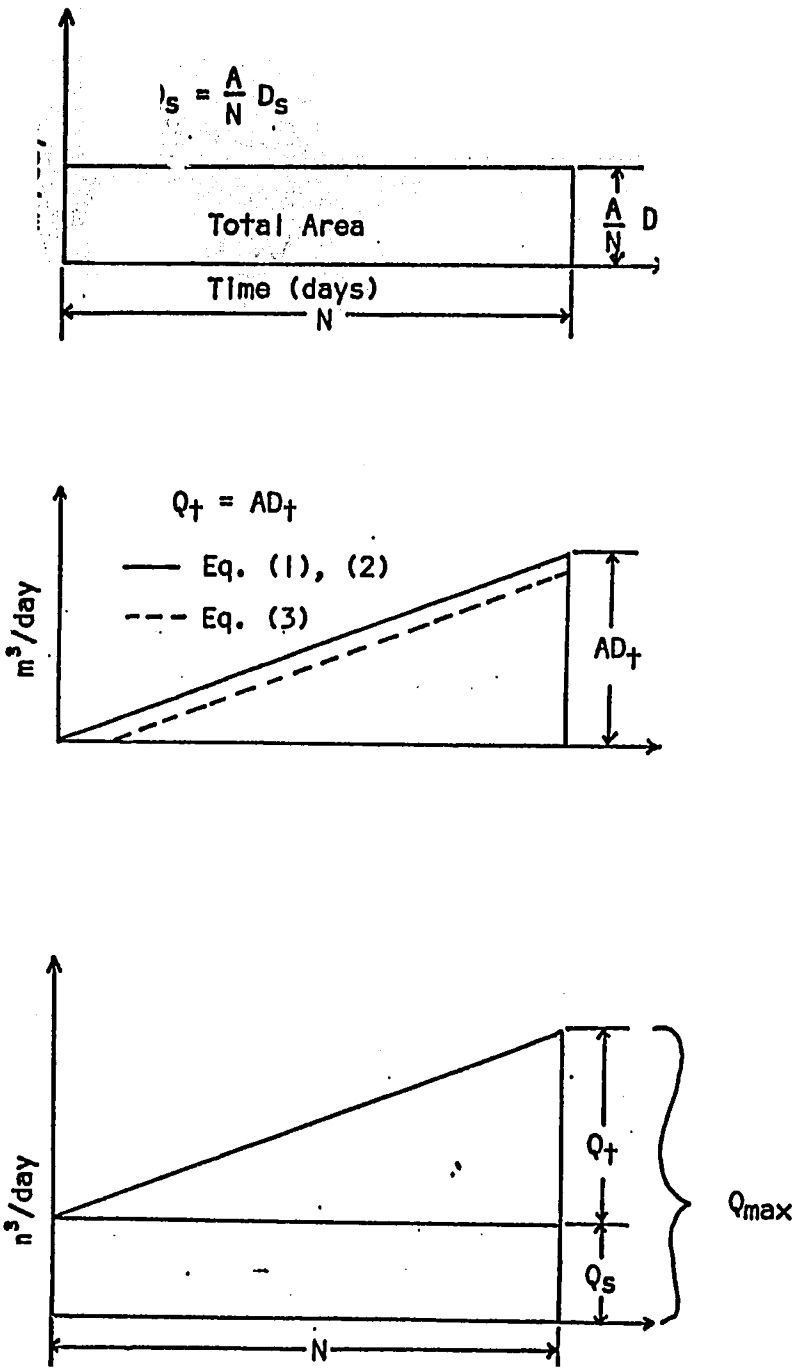


Figure 2. Schematic diagrams of water use in land preparation.

where:

M = maintenance water, in mm/day
I = water required during land preparation for the entire area to be irrigated, in mm/day
T = duration of preparation period, in days
s = depth of water requirement for field preparation, in mm

Wen (37) published the following equation in 1972. The publication was based on his M.S. thesis which was completed in 1970.

$$Q = \frac{AD_t}{E_c(1-e^{-(D_t/D_s)N})} \qquad (5)$$

where:

Q = required canal capacity, in m^3/day
A = total area to be prepared, in m^2
D_t = maintenance water, in m/day
D_s = water requirement for field preparation, in m
= standing water requirement + soil saturation requirement
N = days to prepare the entire area
E_c = conveyance efficiency, in decimals

It can be shown that equations (4) and (5) are equivalent to each other.

Cheng gives a more flexible form of the equation in which the soaking water can be applied in more than one day (4, 18).

$$Q = \frac{A}{8.64}\left[\frac{d}{1-K^n}\right]\frac{1}{1-L} \qquad (6)$$

where:

$$K = \frac{\frac{D}{r} - \frac{d}{2}}{\frac{D}{r} + \frac{d}{2}}$$

and

Q = canal capacity needed, in m^3/sec
A = total area to be prepared, in hectares
d = maintenance water, in m/day
D = depth of water requirement for field soaking, in m
n = number of days to prepare the area
L = conveyance loss, in decimals
r = number of days for the application of soaking water to each area

When r equals 1.0, all the required soaking water is applied in one day equation (6) gives the same value as the previous two equations.

The modified formulas were developed assuming a constant flow rate during the entire land preparation time period. Maintenance water is supplied daily and effective rainfall is negligible for the short period of days. With these assumptions the size of the area prepared daily will decrease as time progresses, Figure 3.

A disadvantage of the above modified formulae is the requirement that maintenance water be applied daily to each field during the land preparation period. This application of a small flow is difficult to do efficiently when the canals have been designed for much larger flow rates. Also, when preparing large areas over a long period of time the daily area prepared becomes very small in the last part of the period. Maximum flow rates determined by the modified formula are smaller than those of the conventional formula, resulting in a smaller canal requirement and lower construction costs.

C. Comparison of Conventional and GZW Formulae

A comparison between the two land preparation rates is made by graphing the percent of total area prepared versus the percent of total time used, Figure 4.

Another way to compare the two formulas is to look at the flow rates. Letting Q_p be the percent difference between the two flow rates, the equation can be written as:

$$Q_p = \frac{Q_c - Q_m}{Q_c} * 100\%$$

where Q_c and Q_n are the conventional and modified flow rates, respectively.

Using equations (1) and (5) as representative of the two types, the equation for Q_p becomes:

$$Q_p = \frac{\left[\frac{AD_s}{N} + AD_t\right] - \left[\frac{AD_t}{1-e^{-(D_t/D_s)N}}\right]}{\frac{AD_t}{N} + AD_t} * 100\%$$

Combining terms and simplifying yields:

$$Q_p = 1 - \frac{1}{\left[\frac{D_s}{ND_t} + 1\right]\left[1-e^{-(D_t/D_s)N}\right]} *100\% \qquad (7)$$

Since both formulas are continuous functions over the time interval N, equation (7) can be differentiated. Setting the differential equal to zero.

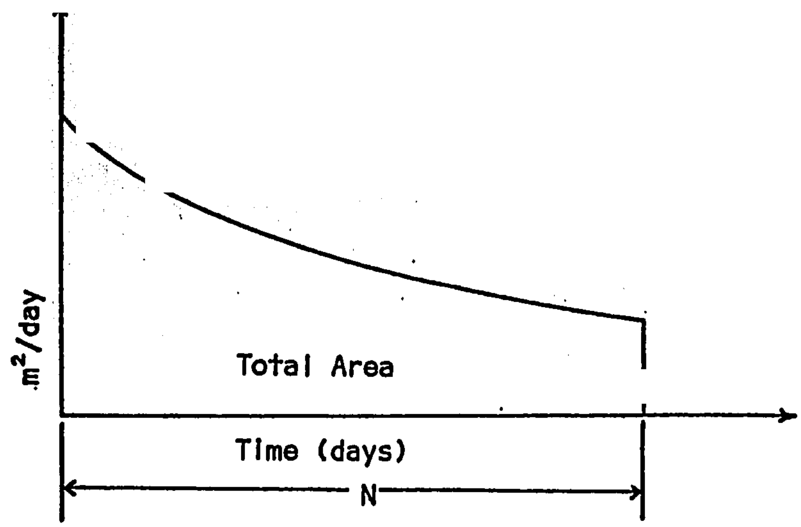


Figure 3. Schematic diagram of land preparation rate.

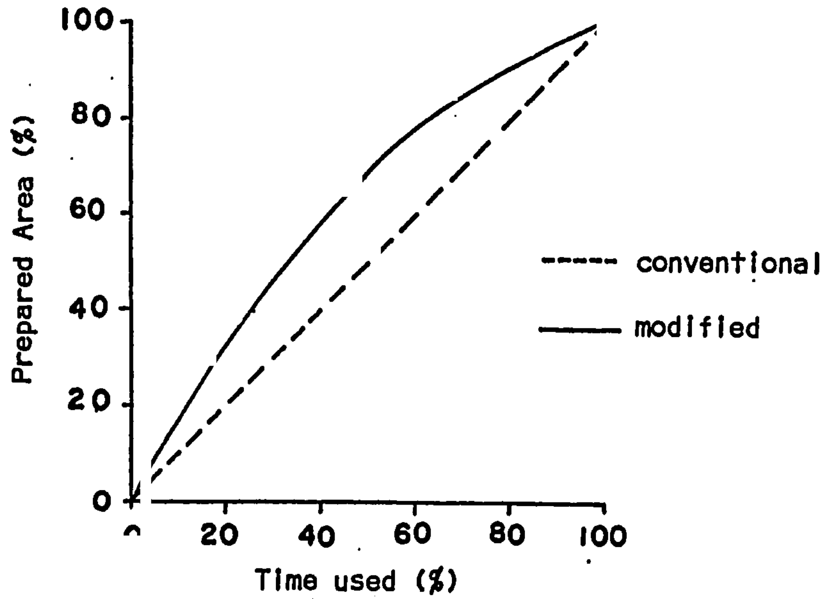


Figure 4. Rates of land preparation for th
conventional and modified formul

and solving will give the point where Q_p is at maximum. For simplification purposes let $R = D_s/ND_t$, then equation (7) becomes:

$$Q_p = \left[1 - \frac{1}{(R+1)\ (1-e^{-1/R})}\right] * 100\% \qquad (7a)$$

Differentiating Q_p with respect to R

$$dQ_p = \left[(R+1)\ (1-e^{-1/R})\right]^{-2} \left[(1-e^{-1/R})dR + (R+1)\ (-\frac{1}{R^2}\ e^{-1/R})dR\right] = 0$$

$$0 = 1-e^{-1/R} = (r+1)\ (-\frac{1}{R^2}\ e^{-1/R})$$

$$e^{1/R} = \frac{R^2 + R + 1}{R^2}$$

Solving by trial and error R = 0.5576367, substituting this value into equation (7a) gives a maximum value for Q_p of 22.98365 percent, for a graphical representation see Figure 5.

D. Conventional and GZW Formulae Management Problems

The two formulae types, conventional and GZW, presently used for System Capacity and capacity design, are derived without giving consideration to the management requirements. Consequently, the design requirements result in a system with management problems when implementation begins.

Requiring the maintenance water to be supplied daily will result in low water conveyance efficiency. The conveyance losses will be large because a small flow will have a low velocity and large wetted perimeter per unit volume, resulting in more seepage, evaporation and percolation losses. The low flow rate demanded is difficult to measure and control with accuracy in canals designed for much larger flows. Also, this requirement necessitates a transition from a daily maintenance water supply to being supplied in the rotation pattern upon completion of transplanting. The transition can cause management difficulties.

The linear increase in water use throughout land preparation is not practical in system operation. It implies a continuous adjustment of water controls from the water supply source down to the area being prepared. Generally, the canals are operated at full capacity, wasting excess water and lowering water use efficiency.

A constant flow rate will eliminate the need for continuous adjustment of water control. Elimination of this difficulty by use of the Goor-Zijlstra-Wen formula brings in another problem, a decrease in area prepared daily. At the end of the preparation period, land soaking times can become excessively long.

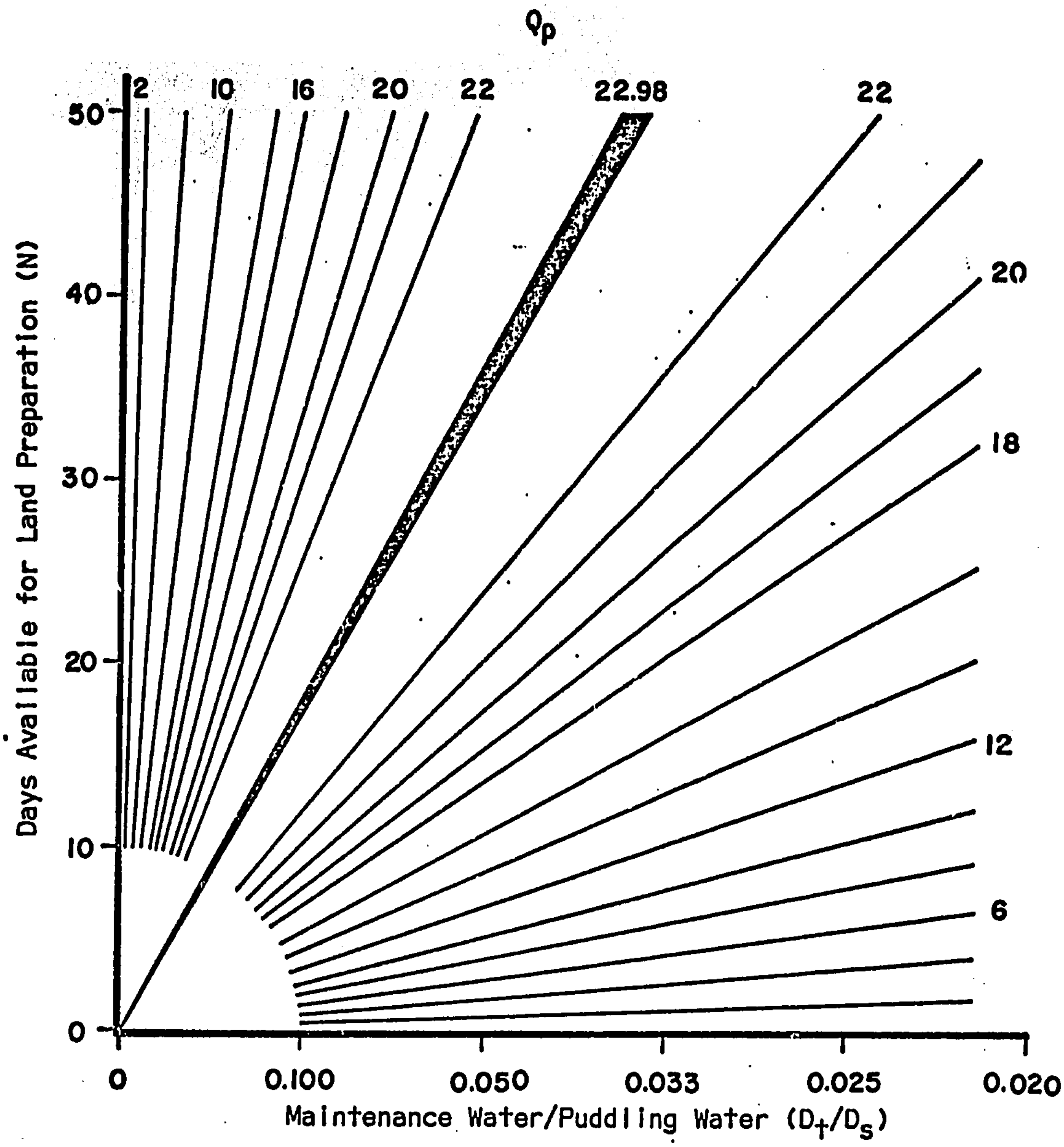


Figure 5. Percent difference between conventional and modified formula flowrates.

$$Q_p = \frac{Q_c - Q_m}{Q_c} * 100\%$$

CHAPTER IV. DEVELOPMENT OF IMPROVED CANAL CAPACITY DESIGN FORMULAE, (9, 10)

One need for the improvement of on-farm management is the development of improved design procedures and formulas for on-farm irrigation systems and the means to implement them. Resources such as financing to pay for construction, equipment necessary to do the construction, and personnel trained to operate the system once constructed are generally limited. Therefore, designs of simple and economic irrigation and drainage systems and economic water control structures are included in this need (33).

What is required to satisfy this need? First, the improved system must have higher conveyance and use efficiencies than the traiditional irrigation system design presently in use. Second, it must be manageable with limited numbers of trained personnel in the beginning but be flexible enough to change as more trained people become available and greater control is desired. Third, the cost of the system must be kept to a minimum. And, the design procedures must be general so they can be applied to different areas.

To fill the requirements mentioned above, two areas will be examined. First, new maximum canal capacity formulae which are devoid of the management problems of the conventional and GZW formulae will be developed. Most of these problems arise from daily application of maintenance water during land preparation. Therefore, the delivery of maintenance water is restricted to the post-transplant rotation schedule for this development. Second, techniques of water management which are integrable with the irrigation system design and offer flexibility to management personnel will be developed. These techniques have to consider local social constraints for the implementation of a successful irrigation system.

The basis for the development of the system canal capacity design formulae are as follows:

1. Over the cropping cycle, including the period for land preparation, maintenance water is delivered according to a predetermined rotational schedule. This requirement implies that maintenance water does not have to be supplied the first rotational period.

2. The number of days during which the peak irrigation demand of the rice production system is a constant is maximized.

3. During a rice production cycle, irrigation water is required for the following requirements.

   a) Maintenance water to meet evapotranspiration requirements and field percolation and seepage losses, D_t (m/day).

   b) Pre-tillage soil conditioning requirement, D_{ss} (m). Sometimes this requirement is called soaking water because it saturates the soil and reduces tillage power requirement by reducing the soil to a plastic state. D_{ss} is determined by the soil type, its structure at the time of the initial water application and the desired depth of saturation.

c) Flooding or standing water requirement, D_{st} (m). D_{st} need not be applied during the land preparation period. Rice seedlings can be transplanted as long as soil is fully saturated. Therefore, the standing water can be applied following transplanting. Normally, as the traditional design formulae indicates, D_{ss} and D_{st} are applied in one operation, but this is not necessary.

4. For convenience, canal conveyance losses are expressed as a percentage of the total system water demand.

5. Effective rainfall is assumed to be negligible over the period of land preparation.

Based on the above, some restrictive conditions can be stated:

1. The sum of pre-tillage soil conditioning and standing water requirements, D_s, is divided into two parts, D_{s1} and D_{s2}. D_{s2} can be applied before, or within a few days of transplanting. For convenience, it can be scheduled immediately following the application of D_{s1} to the entire production area under consideration.

$$D_{s1} + D_{s2} = D_s = D_{ss} + D_{st}$$

2. D_{s1} must be equal to or greater than the water used for pre-tillage soil conditioning.

$$D_{s1} \geq D_{ss}$$

During the first N days, the daily water requirement is Q_{ij},

where

i = 1, 2, 3, ..., n.
j = 1, 2, 3, ..., S.
n = number of rotational periods in N days, where N/S must be an integer.
S = rotational interval in days.

For easy management, the irrigation water requirement should not change during a rotational period. Therefore, for any particular irrigation period I, the daily water requirement is

$$Q_{Ij} = a_{Ij} D_{s1} + S \sum_{i=1}^{I} (a_{i-1,j} D_t) \qquad \text{(A)}$$

where

a_{ij} = the land preparation rate on the jth day of the ith rotational period.
a_{oj} = 0.

