DESIGN WORKSHOP

SMALL SCALE WATER CONTROL STRUCTURES

VOLUME 1

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PREFACE

A Transfer of Knowledge Program is included as part of the CIDA financed Small Scale Water Control Structures III (SSWCS III) Project. This Program is aimed primarily at the technical and engineering staff of the Government of Bangladesh (GOB) and Bangladesh Water Development Board (BWDB). It is intended to enhance technical skills in engineering, agriculture, economics, management, and to increase the understanding of related disciplines such as environment and fisheries. To achieve these ends, several levels of training will be undertaken including design workshops for BWDB officials.

The objective of the design workshops is to provide training to the BWDB design engineers on principles and methodologies for design of water control structures.

In order to achieve the above objective, these manuals have been prepared in which principles and step-by-step computations for design of a drainage regulator and embankment are provided.

The engineers of Bangladesh Water Development Board and Northwest Hydraulic Consultants Ltd. prepared materials for these volumes. The Canadian International Development Agency provided funds, under the SSWCS III project, for development and publication of these training manuals and for organizing the design workshop.

Any suggestion to improve the quality of contents will be greatly appreciated.

HRKhan

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CHAPTER *ONE*

INTRODUCTION TO BANGLADESH AGRICULTURE

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INTRODUCTION

Background

Most of Bangladesh is located within the floodplains of the three great rivers (the Ganges, the Brahmaputra and the Meghna), their tributaries and distributaries (Figure 1). The three rivers drain a total catchment area of about 1.72 million square kilometers in India, Nepal, China, Bhutan and Bangladesh. Only 8 per cent of the catchment area lies within Bangladesh.

From June to September each year, the warm moist air of the monsoon sweeps up the Bay of Bengal from the Indian Ocean producing some of the highest recorded rainfalls in the world over Bangladesh and the upstream catchments of the major rivers, particularly in the Indian states of Meghalaya and Assam. Between 70 and 85 per cent of the annual rainfall is concentrated in the three to four month monsoon season. In Bangladesh, mean annual rainfall increases from about 1200 mm in the west to almost 6000 mm in the extreme east. Average annual rainfall in the Himalayas and in the Meghalaya hills to the north of Bangladesh averages about 5000 mm, but reaches 10,000 mm locally. Tropical cyclones can occur in the pre- and post-monsoon seasons. These affect coastal areas and are sometimes accompanied by storm surges.

The discharges of the three main rivers are among the highest in the world. Peak discharge are of the order of 100,000 m^3/s in the Brahmaputra, 75,000m^3/s in the Ganges, 20,000m^3/s in the Meghna and 160,000m^3/s in the lower Meghna (Figure 2). Dry season discharges are only a small fraction of wet season flows.

Surface water resources are used extensively for dry season irrigation. In addition, there are groundwater resources in the country which are tapped for irrigation and potable water. Groundwater resource potential is regionally variable and depends on recharge from monsoon rainfall and floods.

Figure 1.