It can be demonstrated that the maximum number of days during which the system peak water demand can be maintained at a constant flow rate is (n + 1)S. n*S is the number of days D_{s1} is delivered to the entire area under consideration, and the next S days is the first rotational period during which D_{s2} is delivered to $a_{n+1,j}$, Figure 6.

Let $a_{n+1,j} = a_{1j}$.

$$Q_{n+1,j} = D_{s2}*a_{n+1,j} + A*D_t = D_{s2}*a_{1j} + A*D_t \qquad \text{(B)}$$

To maintain a constant system irrigation water demand during the first n + 1 rotational periods,

$$Q_{ij} = Q_{n+1,j} = \text{constant} = Q \qquad \text{(C)}$$

with

$$a_{1j} = \frac{Q}{D_{s1}} \qquad \text{(D)}$$

$$a_{2j} = \frac{Q(D_{s1} - S*D_t)}{D_{s1}^2}$$

$$\cdot$$

$$\cdot$$

$$\cdot$$

$$a_{nj} = \frac{Q(D_{s1} - S*D_t)^{n-1}}{D_{s1}^n}$$

$$A = \sum_i^n \sum_j^s a_{ij} = S*\sum_i^n a_i$$

Substituting into equation (A) yields,

$$Q_{nj} = Q = a_{nj}*D_{s1} + S\sum_{i=1}^{n} (a_{i-1,j}*D_t)$$

$$\left[\frac{Q(D_{s1} - S*D_t)^{n-1}}{D_{s1}^n}\right] D_{s1} + (A - a_{nj}*S)D_t$$

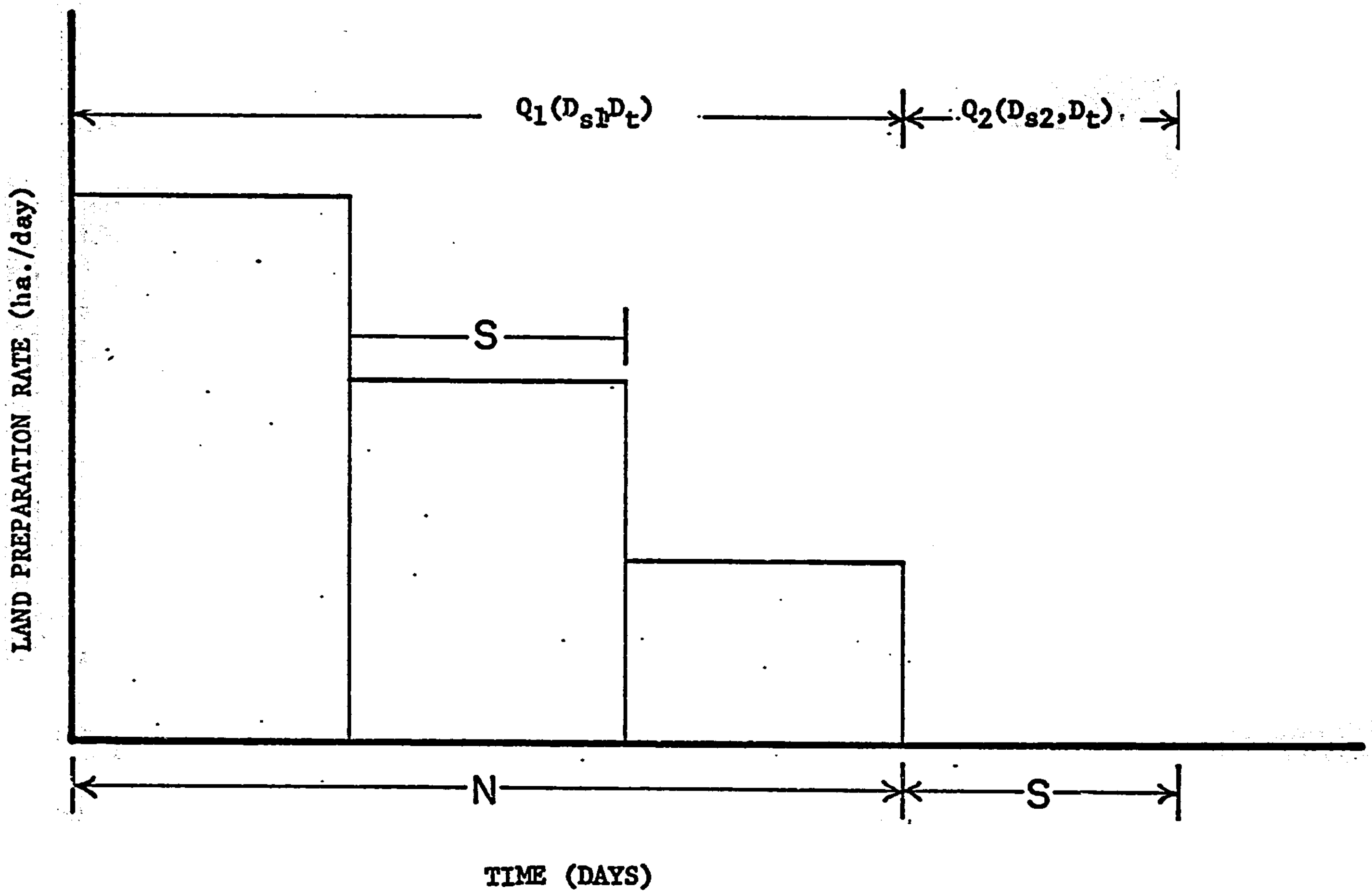


Figure 6. Time intervals of Q_1 and Q_2 with D_s applied in two operations.

$$Q = \left[\frac{Q(D_{s1} - D_t*S)^{n-1}}{D_{s1}^n}\right] D_{s1} + A*D_t - \left[\frac{Q(D_{s1} - D_t*S)^{n-1}}{D_{s1}^n}\right] D_t*S$$

$$= A*D_t + \frac{Q(D_{s1} - D_t*S)^{n-1}}{D_{s1}^n} (D_{s1} - D_t*S)$$

$$= A*D_t + \frac{Q(D_{s1} - D_t*S)^n}{D_{s1}^n}$$

$$1 = \frac{A*D_t}{Q} + (1 - \frac{D_t*S}{D_{s1}})^n$$

Therefore,

$$Q = \frac{A*D_t}{1 - (1 - \frac{D_t*S}{D_{s1}})^n} \qquad (8)$$

Using equations (B), (C), (D) and (8)

$$\frac{D_{s2}}{D_{s1}} = (1 - \frac{D_t*S}{D_{s1}})^n \qquad (9)$$

The condition set forth by equation (9) guarantees minimum peak system irrigation water demand. The peak demand is spread over (n + 1)S days, during which D_{s1} is supplied to the entire area, A, and D_{s2} is supplied to a_{1j}. Since D_t is generally treated as a constant for any particular rice production season, D_{ss} and D_{st} are the only sources contributing toward the peak system irrigation water demand.

However, equation (9) can be modified into equation (E) without violating any conditions previously stated.

$$0 \leq \frac{D_{s2}}{D_{s1}} \leq (1 - \frac{D_t*S}{D_{s1}})^n \qquad (E)$$

When D_{s2} is zero, $D_{s1} = D_{ss} + D_{st} = D_s$, and the peak system irrigation water demand is a constant for the first n*S days of system operation, a higher Q will result.

$$Q = \frac{A*D_t}{1 - (1 - \frac{D_t*S}{D_s})^n} \qquad (10)$$

Including canal conveyance losses, L, the equations become,

$$Q = \left[\frac{A*D_t}{1 - (1 - \frac{D_t*S}{D_{s1}})^n}\right]* \frac{1}{(1 - L)} \qquad (8a)$$

$$Q = \left[\frac{A*D_t}{1 - (1 - \frac{D_t*S}{D_s})^n}\right]* \frac{1}{(1 - L)} \qquad (10a)$$

See Appendix C for the development of equations for Q when N/S is not an integer.

A method of comparison is to look at the percent differences between the three flow rates, conventional, GZW, and the one given by equation (10). Letting Q_c, Q_{pc}, Q_m, Q_{pm}, Q_i be the conventional flow rate, percent difference between the conventional and equation (10) flows, modified flow rate, percent difference between the GZW and equation (10) flows, and the equation (10) flow rate, respectively, the equations can be written as:

$$Q_{pc} = \frac{Q_c - Q_i}{Q_c} * 100\% \qquad Q_{pm} = \frac{Q_m - Q_i}{Q_m} * 100\%$$

Figures 7 and 8 are graphs of Q_{pc} and Q_{pm} where equation (10) is used for the improved formula, therefore the values of Q_{pc} and Q_{pm}, respectively, are only correct for integer values of N/S.

Figure 9 shows another comparison between the three formulas, the percent of total area prepared versus the percent of total time used. As the rotational interval approaches 1.0 the improved formula curve approaches the GZW formula curve.

Example 1. <u>Numerical Comparisons</u>

The known factors are:

A = 100 hectares = 10^6 m^2
N = 20 days
S = 5 days $\quad n = 4$
D_t = 0.008 m/day
D_s = 0.15 m $\quad$ where D_{ss} = 0.10 m and D_{st} = 0.05 m
L = 0.40

1. Conventional equation, (1):

$$Q = \left[\frac{A*D_s}{N} + A*D_t\right]\frac{1}{1 - L} = \left[\frac{10^6*0.15}{20} + 10^6*0.008\right]\frac{1}{1 - 0.4}$$

$= 25833.33\ m^3/day$

$= 0.299\ m^3/sec$

2. GZW equation, (4):

$$Q = \frac{A*D_t}{\left[1 - e^{-(D_t/D_s)N}\right](1 - L)} = \frac{10^6*0.008}{\left[1 - e^{-(0.008/0.15)20}\right]0.6}$$

$= 20329.97\ m^3/day$

$= 0.235\ m^3/sec$

3. Improved equation case I, $D_{s2} > 0$ (9), (8a):

$$\frac{D_{s2}}{D_{s1}} = \left(1 - \frac{D_t S}{D_{s1}}\right)^n \qquad 0.15 \geq D_{s1} \geq 0.10$$

By trial and error $D_{s1} = 0.124$ m $D_{s2} = 0.026$ m

$$Q = \frac{A*D_t}{\left[1 - \left(1 - \frac{D_t*S}{D_{s1}}\right)^n\right](1 - L)} = \frac{10^6*0.008}{\left[1 - \left(1 - \frac{0.008*5}{0.124}\right)^4\right]0.6}$$

$= 16890.18\ m^3/day$

$= 0.195\ m^3/sec$

4. Improved equation case II, $D_{s2} = 0$ (10a):

$$Q = \frac{A*D_t}{\left[1 - \left(1 - \frac{D_t*S}{D_{s1}}\right)^n\right](1 - L)} = \frac{10^6*0.008}{\left[1 - \left(1 - \frac{0.008*5}{0.15}\right)^4\right]0.6}$$

$= 18758.34\ m^3/day$

$= 0.217\ m^3/sec$

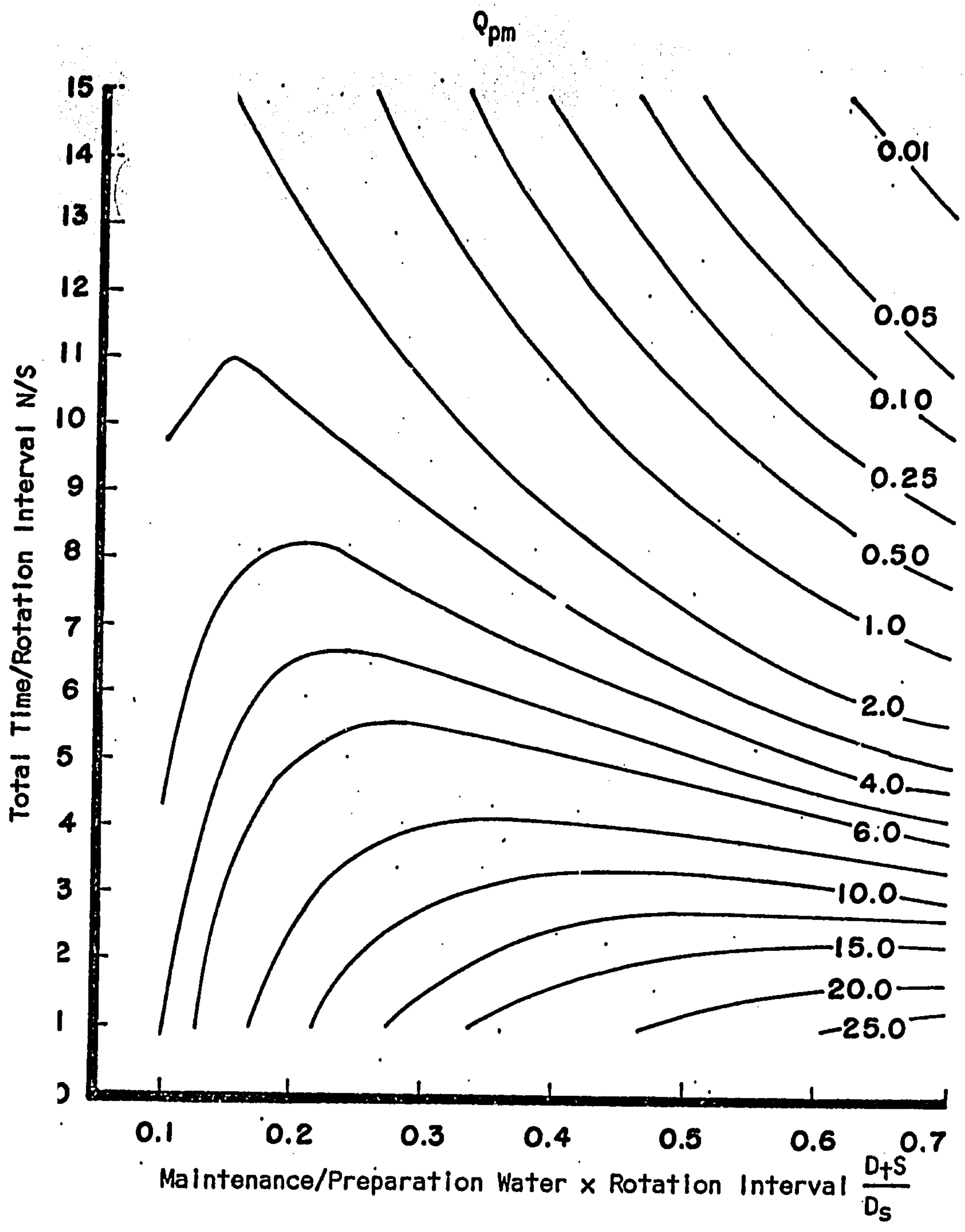


Figure 7. Percent difference between modifed and improved flowrates.

$$Q_{pc} = \frac{Q_c - Q_I}{Q_c} * 100\%$$

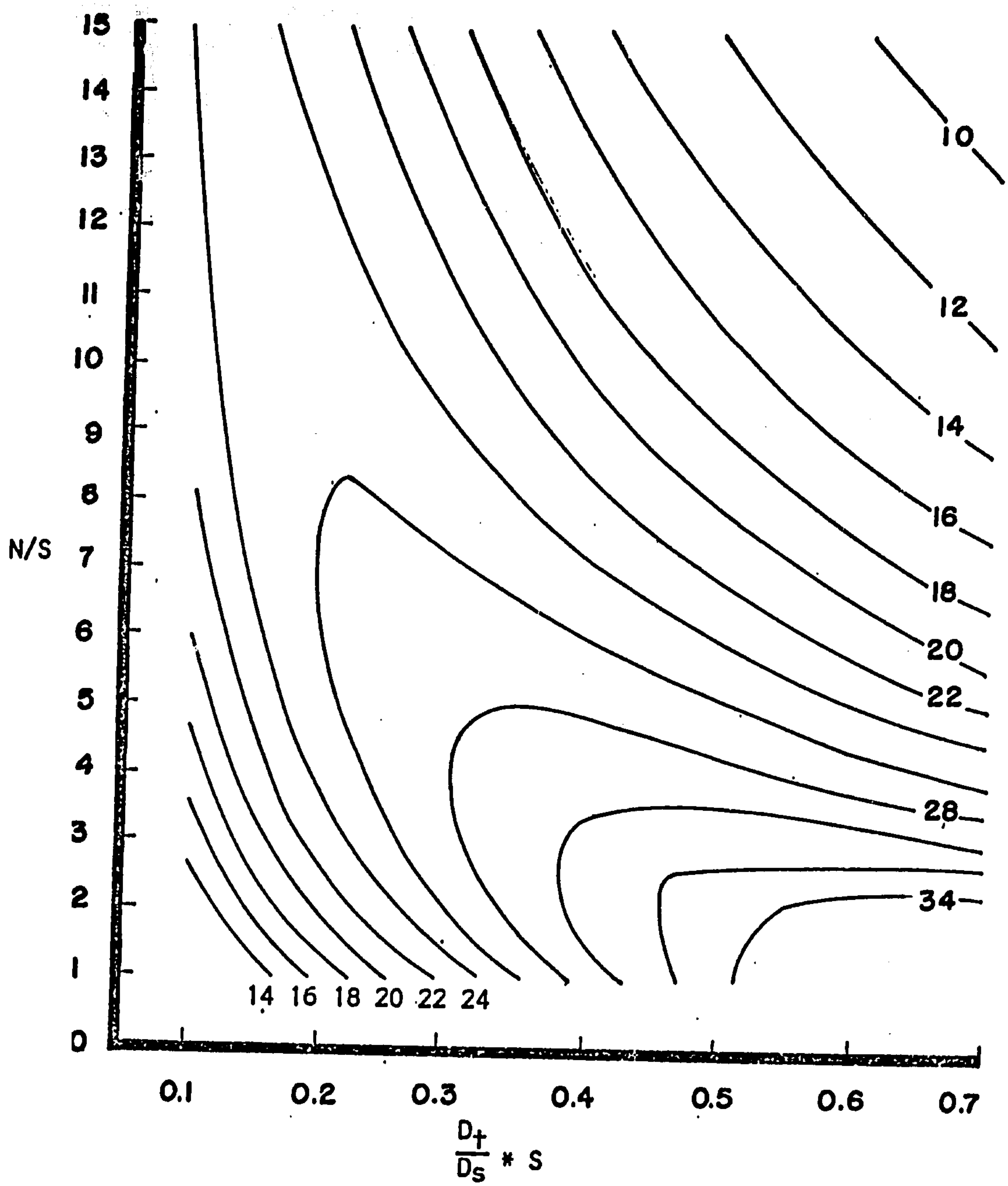


Figure 8. Percent difference between the conventional and improved formula flowrates.

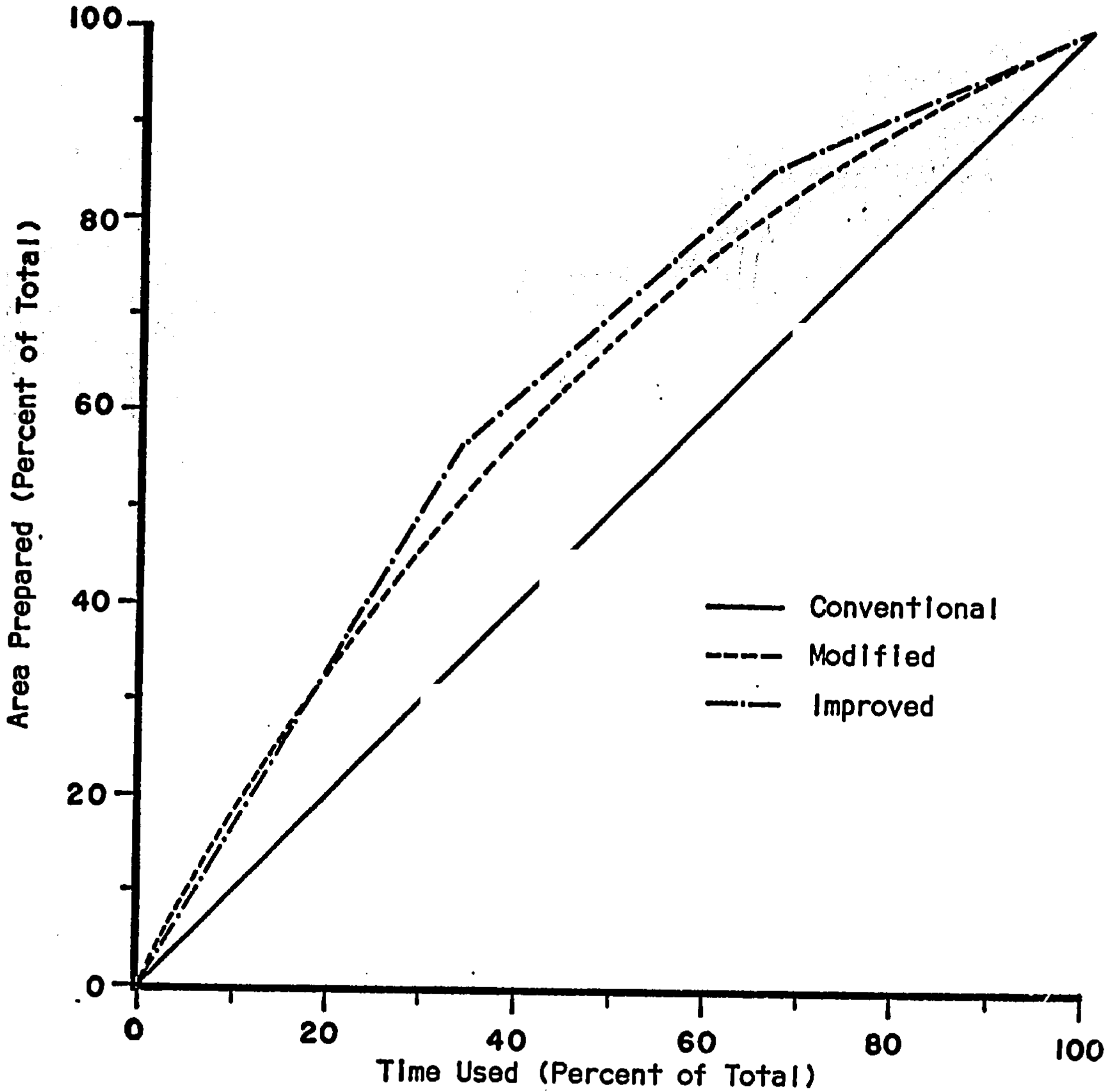


Figure 9. Comparison of land preparation rates.

S = 7 days D_s = 0.13 m D_t = 0.009 m/day

N = 21 days

When management and operation practices are considered at the design stage, a system with a high probability of success upon implementation can be constructed. The improved formula considers these practices in its derivation. Maximizing the number of days of constant peak irrigation demand reduces the number of flow adjustments and decreases the sizes of canals needed. The daily rate of land soaking only decreases at the end of a rotational period. Supplying maintenance water in the rotational pattern from the beginning of water application results in larger and more easily measured flow rates. Also, the transition from daily to rotational period application of maintenance water is eliminated. All of the above contribute to increased efficiency of irrigation system operation and water usage.

The following symbols are used in this chapter.

A = entire area to be irrigated

a_{ij} = land preparation rate on the jth day of the ith rotational period

D_s = water required for pre-tillage soil conditioning and submergence

D_{ss} = water required for pre-tillage soil conditioning

D_{st} = water required for flooding or submergence

D_{s1}, D_{s2} = first and second applications of water to satisfy D_s requirement

D_t = water required to replace percolation and consumptive use losses

L = system conveyance loss

N = days to supply D_s to A

n = number of rotational periods in N days, N/S

Q = maximum canal or pump capacity

S = rotational interval in days

CHAPTER V. RICE PRODUCTION SCHEDULING

A. Introduction

The importance of production scheduling is probably one of the least understood components in the design of rice production systems. The engineer, together with agronomist, controls the timing of crop production activities through production scheduling. The conflicting demands coming from efficient water utilization, climatic constraints, availability of production resources, etc., must be reconciled in the timing of production activities. In the tropic environment where most of the world's paddy rice is being grown today, rice production can be continued around the year, and it is important to optimize production scheduling.

B. Effects of Climate on Rice Production

A great deal is known about the effect of individual climate factors on rice production. However, many of the influences have not been quantified. Furthermore, it is even more difficult to try to integrate the effects of several climate factors on rice production.

According to the International Rice Research Institute (IRRI), the average number of grain per square meter, which is highly correlated with yield, is affected by the temperature found during panicle development. A lower temperature during the panicle development stage generally will result in an increase in the average number of grains per square meter, leading to a higher yield potential.

The rice plant response to nitrogen application will decrease with any decrease in the net solar energy received during the last 45 days before harvest. It has also been found that rice yield is positively correlated to daily solar radiation and negatively correlated with daily mean temperature during the 25-day period preceding flowering. A combination of high solar radiation and low temperature during this period will lead to high yield potentials.

Heavy winds are generally undesirable to rice production. The occurrence of typhoon close to harvesting time can cause severe loss of yields through lodging and shattering. Strong winds during flowering and pollination are also detrimental to yield.

The rice plant is most susceptible to flood damage during periods immediately following transplanting. It is most sensitive to water stresses during the early vegetative, reproductive and early ripening stages, Figure 10.

C. The Selection of Optimal Production Schedule

The selection of an optimal rice production scheduling for the Bicol River Basin of the Philippines will be used to illustrate the general procedure. The Bicol River Basin is located at the southern tip of the Luzon Island. The climate of the River Basin Area is generally

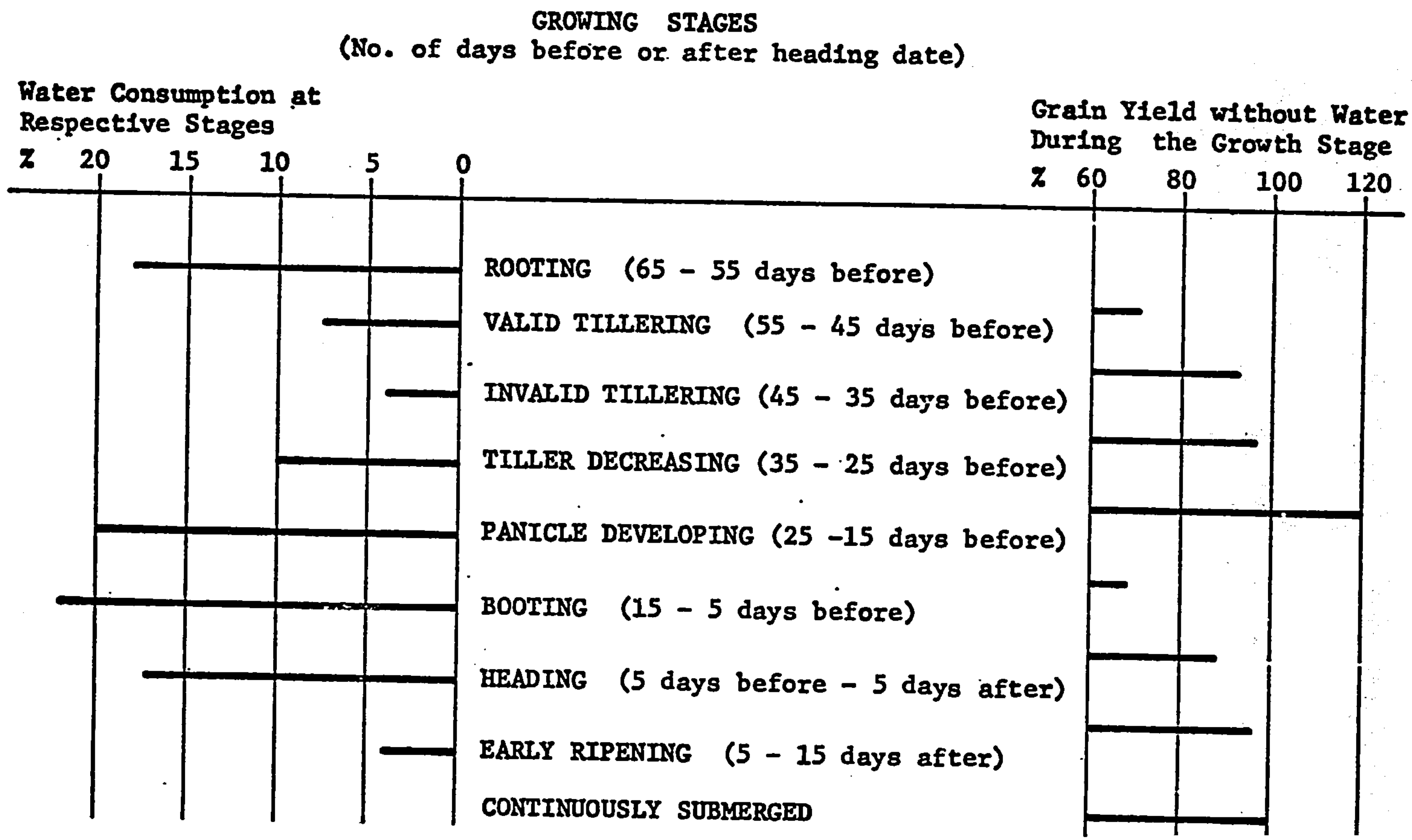


Figure 10. Rice Yield, as affected by non-irrigation at various growth stages (13).

described by Figures 11, 12, and 13. Figure 13 shows that there is an approximately 60% probability that a damaging typhoon will occur during the October through December period. Figure 11 shows the likelihood of heavy rainfall, and therefore flood damage, centers around the month of October. Therefore, it is reasonable to set the time of harvest for the second crop to be no later than the first 10 days of October, in order to avoid flood as well as other damages resulting from heavy rainfall. According to Figure 14, when harvest is scheduled to occur in early October and late September, flowering will then occur in August, which is a period of low cyclone occurrence, and therefore desirable.

When the production schedule is set for the second rice crop of the year, the decision for the first crop will come almost automatically. The problem of this production schedule is that it has totally disregarded climate factors such as net solar irradiation and temperature. As a matter of fact, an examination of Figure 12 indicates the best schedule for production should have the wet season crop harvested in late December in order to take the maximum advantage of solar irradiation. Clearly, further reconciliation between these two schedules is possible. However, more accurate and more descriptive data are required.

D. Selection of Rice Varieties

Table 2 gives a list of rice varieties recommended for the Bicol River Basin. Variety IR-28 is chosen because of its short growing season during both the dry and wet seasons. The choice of a short-season crop will allow the dry season crop, the first crop, to be transplanted in late December or early January in order to avoid typhoon and flood damages to the transplanted seedlings. Before the adoptation of this production schedule, alternative schedules must be investigated to compare their economic advantages.

Table 2. Rice Variety Growing Season

Varieties	Dry Season Days	Wet Season Days
IR-8	125	130
IR-20	120	135
IR-22	115	130
IR-24	125	125
IR-26	130	130
IR-28	105	105
IR-29	115	115
IR-30	106	109
IR-34	120	125
BPI-76	127	-
C4-63	134	128