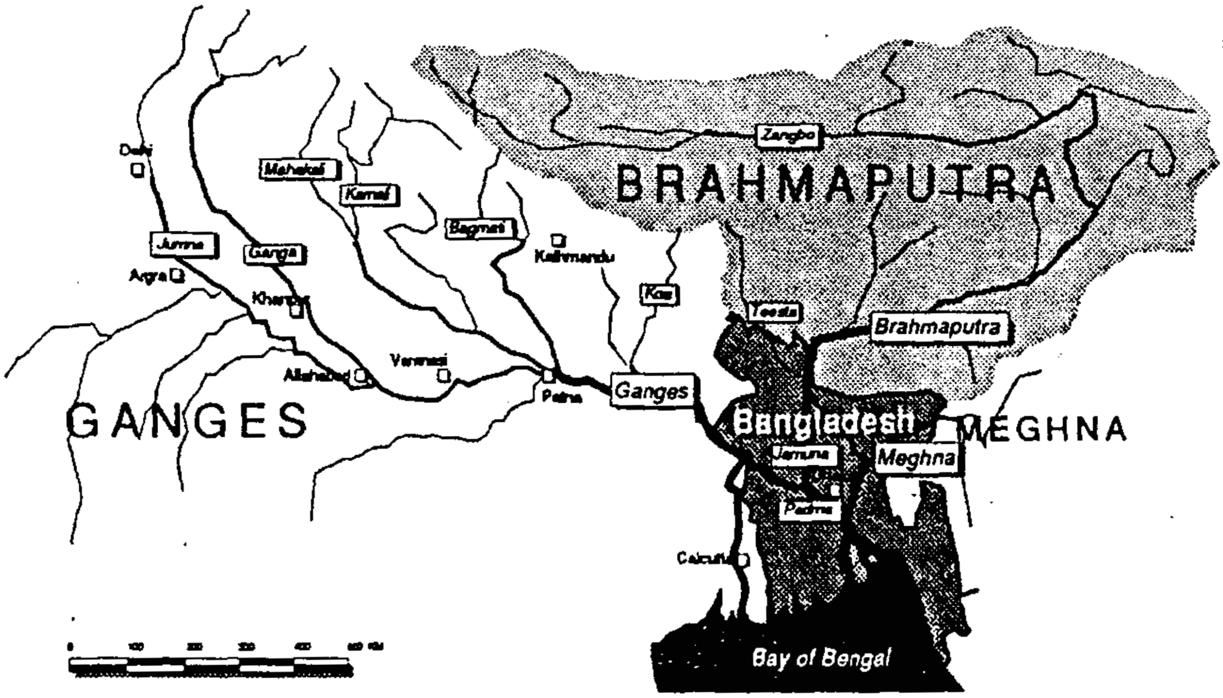
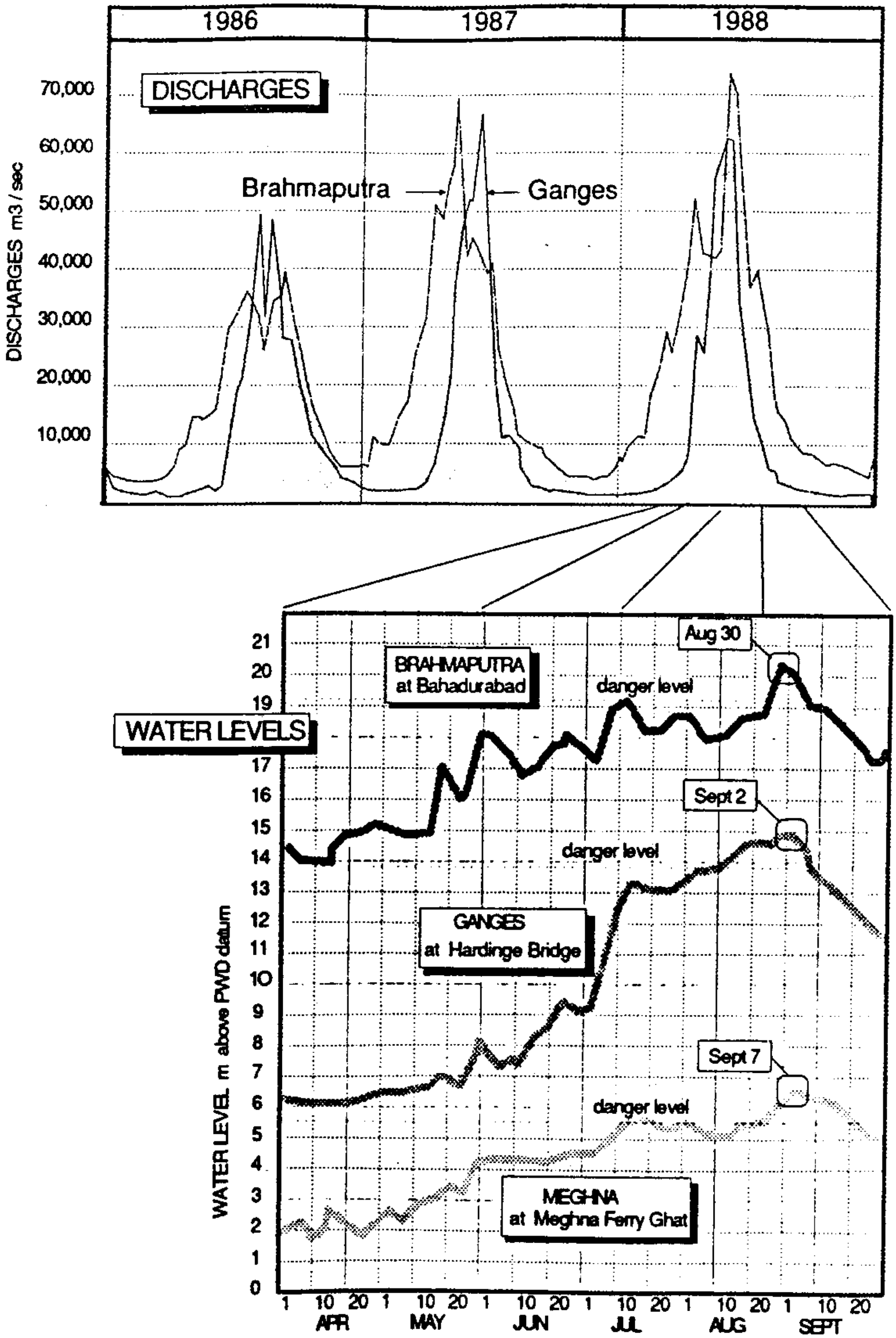


Figure 1 THE GANGES, THE BRAHMAPUTRA AND THE MEGHNA BASINS

MAIN RIVER DISCHARGES 1986-1988 AND FLOOD LEVELS 1988



A major problem that Bangladesh faces is that of too much water in the wet season (when overbank and rain water flooding acts as a limit on agricultural development, necessitating the cultivation of lower-yielding varieties and sometimes resulting in extensive damage to crops, livestock and infrastructure), and too little in the dry season (when irrigation is needed to intensify agriculture).