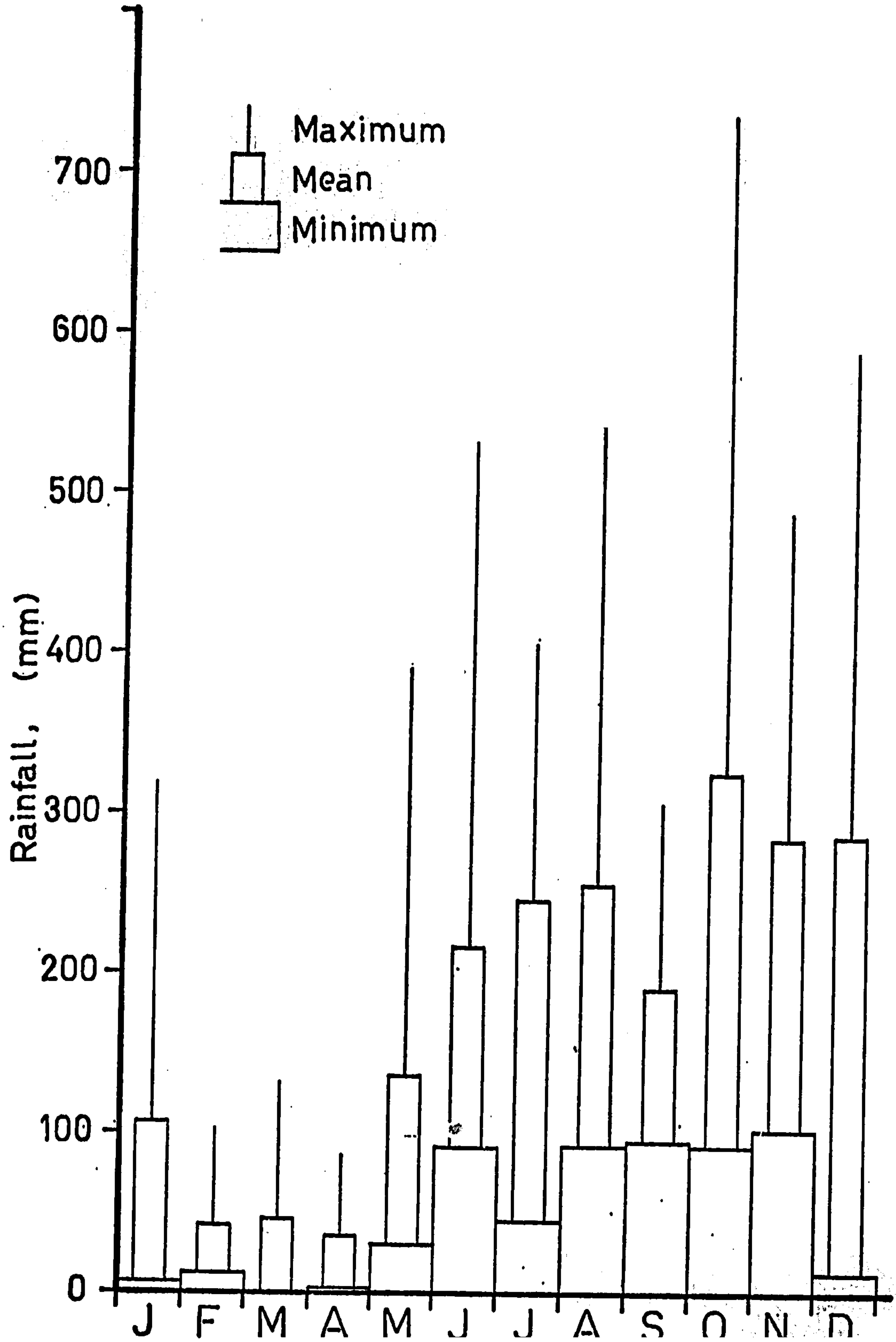


Figure 11. Rainfall, San Agustin, Pili
1966-1975

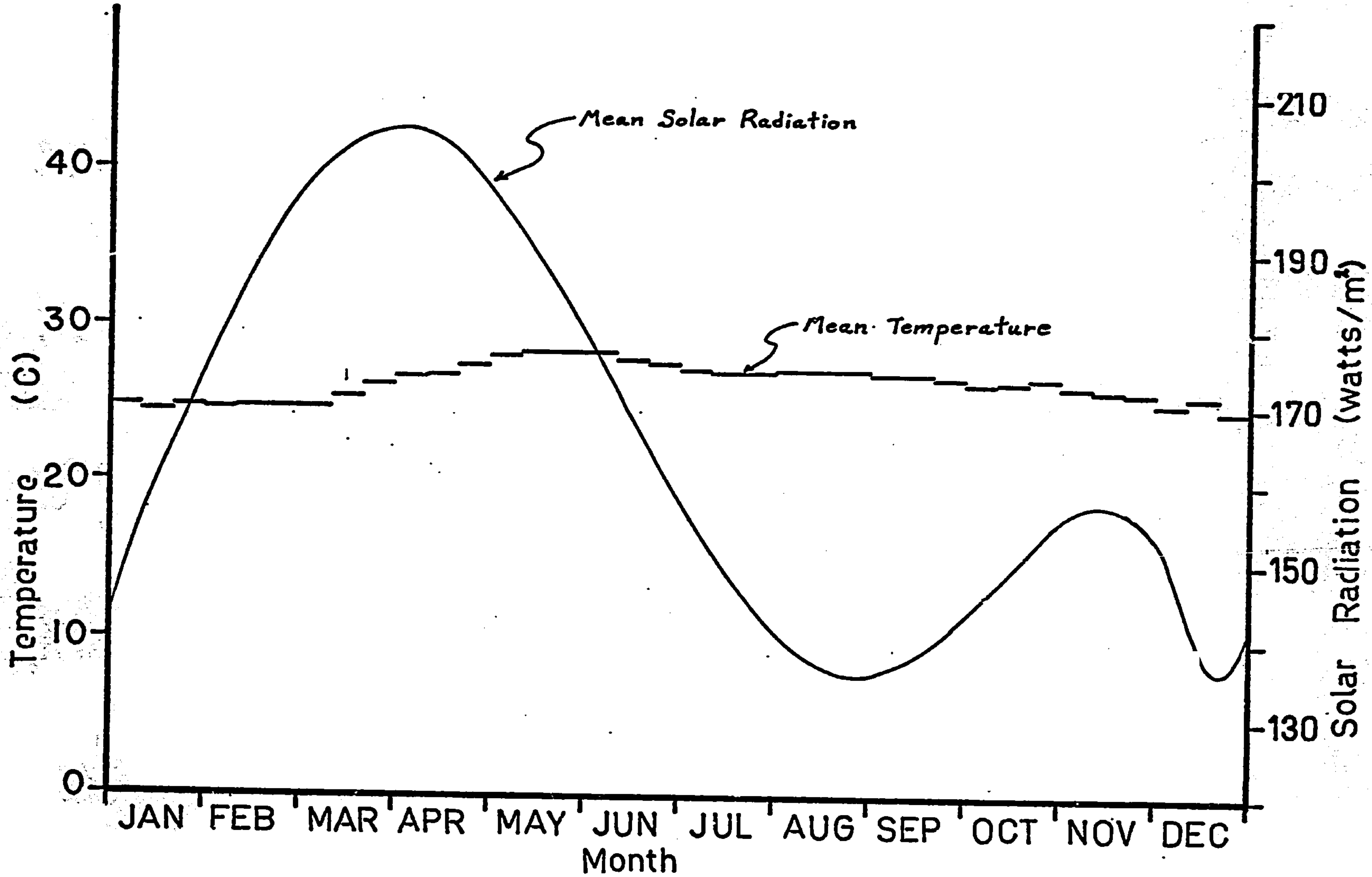


Figure 12. Temperature and radiation, San Agustin, Pili

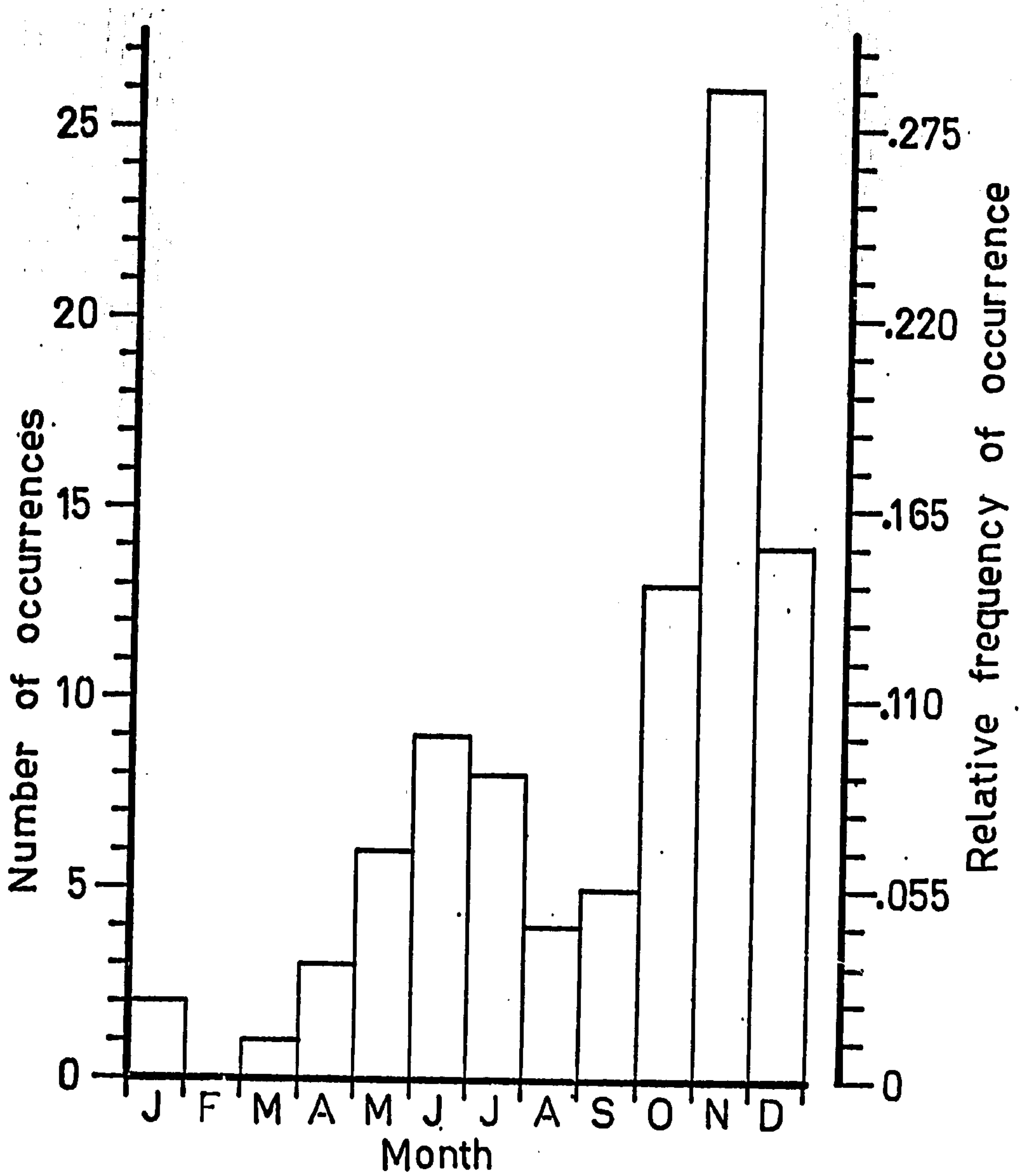


Figure 13. Tropical cyclones causing damage in the Bicol River Basin, 1884 - 1976

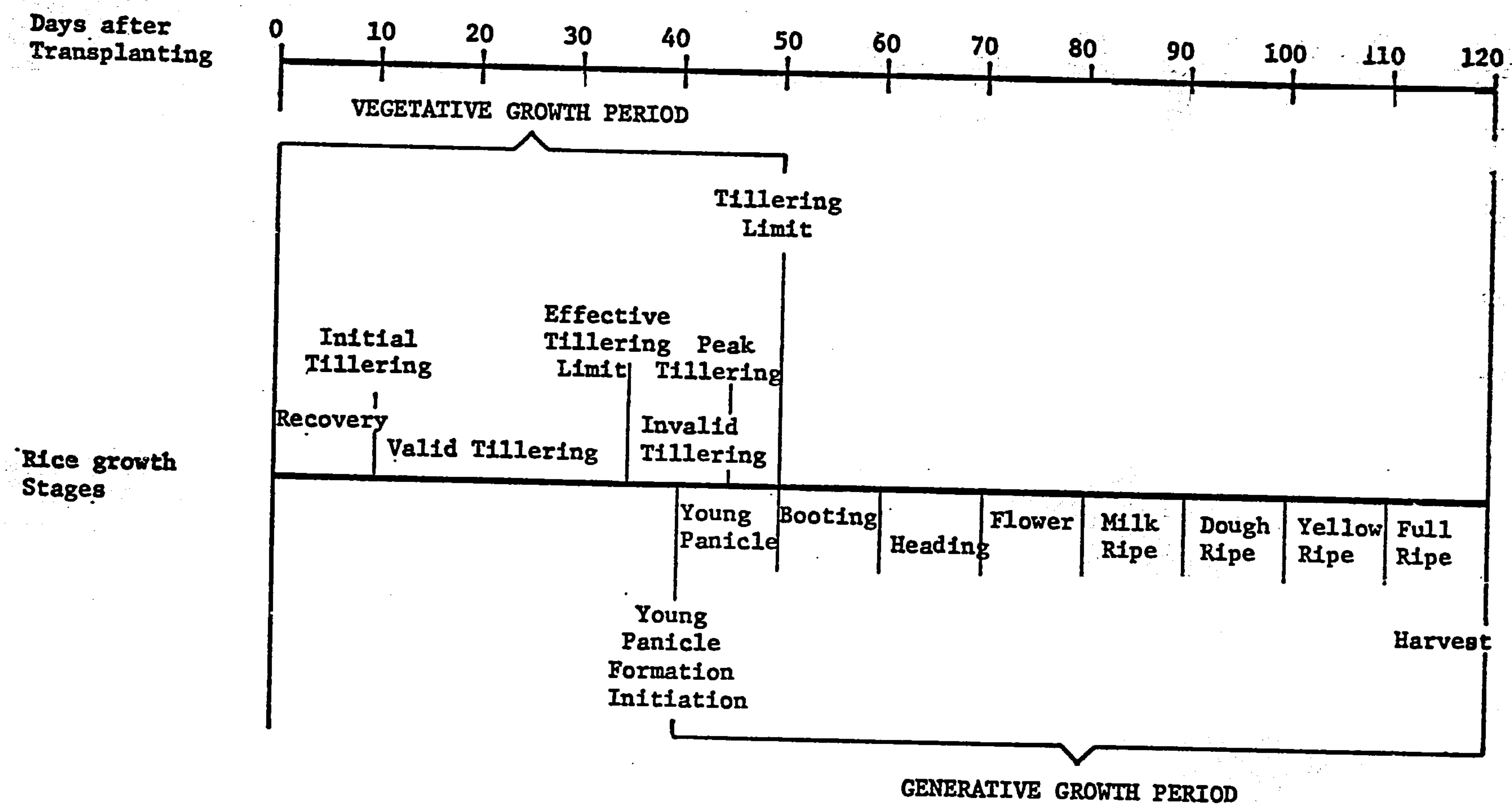


Figure 14. Stages of Rice growth (13).

Once the desired crop varieties have been selected, activity schedules for the crop seasons must be completed. The schedules will b used in the crop planning for an entire area. Daily labor demands are calculated and used with the labor constraint to determine the number o days needed to complete the highest labor demand activity for each crop This number of days determines the age spread of the area's crop. Table 3 is an example activity schedule using IR28 as the crop.

E. Crop Schedule Analysis

The basic considerations in the establishment of optimal cropping schedule are solar radiation, temperature, rainfall, water availability, labor availability, occurrence of strong wind and damaging cyclones.

The International Rice Research Institute, in its 1973 annual report, proposed that there is a high correlation between the Estimated Yield Potential (EYP) for their IR747 line, and temperature and solar radiation during the 25-day period before flowering. The IR747 line matures 96 days from sowing to harvest. It has not yet been released for general production purposes. However, it is an early maturing variety not unlike the 105-day variety which is being recommended for the Bicol River Basin. Therefore, the finding is applied to evaluate the relative goodness of alternative cropping schedule.

$$\text{Estimated Yield Potential (EYP)} = 2.065 \times (278-7.07T) \times S$$

Where T = daily mean temperature, degree C
S = daily solar radiation, watts/m^2/day

In the evaluation of alternative production schedules, adjustments must be made to account for the effect of cyclone occurrences. This can be done by estimating the probability of cyclone occurrences in the production cycle and then reduce the EYP by a factor proportional to the probability of damaging cyclone occurrence. Three alternative production schedules were studied, Figure 15, and the results are shown in Table 4.

1. Schedule 1

The basic approach in Schedule 1 was to minimize cyclone damages. The wet season harvest was scheduled to be completed before October and the 105-day variety is chosen so that flowering will fall in the month of August, another month of relatively low cyclone occurrence. The dry season crop schedule is optimized using both temperature and solar radiation. Two crops are scheduled every year, with no rice cropping activity during the months of October, November and portions of December.

2. Schedule 2

In Schedule 2, the emphasis is placed on the maximization of solar radiation received by the two crops during the period 55 days prior to harvest.

Table 3. Activity schedule for rice maturing in 105 days, IR28

Code	Activity	No Mechanization		Mechanization[a]	
		Labor[b] (M-D/Ha)	Days Allotted	Labor (M-D/Ha)	Days Allotted
01	Water Application	1	1	1	1
02	Soak & Bund Repair	3	6	3	6
03	Plow	8	6	2	2
04	Seedbed Prep.	6	3	6	4
05	1st Harrow	3	3	2	2
06	Seedbed care	1	1	1	3
07	2nd Harrow	3	3	-	-
06	Seedbed care	1	1	-	-
08	3rd Harrow	3	3	3	3
09	Strike off paddy, Pull & Bundle Seedling	6	1	6	1
10	Transplant	13	1	13	1
	Grow	0	20	0	20
11	Weed	10	3	3	3
13	Fertilize	2	2	2	2
12	1st Spray	4	4	4	4
	Grow	0	13	0	13
11	Weed	10	3	3	3
	Grow	0	23	0	23
14	2nd Spray	4	4	4	4
	Grow	0	20	0	20
15	Harvest[c]	15	1	15	1
16	Thresh[c]	25	2	1.5	1

a/ The use of (5-7 hp) hand tractors, 7 hp multicrop axial flow thresher and rotary weeder.

b/ 1. IRRI, Rice Production Manual, 1970.
2. Johnson, Loyd, Power Requirements in Rice Production, IRRI, 1963.
3. IRRI design information for small hand tractors and threshers.

c/ Assumed yield of 80 cav/ha (4 tons/ha).

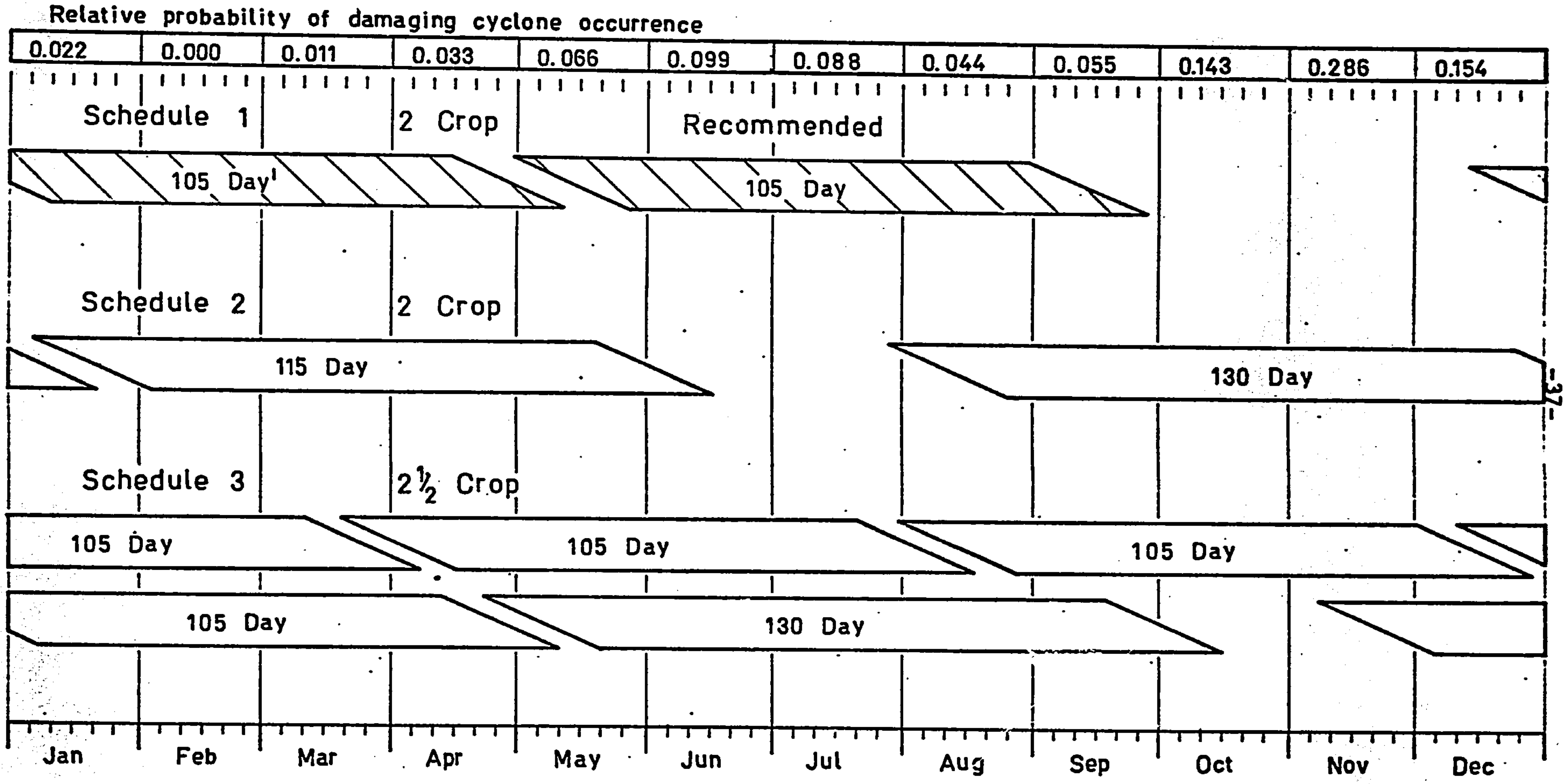


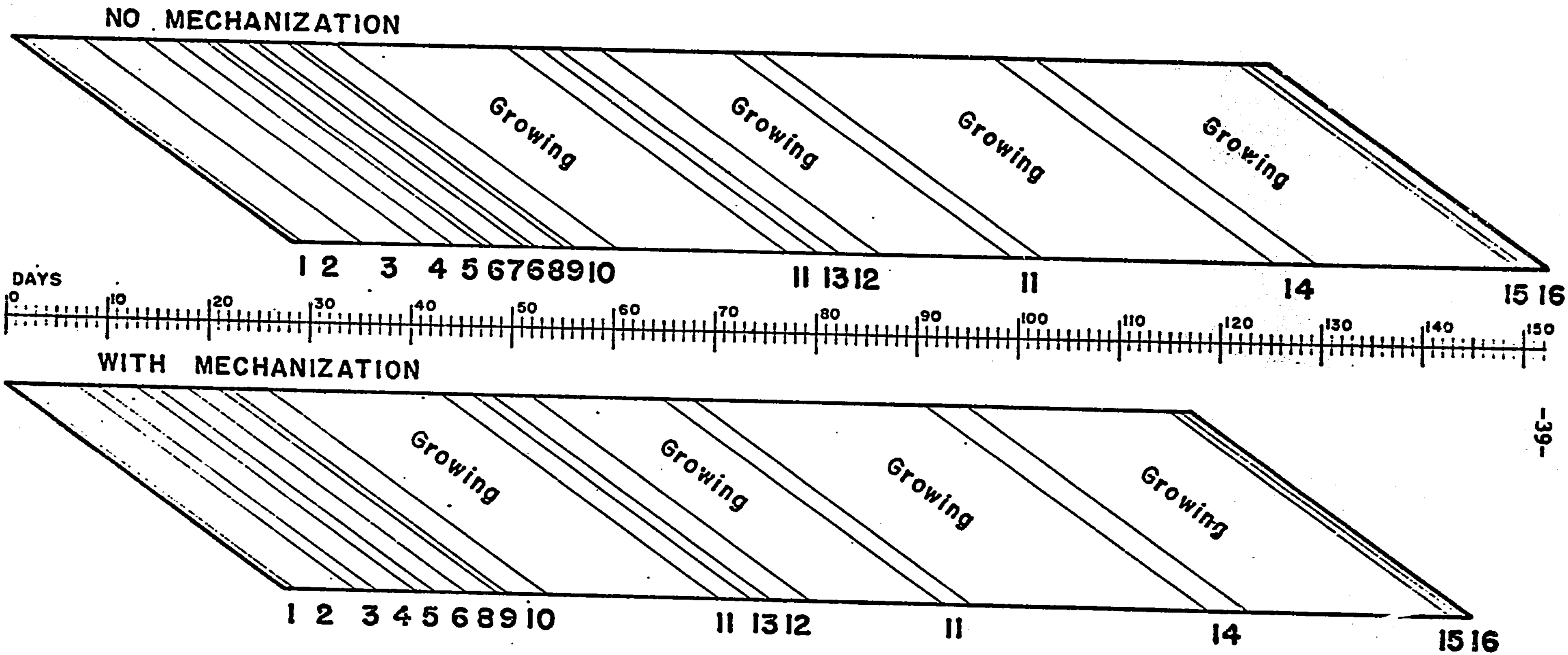
Figure 15. Alternative rice cropping schedules

[1] Length of growing season, sowing to maturity.

Table 4. Analysis of alternative rice crop production schedules for the Bicol River Basin Region

Crop Season Periods Schedules	55 Days before harvest				Probability of cyclone damage	Adjusted EYP to include anticipated cyclone damage
	Date	Mean Temperature °C (T)	Mean Solar Radiation W/m^2-day (S)	Estimated Yield[1] Potential (EYP)	Season (PS)	EYP (1-PS)
Schedule 1						
27 Dec. – 28 Apr.	2 Mar.	24.8	193.63	41,050.6	0.099	36,968.6
14 May – 13 Sep.	18 Jul.	27.2	153.40	27,146.0	0.352	17,590.6
						54,577.2
				Adjusted EYP for two years =		109,154.4
Schedule 2						
21 Jan. – 2 Jun.	6 Apr.	26.8	205.28	37,526.0	0.182	30,696.3
12 Aug. – 7 Jan.	11 Nov.	26.0	167.41	32,558.6	0.693	9,995.5
						40,691.8
				Adjusted EYP for two years =		81,383.6
Schedule 3						
3 Apr. – 3 Aug.	7 Jun.	28.3	180.19	28,992.7	0.314	19,889.0
14 Aug. – 14 Dec.	18 Oct.	26.6	155.82	28,939.9	0.682	9,202.9
25 Dec. – 26 Apr.	24 Feb.	24.8	190.24	40,331.7	0.165	33,677.0
7 May – 1 Oct.	5 Aug.	27.3	144.43	25,347.4	0.435	14,321.3
22 Nov. – 24 Mar.	26 Jan.	24.8	164.13	34,795.8	0.341	22,930.4
				Adjusted EYP for two years =		100,020.6

[1]IRRI, Annual Report 1973, pp. 48-50; EYP = (278 – 7.07T)*2.065S where 2.065 converts w/m^2-day to cal/cm^2-day.



1 INITIAL WATER APPLICATION
2 LAND SOAKING & BUND REPAIR
3 PLOW
4 SEEDBED PREPARATION
5 FIRST HARROW
6 SEEDBED CARE
7 SECOND HARROW
8 THIRD HARROW
9 STRIKE OFF, PULL & BUNDLE SEEDLINGS
10 TRANSPLANTING
11 WEEDING
12 FIRST SPRAYING
13 FERTILIZE
14 SECOND SPRAYING
15 HARVEST
16 THRESHING

Figure 16. Rice production activities for IR-28, 105 days from seeding to maturity.

3. Schedule 3

By using a 105-day variety, 2½ rice crops can be scheduled per year, or 5 crops can be produced in 2 years. There is a widely held belief that a 2½ cropping schedule would diffuse the risk of cyclone damage and, because of its higher intensity in land utilization, will out-produce the 2-crop per year schedule.

Despite the higher land utilization intensity of Schedule 3, the adjusted EYP of Schedule 1 is approximately 10% better than Schedule 3 which has 2½ crops per annum. This suggests the strong influence of cyclone-caused damages on rice production in the Bicol River Basin area.

Because of the procedure used in calculating the adjusted EYP, there can be no assurance that the results are accurate to the 10% range. But even if the difference in adjusted EYP between Schedules 1 and 3 is disregarded, the total production cost for Schedule 3 would be close to 125% of the production cost of Schedule 1. Furthermore, the two crops per year, using 105-day varieties, allows time for irrigation and drainage systems maintenance and farmer education.

Therefore, Schedule 1 is adopted in the overall production system design.

CHAPTER VI. DRAINAGE SYSTEM DESIGN

A. Introduction

Excessive water in the paddy field for a prolonged period will significantly reduce the yield. Therefore, adequate waterways should be provided to drain excess water from a storm of certain return period, within a set time period. The commonly used procedure for highway and urban drainage designs uses peak discharge as their design capacity. This procedure is not suitable for paddy rice production system drainage design, since rice plants at different growth stages can endure limited periods of submergence without substantial damage. The peak discharge design will simply lead to oversizing of drainage facilities. An innovative procedure based on the allowable rice plant submergence at different growth stages is needed to economically and effectively design the drainage systems for paddy rice fields.

B. The Critical Period

Rice plants at different growth stages can endure different periods of submergence for different depths. The most critical depth, submergence period, and growth stage will govern the drainage design. Experiments (8) show the critical period for drainage in rice production is the 2-month period immediately following transplanting. The maximum allowable submergence - stress at that stage is 20 cm for a period of 3 days. This critical period is determined by the crop schedule and seasonal rainfall distribution. Design capacities for the drainage system are based on the amount of rainfall, for a given design return period during the critical period, to be discharged to maintain the water level in the rice paddy at 20 cm for a period lasting 3 days.

C. Rainfall Frequency Analysis

There are two customary ways to select the rainfall data for frequency analysis: the partial series which includes all rainfalls above a selected base value of rainfall and the annual series which only includes the maximum rainfall each year. These two series give very nearly the same return periods for the larger rainfalls. However, the annual series will show smaller rainfalls for the shorter return periods since the second or lower order rainfalls in some years will exceed other years' maximum rainfalls which are included in the series. Tropical regions usually receive more than one typhoon in a year. Sometimes there may even be several typhoons in one month. To better estimate rainfalls for the shorter return periods, the partial series is chosen over the annual series. Data are selected from precipitation records within the critical period and above a base rainfall value such that the number of data points included in the series equals the number of record years. There are a number of ways to combine the daily rainfalls into several days rainfall when such combination of record are needed. All continuous combinations of combined values greater than the base value should be included in the partial series. However, any daily rainfall cannot appear more than once in these combinations, i.e., there can be no overlapping of combinations.

The most commonly used formula, the well-known California method (22), for the computation of return period is

$$T_r = \frac{n + 1}{m} \tag{11}$$

where T_r is the return period in years; n, the number of years of record; and m, the rank of the event (m = 1 for maximum and m = n for minimum event). After arranging the events in a descending order of magnitude, the return periods for all events in a partial rainfall series can be computed from equation (11). The cumulative frequency relation for this partial rainfall series can then be graphically developed by plotting computed return periods vs the magnitude of the respective events and fitting a straight line on log-log paper. This fitted line can be mathematically expressed as

$$\log Y = g + h \log(T_r) \tag{12}$$

where Y is the magnitude of precipitation in mm; g and h are the coefficients of regression. Rainfalls for various return periods will be estimated by either interpolating or extrapolating the fitted line.

Example 6.1

The observed daily rainfall of 25 years record at Buhi, C.S. during the months of December, January, February and March have been combined into 3-day rainfalls. The highest 25 nonoverlapping combinations are included in the frequency analysis and are tabulated, along with the corresponding return periods calculated by equation (11), as follows:

Order m	3-day rainfall Y (mm)	Log (Y)	Return period T_r (yr)	Log (T_r)
1	397.51	2.5994	26.00	1.4150
2	393.70	2.5952	13.00	1.1139
3	302.00	2.4800	8.67	0.9379
4	276.90	2.4423	6.50	0.8129
5	262.64	2.4194	5.20	0.7160
6	246.60	2.3920	4.33	0.6368
7	222.00	2.3463	3.71	0.5699
8	207.77	2.3176	3.25	0.5119
9	207.52	2.3171	2.89	0.4607
10	202.20	2.3058	2.60	0.4150
11	197.10	2.2947	2.36	0.3736
12	188.00	2.2742	2.17	0.3358
13	182.30	2.2608	2.00	0.3010
14	181.86	2.2598	1.86	0.2688
15	176.53	2.2468	1.73	0.2389
16	176.50	2.2467	1.63	0.2109
17	168.15	2.2257	1.53	0.1845
18	166.88	2.2224	1.44	0.1597
19	163.00	2.2122	1.37	0.1362
20	162.05	2.2097	1.30	0.1139
21	157.48	2.1972	1.24	0.0928
22	153.78	2.1869	1.18	0.0726
23	153.67	2.1866	1.13	0.0532
24	148.59	2.1720	1.08	0.0348
25	136.00	2.1335	1.04	0.0170

By linear least square technique, coefficients g and h in equation (2) are estimated to be 2.1632 and 0.3403, respectively. Therefore, the relationship between Y and T_r can be approximately represented by the equation log Y = 2.1632 + 0.3402 log(Tr). Rainfalls of various return periods can then be calculated accordingly. For example, the logarithm of 5 years rainfall will be 2.1632 + 0.3402 x log 5 or 2.4010. And the 5 years rainfall will be $10^{2.4010}$ which turns out to be 251.76 mm.

A FORTRAN computer program for the rainfall frequency analysis based on partial series and the California method has been written and is listed in Appendix D. The inputs for this program are: (1) daily rainfall data; (2) period to be analyzed; (3) number of days from which rainfalls are to be summed; (4) return periods for which the corresponding rainfalls will be estimated. The program will pick up data for the designated period, combine the daily rainfall into the total rainfall for a specific number of days, sort out rainfall data above the trial base value, arrange the sorted data in descending order of magnitude, eliminate any event that has one or more daily rainfall components in common with any higher order event, set the base value so that same number of events as number of record years are included in the analysis, and compute the return periods for all selected events by using equation (1). A plot of rainfall vs return period, for both actual points and points obtained by linear least square fitting, will be printed out on log-log scale along with the predicted rainfalls for different designated return periods.

D. Design of Drainage Systems

Properly designed and constructed ditches are required to remove excess water from the paddy field during storms. These drainage ditches usually have small tributary areas and the most widely used design equation "rational formula" (23) will provide a reasonably accurate estimate of the discharge rate.

$$Q = 2.753\ CIA \tag{13}$$

where Q = surface runoff in cubic meters per second (m^3/sec)

C = dimensionless runoff coefficient which is the ratio expressing the proportional amount of the rainfall that appears as runoff

I = rainfall intensity in centimeters per hour (cm/hr). It has to be uniformly distributed over the area at a uniform rate throughout the duration of the storm

A = total tributary area in square kilometers (km^2)

To apply this equation, the value of coefficient C is selected depending on the retention characteristics of the basin. For paddy fields and submerged areas, runoff coefficients are taken as 1 since all rainfalls there are converted into surface runoff. For other areas, the plants and soils are well saturated under the 3-day rainfall criteria discussed in Section B and the runoff coefficients are taken as 0.9. The rainfall intensity is determined according to the design return period, the critical period and the rainfall frequency analysis discussed previously. For example, if the design return period is 5 years and the magnitude of a 3-day rainfall for the 5 years return period is obtained from rainfall frequency analysis as 144 mm, then the rainfall intensity to be used in the equation will be 144 mm/3 days or 0.2 cm/hr. Finally, the area of the basin can be easily obtained by delineating the watershed on a topographic map and measuring the area with a planimeter.

Example, 6.2

It is determined that the critical period for the paddy fields at Buhi, C.S. is from December 1 to March 31. From example 6.1, the rainfall during this critical period is 251.76 mm/3 days or 0.35 cm/hr. The surface runoff from tributary areas of 1.2 km² paddy field and 0.2 km² coconut land will be estimated from equation (13) as

$$Q = I(C_1A_1 + C_2A_2) = 0.35\ (1 \times 1.2 + 0.9 \times 0.2)$$

$$= 0.483 \text{ cms}$$

Ditches of adequate size should be constructed according to the estimated Q value. The sizing of drainage ditches can be done by utilizing the basic continuity equation and the familiar Manning equation (24).

For steady and incompressible flow, the continuity equation states that the average velocity at a cross-section, for a given discharge, is inversely proportional to the cross-section area. It can be expressed mathematically as

$$Q = AV \tag{14}$$

where Q = discharge in cubic meters per second (cms)
A = cross-sectional area in square meters (m^2)
V = velocity in meters per second (mps)

The average velocity of open channel flow can be estimated by Manning equation

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{15}$$

where n = Manning roughness coefficient

R = hydraulic radius which is the ratio of flow area to wetted perimeter, in meters (m)

S = slope of the energy grade line which is often approximated by the channel slope

Design charts, for the direct solution of the Manning equation, for various-sized open channels of rectangular, trapezoidal and triangular cross-section with different Manning's n values were published by the U.S. Department of Commerce (7). The Manning equation can also be easily programmed on a computer or programmable calculator for systematic sizing of open channels. A FORTRAN program for the solution of Manning equation on irregular open channels is listed in Appendix D.

E. Design of Bund Opening

To effectively drain excess storm water from the paddy field to the drainage ditch, openings should be provided in each plot bund. The maximum water depth required during land preparation is 15 cm. Therefore, the permanent opening in the bund will be from the top to 15 cm above ground. The capacity of the bund opening is calculated by (24)

$$Q = 1.836\ Lh^{3/2} \tag{16}$$

where Q = rate of discharge in cubic meters per second (m^3/sec)
L = length of bund opening in meters (m)
h = head on the bund opening in meters (m)

Combining equations (13) and (16), the length of the required bund opening will be

$$L = 1.499\ \frac{IA}{h^{3/2}} \tag{17}$$

For a critical submergence depth of 20 cm and bund opening height of 15 cm, h will be 20 - 15 = 5 cm. Equation (17) can then be expressed as

$$L' = 1.34\ I \tag{18}$$

where L' = length of bund opening per unit contributing area, in meters per hectare (m/ha), and
I = the rainfall intensity as defined and discussed in Section D.

Example 6.3

The paddy fields specified in example 6.2 are subjected to a critical submergence depth of 20 cm. For a bund opening height of 15 cm, the required length of bund opening can be calculated from equation (18) as

$$L' = 1.34I = 1.34 \times 0.35 = 0.469\ \text{m/ha}$$

F. Peak Discharge and Drainage Area Relationship

From a practical point of view, peak discharge, Q, and the drainage area, A, can be approximately related by the following equation

$$Q = aA^b \tag{19}$$

where a and b are the coefficients of regression. These coefficients are different for different regions, different streams and peak discharges of different return periods. Equation (19) can be converted to

$$\log Q = \log a + b \log A \tag{20}$$

and it can be seen that log Q and log A are linearly related. To estimate the values of a and b, known peak discharges are plotted against their corresponding drainage areas on log-log paper and regression analysis is performed to find a line that best fits these plotted points. Coordinates of any two points on the fitted line can then be substituted for Q and A in equation (20) to establish two equations for the solution of a and b.

It is preferable to estimate coefficients a and b for each stream or even every portion of a stream. However, when there are insufficient records to do so, these two coefficients are often established on the entire river basin or region basis. Care should be taken in the selection of records for regression analysis. Those affected by tidal fluctuation, flow regulation, peculiar rainfall pattern or runoff characteristics, and other unusual topographical, hydrological or meteorological phenomena should be detected and cautiously examined. One simple criteria for the selection is to include only those records plotted reasonably close to the fitted line. In other words, only those records that will constitute a reasonably high correlation coefficient are used to establish the peak discharge and drainage area relationship. It is observed, from relationships of peak discharges for various return periods and drainage areas, that coefficient b does not vary as much as coefficient a. For a stream with insufficient records to reliably estimate both coefficients a and b, it is recommended that coefficient a be estimated from the records at that stream and all usable records from other streams be included to estimate coefficient b.

Example 6.4

There is no streamflow gaging station on the Waras River which is a tributary of the Bicol River. It is desired to estimate the 5-year flood at a reach of the Waras River with 72.73 km² drainage area. Table 5 shows the drainage areas and the 5-year floods of 15 gaging stations in the Bicol River Basin. Coefficients a and b in equation (20) can be estimated, by linear least square technique, as 14.97 and 0.5011, respectively. From equation (19)

$$Q = 14.97\ A^{0.5011}$$

'or A = 72.73, $Q = 14.97 \times 72.73^{0.5011} = 128.27$ cms.

Table 5. List of gaging stations with their drainage areas and 5-yr floods

No.	Gaging Stations	Drainage Area A, km²	5-yr Flood Q, cms
1	Agus River, Agus, Polangui, Albay	111	140
2	Cabilogan River, Bobongsuran, Ligao, Albay	164	248
3	Irraya River, Obaliw, Oas, Albay	217	324
4	San Agustin River, San Agustin, Libon, Albay	262	126
5	San Francisco River, Bobongsuran, Ligao, Albay	131	146
6	Talisay River, Alliang, Ligao, Albay	90	136
7	Ugsong River, Binanuanau, Ligao, Albay	99	30
8	Aslong River, San Isidro, Libmanan, C.S.	12	93
9	Barit River, Sto. Nino, Iriga City, C.S.	142	322
10	Bicol River, Ombao, Bula, C.S.	1630	600
11	Bicol River, Sto. Domingo, Nabua, C.S.	905	209
12	Culacling River, Del Rosario, Lupi, C.S.	64	77
13	Pawili River, San Rogue, Bula, C.S.	540	605
14	Pulantuna River, Napolidan, Lupi, C.S.	172	388
15	Yabo River, San Isidro, Naga City	20	51

Example 6.5

There is one streamflow gaging station on the Barit River. The drainage area of this river at the gaging station is 142 km^2 *and the 5-year flood is 322 cms. However, it is desired to estimate the 5-year flood at the downstream reach of the Barit River with 162 km*^{2} *drainage area. As discussed in this section, coefficient b will be estimated from all usable records in the basin. Its value had been calculated in example 6.4 and is equal to 0.5011. Substitute this value, along with the known Q and A values at the gaging station, into equation (19). Coefficient a can then be estimated and it equals* $322/142^{0.5011}$ *or 26.87. Therefore, for* $A = 162$, $Q = 26.87 \times 162^{0.5011} = 343.92$ *cms.*

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APPENDIX A

BRIEF REVIEW OF RICE IRRIGATION WATER REQUIREMENTS

Irrigation needs for paddy rice production can be classified into four categories:

(1) Seed cultivation:

Seedling production required about 1/25 of the production area. According to the Rice Production Manual (32), the best ages for transplanting wet-bed seedling in the Philippines are when they are 20 to 30 days old. In Taiwan, depending upon the varieties and weather conditions (Primarily temperature), suitable seedling ages for the first crop vary from 30 to 50 days, and from 15 to 20 days for the second crop. "Dapog" seedbed seedlings are ready for transplanting 10 to 12 days after sowing.

(2) Land soaking:

In many tropic regions, but definitely not universally practices, the paddy land is irrigated prior to the tillage operation. This operation is generally done to prevent the drying out of the heavy soil. About 120 mm of water is required for land soaking and it is generally put on 10 days before tillage.

(3) Tillage:

Tillage is generally carried out 15 days before transplanting. For land which has been presoaked, 30 mm of water is generally supplied. For others a one time supply of 120 mm to 150 mm of water is required.

(4) Growing period:

Plant water requirement during the growth period is covered elsewhere and only a rough indication is represented below.

Evapo-transpiration:	4 mm to 5 mm per day during wet season. 5 mm to 8 mm per day during dry season.
Percolation loss:	Usually between 1 mm to 3 mm per day, but up to 10 mm per day is possible when soil is very light and water table is deep.

It may be obvious, but nevertheless should be emphasized, that data given in this section are only summaries of average values. Actual field data should be collected and used in actual design.

APPENDIX B

CONVEYANCE LOSSES AND CANAL CAPACITY

A. Causes of Losses

Generally, the conveyance losses in farm ditches and canals are estimated to be a percentage of the capacity. Actually, conveyance losses are a complex variable, generally proportional to canal length and wetted perimeter. The cleanliness of canals can influence conveyance losses in the following ways (16).

1. Plant growth retards the water velocity, thereby decreasing the canal capacity and increasing evaporation losses.

2. Sedimentation restricts water flow by decreasing the available cross-sectional area.

3. Plant growth prevents the proper inspection of canal banks and provides cover for burrowing animals who ruin the banks.

4. Canal bank weeds are a source of weed seed for crop infestation.

5. The plant growth clogs measuring devices and other parts of the system, adding to the canal cleaning costs, and can cause damage to the banks due to overflow. For example, a plugged culvert or siphon can cause the canal to overflow damaging the banks.

6. Water is lost by transpiration of the canal growth.

Factors 1 and 2 cause the channel roughness to increase thereby decreasing the velocity and flow rate, Table B.1.

Example 2. Given a trapezoidal cross-section canal with the following characteristics

$Z = 1$	$b = 0.3$ m
$S = 0.001$	$d = 0.2$ m
$A = 0.1$ m^2	$P = 0.8656$ m
$R = 0.1155$	

Let Manning's n be variable.

$n = 0.0225, 0.072, 0.12$

Then using Manning's equation, the results are as follows:

$$Q = \frac{AR^{2/3}S^{1/2}}{n}$$

Table B.1. Roughness Coefficient, n, for Manning Formula

Type and Description of Canals	n Values Min.	Design	Max.
(1) Earth bottom, rubble sides Drainage ditches, large, no vegetation	0.028	0.032	0.035
(a) 2.5 hydraulic radius	0.040		0.045
(b) 2.5-4.0 hydraulic radius	0.035		0.040
(c) 4.0-5.0 hydraulic radius	0.030		0.035
(d) 5.0 hydraulic radius	0.025		0.030
(2) Small drainage ditches	0.035	0.040	0.040
(3) Stony bed, weeds on bank	0.025	0.035	0.040
(4) Straight and uniform	0.017	0.0225	0.025
(5) Winding, sluggish	0.0225	0.025	0.030
(6) Dense, uniform stands of green vegetation more than 10 inches long			
(a) Bermuda grass	0.04		0.20
(b) Kudzu	0.07		0.23
(c) Lespedeza, common	0.047		0.095
(7) Dense uniform stands of green vegetation cut to a length less than 2.5 inches			
(a) Bermuda grass	0.034		0.11
(b) Kudzu	0.045		0.16
(c) Lespedeza	0.023		0.05

Q CMS	n			
0.03336	0.0225	Clean, straight canals		
0.01042	0.072	Dense, short grass	6.5	cm
0.00625	0.12	Dense, long grass	25	cm

The flow rate decreases significantly with an increase in channel roughness.

B. Measurement of Seepage Losses[1]

1. Inflow-outflow method

The over-all loss in a canal system can be determined by inflow-outflow measurements, using current meters or available measuring structures. In a short reach of canal the quantitative accuracy of inflow-outflow measurements with existing methods is not adequate to establish the extent of the loss, or even to determine whether or not there is a loss. This is an inherent weakness of the method and a limitation that confronts anyone attempting to establish losses based on small differences involving the measurements of large volumes.

2. Ponding method

Ponding measurements are obtained by sealing off a section of canal with dikes, or water tight structures, and determining the rate at which the water elevation in the ponded section lowers with time. This is probably the most accurate method that is available for evaluating seepage losses in short reaches of canal. Ponding, however, interrupts water deliveries and, in large canals, involves considerable expense. Inaccuracies arise from the fact that seepage varies from year-to-year and during the irrigation year within any given reach of canal.

C. Calculation of Losses (25)

The two largest sources of water loss are seepage and percolation, and leakage past control structures. Seepage and percolation losses are generally expressed in terms of volume per wetted area of the channel, m^3/m^2/day. Leakage through structures is often estimated to be 5% of the farm delivery requirement, for more precise data field experimentation is necessary. Leakage through banks is estimated to be equal to seepage losses.

Example 3. It was determined that in a silty clay loam soil the farm ditch loss coefficient is 0.075 m^3/m^2/day during the wet season and 0.1 m^3/m^2/day in the dry season. Assuming average farm ditch size: b = 0.30 m and d = 0.12 m, what is the conveyance loss per 100 m, ditch side slope is 1:1.

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[1]Robert M. Hagan, et al. Irrigation of Agricultural Lands, American Society of Agronomy, Madison, 1974, p 1106.

Wetted perimeter = $b + 2\sqrt{2} \times d$

Solution:

Wetted perimeter = $0.30 + 2\sqrt{2} \times 0.12 = 0.64$ m
Wetted area/100 m = $0.64 \times 100 = 64$ m^2
Conveyance loss per 100 m of ditch per day in dry season = $64 \times 0.10 = 6.4$ $m^3/100$ m

D. Diversion Requirement (25)

Definitions:

Water requirement (WR) = Evapotranspiration (Etp) + Percolation (P)

Irrigation requirement (IR) = WR + Farm waste (FW) - Effective rainfall (ER)

Farm delivery requirement (FD) = IR + Farm ditch loss (FDL)

Diversion requirement (DR) = FD + Conveyance loss upstream of the farm ditch

Example 4. Determine the diversion requirement for 50 ha given the following data: (dry season)

Farm Ditch	5000 m	Water depth	0.12 m
Evapotranspiration	6 mm/d	Base	0.3 m
Percolation	2 mm/d		
Canal Lateral	2 km	Water depth	0.32 m
Seepage Rate	0.1 $m^3/m^2/d$	Base	0.8 m

Assumptions:

Farm Waste	20%	of water requirement
Effective Rainfall	0	
Leakage thru gates	5%	of farm delivery requirement
Leakage thru dikes	100%	of seepage loss

Computation:

<u>WR = Etp + P</u> = 6 + 2 = 8 mm/d

<u>IR = WR + FW - ER</u>

IR = 8 + (8 x 0.2) - 0 = 9.6 mm/d

Total Requirement =

IR x Area $9.6 \times 10^{-3} \times 50 \times 10^4 = 4800$ m^3/d

Farm Ditch Loss = Wetted perimeter x Length x Seepage rate

Wetted Perimeter $0.3 + 2\sqrt{2} \times 0.12 = 0.64$ m

FDL = $0.64 \times 0.1 \times 5000 = 320$ m^3/d

<u>FD = IR + FDL</u>

FD = 4800 + 320 = 5120 m^3/d

Conveyance losses:

Wetted Perimeter 0.8 + 2 $\sqrt{2}$ x 0.32 = 1.7 m

Seepage 1.7 x 0.1 x 2000 = 340 m^3/d

Leakage (gate) 0.05 x 5120 = 256 m^3/d

Leakage (dike) = <u>340 m^3/d</u>

Total 936 m^3/d

Diversion Requirement = FD + Conveyance losses

DR = 5120 + 936 = 6056 m^3/d

$$6056 \times \frac{1}{86400} = 0.0702 \ m^3/sec$$

E. Canal Characteristics

The following useful relations have been derived from the most efficient trapezoidal canal cross-section relations and the Manning equation.

Manning Equation:

$$V = \frac{R^{2/3}S^{1/2}}{n}$$

$$A = \frac{1.4144(Qn)^{0.75}}{S^{0.375}} (2\sqrt{Z^2+1}\ Z)^{0.25}$$

$$d = [-b + \sqrt{b^2+4AZ}\ ]/2Z$$

$$R = d/2$$

$$S = \frac{2.5198(Vn)^2}{d^{1.3333}}$$

where:

- A = cross-sectional area
- b = canal bottom width
- d = water depth
- R = hydraulic radius
- n = Manning canal roughness coefficient
- S = canal slope
- Q = canal flow rate
- V = velocity of flow
- Z = horizontal component of the side slope ratio when the vertical component is 1

Example 5. Find d,V. Given: $Q = 0.12\ m^3/sec$, $S = 0.001$, $n = 0.04$, $b = 0.4\ m$, $Z = 1.0$, $V = 0.2 \rightarrow 0.7\ m/s$

$$A = \frac{1.4144(0.12*0.04)^{0.75}}{(0.001)^{0.375}}(2\sqrt{1+1}-1)^{0.25} = 0.399\ m^2$$

$$V = Q/A = 0.12/0.399 = 0.301\ m/sec \qquad 0.2 < 0.301 < 0.7$$

$$d = (-0.4 + \sqrt{0.16+1.596})/2 = 0.465\ m$$

$$d = 0.465 m \qquad V = 0.301\ m/sec$$

For $b = 0.6\ m$

$$d = (-0.6 + \sqrt{0.36+1.596})/2 = 0.399\ m$$

Example 6. Find d,V,S. Given: $Q = 0.04\ m^3/sec$, $S = 0.0005$, $Z = 1.0$, $V = 0.2 \rightarrow 0.7\ m/s$, $n = 0.04$, $b = 0.4\ m$

$$A = \frac{1.4144(0.04*0.04)^{0.75}\ (2\sqrt{1+1}-1)^{0.25}}{(0.0005)^{0.375}} = 0.284\ m^2$$

$$V = Q/A = 0.04/0.284 = 0.141\ m/s \qquad 0.141 < 0.2$$ greater slope is needed.

$$A = Q/V = 0.04/0.2 = 0.20\ m^2$$

$$d = (-0.4 + \sqrt{0.16+0.80})/2 = 0.29\ m$$

$$S = \frac{2.52(0.2*0.04)^2}{(0.29)^{1.333}} = 0.00084$$

$$d = 0.29\ m \qquad V = 0.20\ m/sec \qquad S = 0.00084$$

TABLE B.2. CONVERSION FACTORS, BRITISH TO METRIC UNITS

Multiply	By	To Obtain
	LENGTH	
Inches	25.4 E*	Millimeters
Feet	30.48 E	Centimeters
Miles	1.609344 E	Kilometers
	AREA	
Square inches	6.4516 E	Square centimeters
Square feet	0.092903 E	Square meters
Acres	0.404686	Hectares
Square miles	2.58999	Square kilometers
	VOLUME	
Cubic feet	0.0283168	Cubic meters
	28.3168	Liters
Gallons (U.S.)	3.78531	Liters
Cubic yards	0.76456	Cubic meters
Acre feet	1233.5	Cubic meters
	VELOCITY	
Feet per second	30.48 E	Centimeters per second
Inches per hour	2.540 E	Centimeters per hour
	FLOW	
Cubic feet per second	0.028317	Cubic meters per second
	28.317	Liters per second
Cubic feet per minute	0.4719	Liters per second
Gallons per minute	0.06309	Liters per second
	3.7854	Liters per minute
	6.309×10^{-5}	Cubic meters per second
	SEEPAGE	
Cubic feet per square foot per day	304.8	Liters per square meter per day

*E - conversion factor is exact.

APPENDIX C

IMPROVED FORMULA, GENERAL CASE (9)

In the general case where N/S is not an integer, let R be the number of days in the last rotational period that are used, $R < S$, Figure C.1.

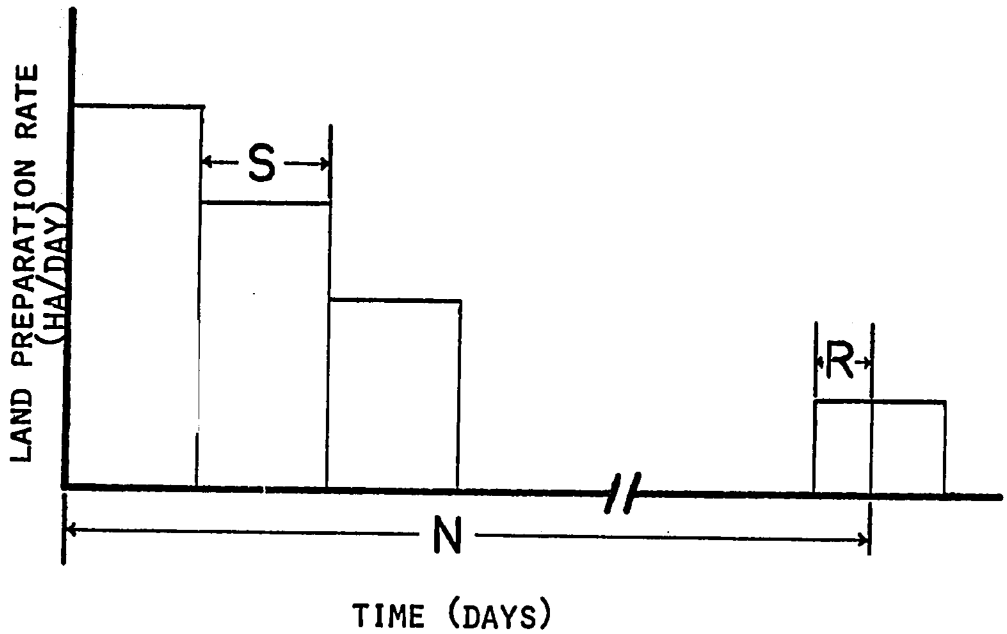


FIGURE C.1. TIME INTERVAL OF LAND PREPARATION.

where:

F = N/S decimal part only

$$R = \begin{cases} FS & F > 0 \\ S & F = 0 \end{cases}$$

$$n = \begin{cases} N/S + 1 & F > 0 \\ N/s & F = 0 \end{cases} \quad \text{Integer part only}$$

Then

$$A = \sum_{i}^{n-1} \sum_{j}^{s} a_{ij} + a_n * R = S * \sum_{i}^{n-1} a_i + a_n * R$$

And

$$Q = a_{nj}{*}D_s + \sum_{i=1}^{n} (a_{i-1,j}{*}D_t)$$

$$Q = a_{nj}{*}D_s + (A - a_{nj}{*}R)D_t$$

Substituting

$$a_{nj} = \frac{Q(D_s - S{*}D_t)^{n-1}}{D_s^{\,n}}$$

and simplifying

$$Q = \frac{AD_t}{1 - \frac{(D_s - D_t S)^{n-1}}{D_s^{\,n}} (D_s - D_t R)}$$

Considering conveyance efficiency E_c, in decimals and equal to (1-L), will give the final equation as:

$$Q = \frac{AD_t}{E_c \left[1 - \frac{(D_s - D_t S)^{n-1}}{D_s^{\,n}} (D_s - D_t R) \right]} \; m^3/day \tag{21}$$

or, with the total area in hectares (A_h)

$$Q = \frac{A_h D_t}{8.64\, E_c \left[1 - \frac{(D_s - D_t S)^{n-1}}{D_s^{\,n}} (D_s - D_t R) \right]} \; m^3/sec \tag{21a}$$

When F equals zero, N/S is an integer, equation 21 simplifies to equation 10a.

Figure C.2 shows a comparison of the equation 21 and conventional formula flow rates, Q_{pc}. Figure C.3 shows the percent difference between the conventional and improved formula flow rates, equations 1 and 21, as the number of days allowed for soaking and puddling water application changes.

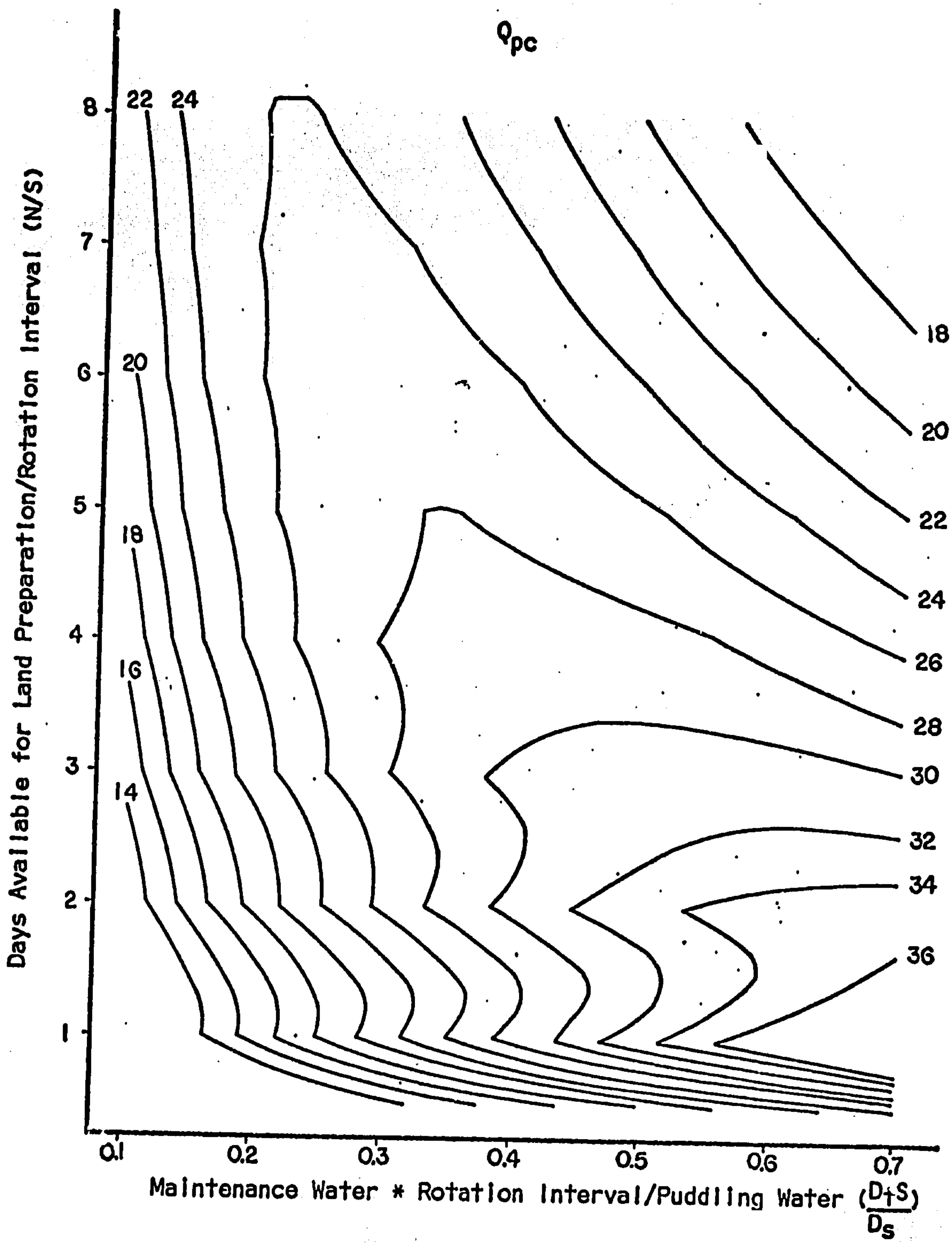


Fig. C.2. Percent difference between the conventional and improved formula flowrates.

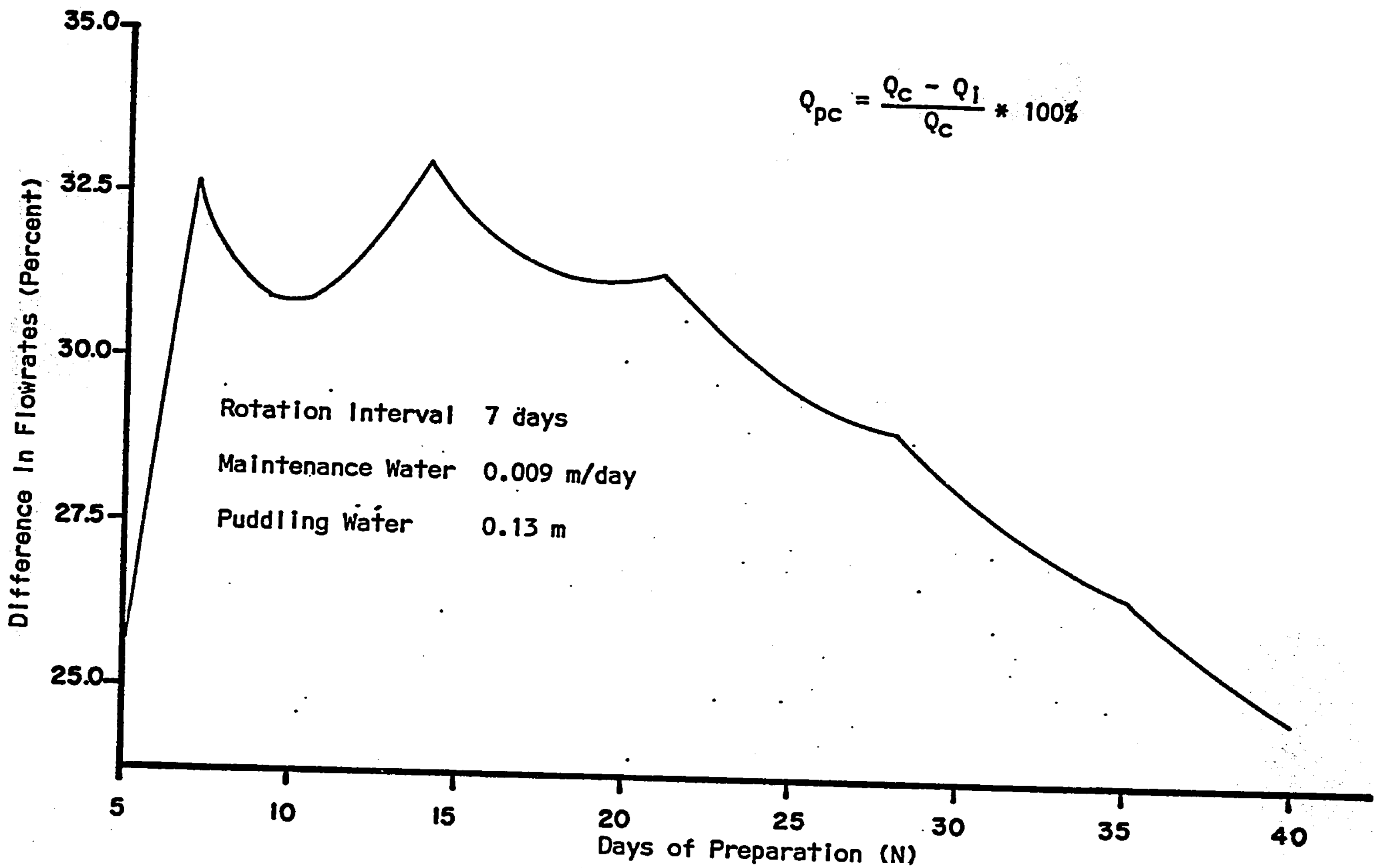


Fig. C.3. Effect of preparation time on percent differences in flowrates (conventional and improved).

APPENDIX D

RAINFALL FREQUENCY ANALYSIS

Input data description

I. Title cards: format (I2,78A1)
NT, (TITLE (I), I = 1, NT), where
NT = number of characters in TITLE
TITLE (I) = descriptive title of rainfall analysis
1. main title
2. subtitle (1)
3. subtitle (2)

II. Input control/option cards
format (16I5)
1. KOMDAY = number of days in each combination
2. IBUG = debugging flag;
IBUG = 0 implies debugging print is not envoked
IBUG = 1 implies debugging print is envoked
3. NY = number of years of data to be processed
4. IA = starting (calendar) day of the period
5. IB = ending (calendar) day of the period
6. INTOP = interpolation/extrapolation flag;
INTOP = 0 implies interpolation/extrapolation is not envoked
INTOP = 1 implies interpolation/extrapolation is envoked
7. LENGTH = number of months included in the period
8. NPERIO(I) = the months (in numeric value) included in the period

III. Months (that have data) in each year
format (16I5)
(MONTH (I), I = 1, NY) where
MONTH (I) = months of the year
NY is as defined in II

IV. Vector of points to be interpolated/extrapolated
(This card may be skipped if INTOP = 0)
format (I5, 10F5.1)
IEXT, (EXT (I), I = 1, IEXT) where
IEXT = number of points to be interpolated/extrapolated
EXT (I) = vector of points to be interpolated/extrapolated

V. Raw rainfall data
NYEAR, MT, ND, (C (I), I = 1, ND) where
NYEAR = year
MT = month
ND = number of days in MT
C (I) = raw rainfall data
format (3I2, 2X, 12F6.3/2(8X, 12F6.3 /))

```
C-----RAINFALL FREQUENCY ANALYSIS -----
      DIMENSION TR(2500), RAIN(2500), RESID(2500), YFIT(2500),
     *          SAVE(372), C(12,31), NDY(5), MONTH(30)
      DIMENSION X(100,4), Y(100,4), NPTS(4)
      DIMENSION EXT(20), EXTY(20), TITLE(80) , NPERIO(12)
      LOGICAL LOWFLO
      COMMON IBUG,IWRITE
      DATA LOWFLO / .FALSE. /
      DATA IREAD, ITAPE / 5, 8 /
      IWRITE = 6
      IPT = 0
C
C----- DEFINITION OF LOCAL VARIABLES :
C     TR(I)      = VECTOR OF RETURN PERIODS
C     RAIN(I)    = VECTOR OF RAINFALL,
C     RESID(I)   = VECTOR OF RESIDUALS AFTER LEAST SQUARE FIT,
C     YFIT(I)    = VECTOR OF FITTED VALUES OF RAINFALL,
C     SAVE(I)    = TEMPORARY STORAGE,
C     C(I,J,K)   = TEMPORARY STORAGE FOR INPUT,
C     EXT(I)     = VECTOR OF POINTS TO BE INTERPOLATED,
C     EXTY(I)    = VECTOR OF INTERPOLATED VALUES,
C     KOMDAY     = # OF DAYS IN EACH COMBINATION,
C     KASE      = DELETION OF DATA INDICATOR, DATA MAY BE DELETED IF < 50%
C                 IS AVALIABLE WITHIN A GIVEN PERIOD.
C     NY         = # OF YEARS OF DATA TO BE PROCESSED,
C     NMCN       = # OF MONTHS THAT HAVE DATA IN A CERTAIN YEAR,
C     NDAY       = # OF DAYS IN A CERTAIN YEAR,
C     LAP        = # OF NON-OVERLAPPED AND ABOVE BASE LEVEL POINTS,
C     BASE       = BASE VALUE,
C     A          = CONSTANT OF LEAST SQUARE FIT,
C     B          = COEFFICIENT OF LEAST SQUARE FIT,
C     XBAR       = MEAN OF TR THAT ARE ABOVE THE BASE LEVEL,
C     YBAR       = MEAN OF RAINFALL ABOVE THE BASE LEVEL. -----
C----- INTOP IS EXTRAPOLATION INDICATOR,
C      INTOP = 1 => EXTRAPOLATION IS REQUESTED,
C      INTOP = 0 => EXTRAPOLATION IS NOT NECESSARY. -----
C----- IBUG IS DEBUGGING FLAG,
C----- IBUG = 0 => BUGS ARE CLEARED,
C----- IBUG = 1 => DEBUGGING IS NECESSARY. -----
C
      WRITE(IWRITE,3)
      DO 2 J = 1, 3
      READ(IREAD,1) NT, ( TITLE(I),I=1,NT )
 1    FORMAT ( I2, 78A1 )
      CALL PUTHD( NT, TITLE )
 2    CONTINUE
 3    FORMAT( '1' )
      READ(IREAD,4) KOMDAY, IBUG, NY, IA, IB, INTOP, LENGTH,
     *              ( NPERIO(I),I=1,LENGTH )
      IF ( IBUG.EQ.0 ) GOTO 40
      READ(IREAD,4) ( MONTH(IY), IY=1, NY )
 4    FORMAT( 16I5 )
      WRITE(IWRITE,20)  KOMDAY, IBUG, NY, IA, IB, INTOP, LENGTH,
     *              ( NPERIO(I),I=1,LENGTH )
 20   FORMAT( ' ***** YOUR INPUT CONTROL/OPTION CARDS ARE AS FOLLOWS
     *        / 1X, 20I5 )
```

```
      WRITE(IWRITE,30) ( MONTH(IY),IY=1,NY )
 30   FORMAT( 1X, 16I5 )
 40   YADMOK = FLOAT( KOMDAY )
      BASE = SQRT( YADMOK ) * 60.
      YEAR = 1.
      KASE = LENGTH / 2
      IF ( LENGTH.EQ.1 ) KASE = 1
      IF ( INTOP .EQ. 0 ) GOTO 80
C-----READ IN VECTOR OF POINTS TO BE INTERPOLATED. -----
      READ(IREAD,50) IEXT, ( EXT(I),I=1,IEXT )
      IF ( IBUG.EQ.1 ) WRITE(IWRITE,50) IEXT, ( EXT(I),I=1,IEXT )
 50   FORMAT( I5, 10F5.1 )
C
 80   DO 500 IY = 1, NY
         NDAY = 0
         NMON = MONTH(IY)
         KOUNT = 0
C
 90      DO 190 MON = 1, NMON
            READ(ITAPE,155) NYEAR, MT, ND, ( C(MON,K),K=1,12 )
 105        FORMAT( / '  NYEAR, MT :', 2I4 / )
            IF ( IBUG.EQ.1 ) WRITE(6,105) NYEAR, MT
 155        FORMAT(    3I2, 2X, 12F6.3 )
            READ(ITAPE,170) ( C(MON,K),K=13,ND )
            DO 156 IJ = 1, LENGTH
            IF ( NPERIO(IJ) .EQ. MT ) GOTO 159
 156        CONTINUE
            GOTO 190
 159        IF ( LENGTH .EQ. 1 ) GOTO  168
            IF ( IJ .EQ. LENGTH ) GOTO 166
            IF ( IJ .EQ. 1       ) GOTO 160
 162        KSTART = 1
            KEND = ND
            GOTO 175
 160        KSTART = IA
            KEND = ND
            GOTO 175
 166        KSTART = 1
            KEND = IB
            GOTO 175
 168        KSTART = IA
            KEND = IB
 170        FORMAT( 8X, 12F6.3 )
 175        IDAY = 0
            DO 180 K = KSTART, KEND
            NDAY = NDAY + 1
            SAVE(NDAY) = C(MON,K)
 180        CONTINUE
            KOUNT = KOUNT + 1
 190     CONTINUE
         IF ( KOUNT .GE. KASE ) GOTO 200
         IF ( IBUG.EQ.1 ) WRITE(IWRITE,195)
 195     FORMAT( '   ***** INSUFFICIENT DATA, FURTHER PROCESSING',
     *           ' OF THIS PERIOD IGNORED *****' )
         GOTO 500
 200     YEAR = YEAR + 1.
```

```
      IF ( NDAY .LE. 0 ) GOTO 500
C
      IF ( LOWFLO ) GOTO 245
 205  CALL GROUP ( NDAY, KOMDAY, SAVE, BASE )
      LAP = NDAY
C
C-----  STRING UP ALL POINTS ABOVE THE BASE LEVEL IN ARRAY RAIN -----
 216  IF ( IBUG .EQ. 1 ) WRITE(IWRITE,220)LAP,  ( SAVE(I),I=1,LAP )
 220  FORMAT(   ' LAP =', I4, / ( 10F10.3) )
      IF (LAP.EQ.0) GO TO 500
      DO 240 I = 1, LAP
         IPT = IPT + 1
         RAIN(IPT) = SAVE(I)
 240  CONTINUE
      GOTO 500
 245  YMIN = 1.E+70
      DO 246 MN = 1, NDAY
         IF ( YMIN .GT. SAVE(MN) ) YMIN = SAVE(MN )
 246  CONTINUE
      LAP = 1
      IF ( IBUG .EQ. 1 ) WRITE(IWRITE,220) LAP, YMIN
 250  IPT = IPT + 1
      RAIN(IPT) = YMIN
 500  CONTINUE
C
      IF ( LOWFLO ) GOTO 611
C-----SORT STRING IN DESCENDING ORDER -----
      CALL BSORT ( IPT, RAIN )
C
 550  IF ( IBUG .EQ. 1 ) WRITE(IWRITE,552) IPT, BASE
 552  FORMAT( '  IPT =', I4, '   BASE =', F10.3 )
      IF ( IPT .GE. NY ) GOTO 555
      BASE = BASE / 2.
      GOTO 205
 555  IPT = YEAR - 1.
      BASE = RAIN(IPT)
      WRITE(IWRITE,600) IPT, BASE
 600  FORMAT(     //// ' TOTAL # OF DATA POINTS USED IN ANALYSIS =',I3,
     $       ',    BASE VALUE =', F7.3 //
     $     ' SORTED RAINFALL (ABOVE BASE LEVEL) IN DESCENDING ORDER :'/)
 610  FORMAT( 10F10.3 )
      GOTO 612
 611  CALL ASCEND ( IPT, RAIN )
      WRITE(IWRITE,614) IPT, BASE
 614  FORMAT( //// '  TOTAL # OF DATA POINTS USED IN ANALYSIS =', I3,
     *       '       BASE VALUE =', F6.3 // '  SORTED DATA (BELOW BASE ',
     *       'LEVEL) IN ASCENDING ORDER :', / )
 612  WRITE(IWRITE,610) ( RAIN(I),I=1,IPT )
      WRITE(IWRITE,615)
 615  FORMAT( 4(/), '' ROW INDEX', 12X, 'LOG(X)', 10X, 'LOG(Y)', 5X,
     $       'LOG(Y-FITTED)', 6X, 'RESIDUAL'/ )
      DO 620 I = 1, IPT
 620  TR(I) = YEAR / FLOAT( I )
C
      DO 625 I = 1, IPT
         TR(I) = ALOG10 ( TR(I) )
```

```
         RAIN(I) = ALOG10 ( RAIN(I) )
  625  CONTINUE
 C
       CALL LTSQ ( IPT, TR, RAIN, A, B, YFIT, RESID, XBAR, YBAR )
 C
       DO 628 I = 1, IPT
          RESID(I) = 10. ** RAIN(I) - 10. ** YFIT(I)
          WRITE(IWRITE,630) I, TR(I), RAIN(I), YFIT(I), RESID(I)
  630     FORMAT( I5, 11X, 3E15.6, 5X, F10.3 )
  628  CONTINUE
  655   IF ( IPT .GT. 100 ) IPT = 100
       NPTS(1) = IPT
       NPTS(2) = IPT
       NPTS(3) = 0
       DO 660 I = 1, IPT
          X(I,1) = TR(I)
          X(I,2) = TR(I)
          Y(I,1) = YFIT(I)
          Y(I,2) = RAIN(I)
          TR(I) = 10. ** TR(I)
          RAIN(I) = 10. ** RAIN(I)
          YFIT(I) = 10. ** YFIT(I)
  660  CONTINUE
       WRITE(IWRITE,663) A,B
  663  FORMAT( 5(/), ' LINEAR LEAST SQUARE MODEL :  LOG(Y) = A + B*LOG(X)
      1'/'CONSTANT A =',F12.6,5X,'COEFFICIENT B =',F12.6)
 C
       CALL PLOTZ ( X, Y,NPTS,25,'RAINFALL VS RETURN PERIOD', 13,
      $            'RAINFALL | MM', 19, 'RETURN PERIOD (YRS)' )
       WRITE(IWRITE,665)
  665  FORMAT( ' . . . . . . NOTE: '/ 15X, ' THE SYMBOL * REPRESENTS THE',
      $        ' LINEAR LEAST SQUARE FITTED RAINFALLS', / 15X,
      $       ' THE SYMBOL O REPRESENTS THE ACTUALLY RECORDED RAINFALLS')
 C
       WRITE(IWRITE,670)
  670  FORMAT( '1', 'ROW INDEX', T28, 'X', T43, 'LOG(X)', T62, 'Y', T79,
      $        'LOG(Y)', T96, 'Y-FITTED', T112,'LOG(Y-FITTED)' )
       DO 680 I = 1, IPT
  680  WRITE(IWRITE,685)I,TR(I),X(I,1),RAIN(I),Y(I,2),YFIT(I),Y(I,1)
  685  FORMAT( I5, 10X, 6E18.6 )
 C
 C-----SKIP INTERPOLATION IF NOT REQUESTED.-----
       IF ( INTOP .EQ. 0 ) GOTO 999
 C
       WRITE(IWRITE,642)
  642  FORMAT(  8(/),
      $         ' THEORITICAL RAINFALL VOLUMES OF VARIOUS RETURN PERIODS',
      $       /// ' RETURN PERIOD (YRS)', T28, 'RAINFALL (MM)' / )
       DO 650 I  = 1, IEXT
          EXTY(I) = A + B * ALOG10( EXT(I) )
          EXTY(I) = 10. ** EXTY(I)
          WRITE(6,645)    EXT(I), EXTY(I)
  645     FORMAT( 5X, F7.2, T28, F10.4 )
  650  CONTINUE
  999  STOP
       END
```

```
      SUBROUTINE GROUP ( NDAY, KOMDAY, SAVE, BASE )
C
      DIMENSION SAVE(NDAY),TEMP(372),INDX(372 ),ISELE(372 ),ID(372 )
      COMMON IBUG,IWRITE
      K1=NDAY-KOMDAY+1
      DO 600 K=1,K1
      TEMP(K)=0
      K2=K+KOMDAY-1
      DO 200 I=K,K2
200   TEMP(K) = TEMP(K) + SAVE(I)
 600   CONTINUE
      IF (IBUG.EQ.1) WRITE (IWRITE,310) K1,(TEMP(I),I=1,K1)
 310  FORMAT (' KD =', I4 / (12F10.3 ) )
      K=1
      DO 150 I=1,K1
      IF (TEMP(I).LT.BASE) GO TO 150
      SAVE(K)=TEMP(I)
      INDX(K)=I
      K=K+1
 150   CONTINUE
      K=K-1
      NDAY = 0
      IF (K.EQ.0) GOTO 98
      DO 400 I=1,K
 400   ISELE(I)=0
      IQP=1
      DO 350 I=1,K
      COMP=0
      DO 250 J=1,K
      IF (ISELE(J).EQ.1) GO TO 250
      IF (SAVE(J).LT.COMP) GO TO 250
      COMP=SAVE(J)
      JJ=J
 250   CONTINUE
      ID(IQP)=INDX(JJ)
      ISELE(JJ)=1
      IF (KOMDAY.EQ.1) GO TO 120
      IF (IQP.EQ.1) GO TO 120
      J1=IQP-1
      DO 500 J=1,J1
      IF (IABS(ID(IQP)-ID(J)).LT.KOMDAY) GO TO 350
500   CONTINUE
120   TEMP(IQP)=COMP
      IQP=IQP+1
350   CONTINUE
      NDAY = IQP-1
98    IF (NDAY.EQ.0) GO TO 99
      DO 300 I=1,NDAY
300   SAVE(I) = TEMP(I)
99    RETURN
      END
```

```
      SUBROUTINE LTSQR ( N,X,Y,A,B,YFIT,RESID, XBAR, YBAR )
C----- LINEAR LEAST SQUARE -- Y = A + B*X -----
C
C------ INPUT: N = TOTAL # OF DATA POINTS.
C              X(I) = ARRAY OF X VALUES (INDEPENDENT VARIABLE ),
C              Y(I) = ARRAY OF TRUE Y VALUES (DEPENDENT VARIABLE ),
C              FOR I = 1, 2, 3, . . . N.
C       OUTPUT: A = CONSTANT OF REGRESSION LINE.
C              B = COEFFICIENT.
C              YFIT(I) = ARRAY OF FITTED VALUES BY LEAST SQUARE FIT.
C              RESID(I) = ARRAY OF RESIDUALS.
      DIMENSION X(N), Y(N), YFIT(N), RESID(N)
      SUMX = 0.
      SUMY = 0.
      DO 10 I = 1, N
         SUMX = X(I) + SUMX
         SUMY = Y(I) + SUMY
 10   CONTINUE
      XBAR = SUMX / N
      YBAR = SUMY / N
      SXBYB = 0.
      SXB2 = 0.
      SYB2 = 0.
      DO 30 I = 1, N
         SXB = ( X(I) - XBAR )
         SXB2 = SXB * SXB + SXB2
         SYB = Y(I) - YBAR
         SYB2 = SYB * SYB + SYB2
         SXBYB = ( SXB * SYB ) + SXBYB
 30   CONTINUE
      B = SXBYB / SXB2
      A = YBAR - B * XBAR
      DO 40 I = 1, N
         YFIT(I) = A + B * X(I)
         RESID(I) = Y(I) - YFIT(I)
 40   CONTINUE
      RETURN
      END


      SUBROUTINE BSORTA ( N, RAIN )
C-----SUBROUTINE BSORT SORTS A GIVEN LIST OF ELEMENTS OF SIZE N
C----- IN  ASCENDING ORDER, AND RETURNS THE SORTED LIST.
      DIMENSION RAIN(N)
 30   NLESS1 = N - 1
      DO 50 I = 1, NLESS1
         NI = N - I
         DO 40 J = 1, NI
            JPLUS1 = J + 1
            IF ( RAIN(J) .LT. RAIN(JPLUS1) ) GOTO 40
            BUFFER = RAIN(J)
            RAIN(J) = RAIN(JPLUS1)
            RAIN(JPLUS1) = BUFFER
 40      CONTINUE
 50   CONTINUE
      RETURN
      END
```

Output from the program

I. Title and subtitles

II. Debugging prints (will be printed only when IBUG = 0)
1. All control/option cards
2. All points of KOMDAY combinations within the period of the analysis
3. All points above the base level

III. Total number of points used in the analysis, and the base value

IV. Sorted rainfall (above base value) in decending order

V. Table of the actual and predicted rainfall, and residuals, after the Least Square Fit

VI. The computed constant and coefficient of the Least Square Model

VII. A graphic display of the Least Square fitted line and the actual data points.

VIII. A conversion table of natural and logarithmic values of the actual and predicted rainfall

IX. Theoretical rainfall volumes of various return periods

3-DAY RAINFALL FREQUENCY ANALYSIS

BUHI, 1950-69,1971-75, CRITICAL PERIOD (DEC 1 - MAR 31)

PARTIAL SERIES

TOTAL # OF DATA POINTS USED IN ANALYSIS = 25, BASE VALUE =136.000

SORTED RAINFALL (ABOVE BASE LEVEL) IN DESCENDING ORDER :

397.510	393.700	302.005	276.900	262.636	246.600	221.996	207.772	207.518	202.200
197.100	188.000	182.300	181.864	176.530	176.500	168.148	166.878	163.000	162.052
157.480	153.780	153.670	148.590	136.000					

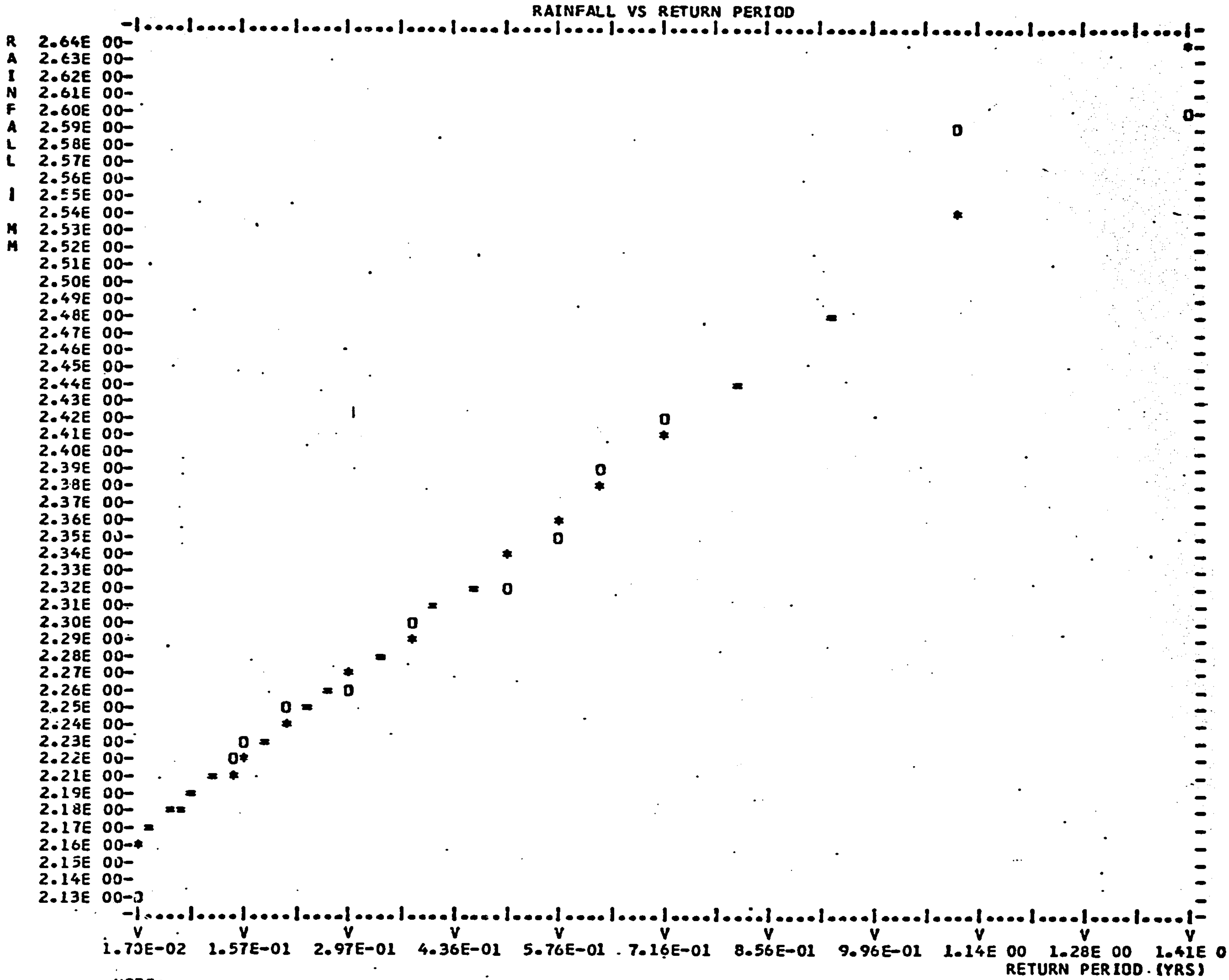
ROW INDEX	LOG(X)	LOG(Y)	LOG(Y-FITTED)	RESIDUAL
1	0.141497E 01	0.259935E 01	0.264460E 01	-43.657
2	0.111394E 01	0.259517E 01	0.254218E 01	45.221
3	0.937852E 00	0.248001E 01	0.248226E 01	-1.567
4	0.812913E 00	0.244232E 01	0.243975E 01	1.635
5	0.716003E 00	0.241935E 01	0.240678E 01	7.497
6	0.636822E 00	0.239199E 01	0.237984E 01	6.807
7	0.569875E 00	0.234634E 01	0.235706E 01	-5.543
8	0.511883E 00	0.231759E 01	0.233732E 01	-9.661
9	0.460730E 00	0.231706E 01	0.231992E 01	-1.372
10	0.414973E 00	0.230578E 01	0.230435E 01	0.665
11	0.373581E 00	0.229469E 01	0.229027E 01	1.996
12	0.335792E 00	0.227416E 01	0.227741E 01	-1.413
13	0.301030E 00	0.226079E 01	0.226558E 01	-2.024
14	0.268845E 00	0.225975E 01	0.225463E 01	2.130
15	0.238882E 00	0.224682E 01	0.224443E 01	0.967
16	0.210853E 00	0.224674E 01	0.223490E 01	4.749
17	0.184524E 00	0.222569E 01	0.222594E 01	-0.096
18	0.159701E 00	0.222240E 01	0.221749E 01	1.874
19	0.136220E 00	0.221219E 01	0.220950E 01	1.004
20	0.113943E 00	0.220965E 01	0.220192E 01	2.859
21	0.927537E-01	0.219723E 01	0.219471E 01	0.908
22	0.725506E-01	0.218690E 01	0.218784E 01	-0.333
23	0.532452E-01	0.218659E 01	0.218127E 01	1.870
24	0.347620E-01	0.217199E 01	0.217498E 01	-1.027
25	0.170333E-01	0.213354E 01	0.216895E 01	-11.554

LINEAR LEAST SQUARE MODEL : LOG(Y) = A + B*LOG(X)

CONSTANT A = 2.1632

COEFFICIENT [illegible] = 0.3603

ONE UNIT ON THE X AXIS = 1.40E-01 ONE UNIT ON THE Y AXIS = 1.02E-02
= INDICATES THAT MORE THAN ONE POINT IS PLOTTED IN THAT SPACE



NOTE:
THE SYMBOL * REPRESENTS THE LINEAR LEAST SQUARE FITTED RAINFALLS
THE SYMBOL O REPRESENTS THE ACTUALLY RECORDED RAINFALLS

ROW INDEX	X	LOG(X)	Y	LOG(Y)	Y-FITTED	LOG(Y-FITTED)
1	0.260000E 02	0.141497E 01	0.397509E 03	0.259935E 01	0.441166E 03	0.264460E 01
2	0.130000E 02	0.111394E 01	0.393699E 03	0.259517E 01	0.348478E 03	0.254218E 01
3	0.866666E 01	0.937852E 00	0.302004E 03	0.248001E 01	0.303571E 03	0.248226E 01
4	0.649999E 01	0.812913E 00	0.276899E 03	0.244232E 01	0.275264E 03	0.243975E 01
5	0.519999E 01	0.716003E 00	0.262635E 03	0.241935E 01	0.255138E 03	0.240678E 01
6	0.433333E 01	0.636822E 00	0.245599E 03	0.239199E 01	0.239792E 03	0.237984E 01
7	0.371428E 01	0.569875E 00	0.221996E 03	0.234634E 01	0.227539E 03	0.235706E 01
8	0.325000E 01	0.511883E 00	0.207772E 03	0.231759E 01	0.217432E 03	0.233732E 01
9	0.288888E 01	0.460730E 00	0.207518E 03	0.231706E 01	0.208890E 03	0.231992E 01
10	0.260000E 01	0.414973E 00	0.202200E 03	0.230578E 01	0.201535E 03	0.230435E 01
11	0.236364E 01	0.373581E 00	0.197099E 03	0.229469E 01	0.195104E 03	0.229027E 01
12	0.216667E 01	0.335792E 00	0.188000E 03	0.227416E 01	0.189412E 03	0.227741E 01
13	0.200000E 01	0.301030E 00	0.182299E 03	0.226079E 01	0.184323E 03	0.226558E 01
14	0.185714E 01	0.268845E 00	0.181864E 03	0.225975E 01	0.179733E 03	0.225463E 01
15	0.173333E 01	0.238882E 00	0.176530E 03	0.224682E 01	0.175563E 03	0.224443E 01
16	0.162500E 01	0.210853E 00	0.176499E 03	0.224674E 01	0.171750E 03	0.223490E 01
17	0.152941E 01	0.184524E 00	0.168148E 03	0.222569E 01	0.168243E 03	0.222594E 01
18	0.144444E 01	0.159701E 00	0.166878E 03	0.222240E 01	0.165003E 03	0.221749E 01
19	0.136842E 01	0.136220E 00	0.162999E 03	0.221219E 01	0.161995E 03	0.220950E 01
20	0.130000E 01	0.113943E 00	0.162052E 03	0.220965E 01	0.159192E 03	0.220192E 01
21	0.123809E 01	0.927537E-01	0.157480E 03	0.219723E 01	0.156572E 03	0.219471E 01
22	0.118182E 01	0.725506E-01	0.153779E 03	0.218690E 01	0.154113E 03	0.218784E 01
23	0.113043E 01	0.532452E-01	0.153670E 03	0.218659E 01	0.151799E 03	0.218127E 01
24	0.108333E 01	0.347620E-01	0.148590E 03	0.217199E 01	0.149617E 03	0.217498E 01
25	0.104000E 01	0.170333E-01	0.136000E 03	0.213354E 01	0.147553E 03	0.216895E 01

THEORITICAL RAINFALL VOLUMES OF VARIOUS RETURN PERIODS

RETURN PERIOD (YRS)	RAINFALL (MM)
5.00	251.7563
7.00	282.2930
10.00	318.7180
13.00	348.4783
25.00	435.3179

PROGRAM NAME: Manning Equation

Inputs:

FN: Manning roughness coefficient.

S: Slope of the channel bottom.

M: Number of segments on one side of the cross section. (Draw center line through the lowest point in the cross section.) If the number of segments on left and right sides of the center line are different, use the larger number as m.

N: An index for the input cards on FN, S and M. N equals 1 for the last card. N equals 0 for all other cards.

DL(I): Vertical projection of segment I on the left side of the center line. (I = 1 for the segment on the immediate left of the center line. I = 2 for the segment on the immediate left of the first segment and so on.) DL(I) will be positive when left side of segment I is higher than the right side of segment I. DL(I) will be negative otherwise.

BL(I): Horizontal projection (in meters) of segment I on the left side of center line.

DR(I): Vertical projection (in meters) of segment I on the right side of center line. DR(I) will be positive when right side of segment I is higher than the left side of segment I. DR(I) will be negative otherwise.

BR(I): Horizontal projection (in meters) of segment I on the right side of center line.

ID: An index to identify the objective of the program. ID = 1 if the objective is to calculate the velocity and capacity of a given cross section at a given depth. ID = 0 if the objective is to calculate the depth and velocity of a given discharge.

D: Design depth in meters when ID = 1. Trial depth in meters when ID = 0.

Q: Any value when ID = 1. Design discharge in cubic meters per second when ID = 0.

Outputs:

Depth: Design depth (in meters) for the capacity calculation.

Trial depth: Estimated depth (in meters of a given discharge based on trial and error procedure.

V: Velocity in meters per second.

Q: Discharge in cubic meters per second.

```
C-----MANNING EQUATION-----
      DIMENSION DL(100),BL(100),DR(100),BR(100)
      COMMON SA,P,M
 3    READ (5,101) FN,S,M,N
 101  FORMAT (2F1.4,2I4)
      READ (5,102) (DL(I),BL(I),DR(I),BR(I),I=1,M)
 102  FORMAT (7X,4F10.4)
      READ (5,103) ID,D,Q
 103  FORMAT (I2,2F10.4)
      SDL=0.
      SDR=0.
      DO 15 I=1,M
      SDL=SDL+DL(I)
 15   SDR=SDR+DR(I)
      IF (SDR.LT.SDL) SDL=SDR
      IF (D.LE.SDL) GO TO 7
      WRITE (6,203) D
 203  FORMAT (5X,'TRAIL WATER LEVEL OF',F10.4,2X,'M IS HIGHER THAN',/6X
     1'TOP OF THE INPUT CROSS SECTION')
      D=D-1
 7    DD=1.0
 1    K1=0
 2    IF (D.LT.0) D=0.
      IK=0
      IF (D.LT.SDL) GO TO 12
      D=SDL
      IK=1
 12   SA=0.
      P=0.
      B=0.
      CALL CALA (X,B,D,DL,BL)
      B=0.
      CALL CALA (X,B,D,DR,BR)
      V1=0.
      IF (P.LT.0.0001) GO TO 11
      V1 = 1./FN*(SA/P)**(2./3.)*S**0.5
 11   Q1 = V1*SA
      IF (ID.EQ.1 ) GOTO 5
      WRITE (6,201) D,V1,Q1
 201  FORMAT (5X,'TRIAL DEPTH',F10.4,2X,'M',5X,'V = ',F10.4,2X,'MPS',5X,
     1'Q = ',F10.4,2X,'CMS')
      IF (ABS(Q-Q1).LE.0.001) GOTO 6
      K=1
      IF (IK.EQ.1.AND.Q.GT.Q1) GO TO 9
      IF (Q.LT.Q1) K=-1
      IF (K*K1.EQ.-1) GOTO 4
      K1=K
      D=D+K*DD
      GO TO 2
 4    DD=DD/10.
      D=D+K*DD
      GOTO 1
 9    WRITE (6,204) Q
 204  FORMAT (5X,'CAPACITY OF THE INPUT CROSS SECTION IS LESS THAN',F10.
     14,2X,'CMS')
      GO TO 6
 5    WRITE (6,202) D,V1,Q1
 202  FORMAT (5X,'DEPTH',F10.4,2X ,'M',5X,'V = ',F10.4,2X,'MPS',5X,'Q =
     1',F10.4,2X,'CMS')
 6    IF (N.NE.1) GOTO 3
      STOP
      END

      SUBROUTINE CALA (X,Y,Z,U,V)
      DIMENSION U(100),V(100)
      COMMON SA,P,M
      DO 10 I=1,M
      X = Y + U(I)
      C=ABS(X-Y)
      IF (IFIX(1000*(X-Z)).LE.0) GOTO 1
      SA=SA+(Z-Y)**2/C/2*V(I)
      P=P+(U(I)**2 + V(I)**2) **0.5*(Z-Y)/C
      GOTO 2
 1    SA=SA + (Z-(X+Y)/2)*V(I)
      P=P+(U(I)**2 + V(I)**2) **0.5
      Y=X
 10   CONTINUE
 2    RETURN
      END
```

Sample Problem:

Given: FN = 0.035, S = 0.000251, M = 3, N = 1
DL(1) = 1.3m., BL(1) = 2.4m., DR(1) = 0.5m., BR(1) = 8.9m.
DL(2) = 4.0m., BL(2) = 0.0m., DR(2) = 1.0m., BR(2) = 3.6m.
DL(3) = 0.0m., BL(3) = 0.0m., DR(3) = 3.8m., BR(3) = 0.0m.

1. For D = 5.3m., V = ?, Q = ?

Pintout:

```
DEPTH    5.3000 M    V =    0.9599  MPS    Q =    68.7172  CMS
```

2. For Q = 60 cms, d = ?, V = ?. Initial trial d = 7m.

Printout:

```
TRAIL WATER LEVEL OF    7.0000  M IS HIGHER THAN
    TOP OF THE INPUT CROSS SECTION
TRIAL DEPTH    5.3000  M    V =    0.9599  MPS    Q =    68.7172  CMS
TRIAL DEPTH    4.3000  M    V =    0.8726  MPS    Q =    49.4615  CMS
TRIAL DEPTH    4.4000  M    V =    0.8822  MPS    Q =    51.3247  CMS
TRIAL DEPTH    4.5000  M    V =    0.8917  MPS    Q =    53.2030  CMS
TRIAL DEPTH    4.6000  M    V =    0.9009  MPS    Q =    55.0958  CMS
TRIAL DEPTH    4.7000  M    V =    0.9099  MPS    Q =    57.0028  CMS
TRIAL DEPTH    4.8000  M    V =    0.9187  MPS    Q =    58.9234  CMS
TRIAL DEPTH    4.9000  M    V =    0.9273  MPS    Q =    60.8573  CMS
TRIAL DEPTH    4.8900  M    V =    0.9265  MPS    Q =    60.6633  CMS
TRIAL DEPTH    4.8800  M    V =    0.9256  MPS    Q =    60.4695  CMS
TRIAL DEPTH    4.8700  M    V =    0.9248  MPS    Q =    60.2757  CMS
TRIAL DEPTH    4.8600  M    V =    0.9239  MPS    Q =    60.0822  CMS
TRIAL DEPTH    4.8500  M    V =    0.9231  MPS    Q =    59.8887  CMS
TRIAL DEPTH    4.8510  M    V =    0.9232  MPS    Q =    59.9080  CMS
TRIAL DEPTH    4.8520  M    V =    0.9232  MPS    Q =    59.9274  CMS
TRIAL DEPTH    4.8530  M    V =    0.9233  MPS    Q =    59.9467  CMS
TRIAL DEPTH    4.8540  M    V =    0.9234  MPS    Q =    59.9660  CMS
TRIAL DEPTH    4.8550  M    V =    0.9235  MPS    Q =    59.9853  CMS
TRIAL DEPTH    4.8560  M    V =    0.9236  MPS    Q =    60.0047  CMS
TRIAL DEPTH    4.8559  M    V =    0.9236  MPS    Q =    60.0027  CMS
TRIAL DEPTH    4.8558  M    V =    0.9236  MPS    Q =    60.0008  CMS
```