Population and Socio-Economic Conditions

The population of Bangladesh is about 110 million, and the population density is one of the highest in the world (770 persons per km^2, 12 persons per cultivable hectare). In some floodplain areas, the population density reaches 1500 per km^2. There are indications that population growth may be starting to decline but even so, the population is projected to rise to nearly 170 million over the next 20 years. About 85 per cent of the population live in rural areas, but this figure is expected to decline in coming decades with increasing out migration to urban areas. Migrants to towns are often people who have lost their land and houses as a result of riverbank erosion.

The economy as a whole has grown in recent years by about 4 per cent annually, but with population growth of 2.5 per cent and faster economic growth in urban areas, average living standards in rural areas have stagnated.

Over the last thirty years, there has been a marked increase in poverty in rural areas. Between the 1960s and 1980s, owing mainly to pressure of population, average farm size declined from 1.45 ha to 0.90 ha and the proportion of households that are landless rose from about 35 per cent to 45 per cent. Under-employment levels in rural areas at present are about 30-40 per cent. From the 1960s to the early 1980s there was a decline in wage rates and landless household incomes. In the mid-1980s, there were indications of a slight upturn in the incomes of the poor, possibly due to rising demand for labour in 'green revolution' agriculture, but it is probable that any such gains would have been lost following the 1987 and 1988 floods. Over the next twenty years, it is likely that the average farm size will drop to 0.6 ha and the proportion of rural households without land will increase to nearly 60 per cent.

The male labour force alone increases by almost one million a year. The participation rate of women in the labour force is increasing but the magnitude of the increase is not known. Only 3 million of the 10 million farms are large enough to support a family but the total number of rural households is approaching 15 million. In spite of demand for seasonal farm labour and evidence of a significant growth in non-agricultural employment, rural employment opportunities do not grow fast enough to absorb most of the growing labour force.

Despite the increasing poverty and landlessness in rural areas, agriculture is expected to remain the main source of new jobs in rural areas, both directly and indirectly, through linkages to input supply and processing industries.

Agriculture

Agriculture dominates the economy. It accounts for 45 per cent of GNP, 60 per cent of employment and 60 per cent of merchandise exports. The recent UNDP-funded Agriculture Sector Review foresees that agriculture will remain the mainstay of the economy for the next 20 years, despite increasing contributions to GNP from the industrial and service sectors.

Bangladesh has an area of 14.8 million hectares of which 8.2 million hectares (55%) are cultivated.

At present 8.2 million hectares, or almost all of the arable lands are cultivated, but only 1.0 million hectares are triple cropped and 3.9 million hectares are double cropped and 3.3 million hectares are single cropped 1/. The cropping intensity is 1.54. This implies the potential for cultivating additional crops particularly in the dry season (November to April) through the provision of irrigation. Irrigation presently covers about 36% of the cultivated area but could be expanded to cover a large part of the country by choosing proper technology.

Rice is the main crop grown in Bangladesh, covering about 78 per cent of the cropped area. There are three rice growing seasons a year - aus (harvested in the monsoon), aman (grown in the monsoon but harvested after) and boro (grown in the dry season). Aman is the most important (44 per cent of population), followed by boro (38 per cent) and aus (18 per cent). The other main crops are the export crop, Jute (5 per cent of the cropped area), and wheat (4 per cent).

Agricultural production has increased considerably since the Partition of the sub-continent in 1947. Up to the early 1960s, most of the growth came from more land being brought under cultivation and more intensive use (e.g., through double-cropping on previously single-cropped land). Since then, Government has made major efforts to raise agricultural output through the introduction of minor irrigation (low lift pumps and tubewells), high yielding varieties of rice and wheat, and chemical fertilizers. Flood control and drainage projects have enabled farmers to cultivate higher-yielding rice varieties in the monsoon season in some areas and have protected boro crops from early flooding in many areas.

From 1973 to 1988, foodgrain production grew at an average of 2.5 per cent annually, which was broadly in line with population growth. Despite this, production fell short of consumption needs. There was a 'food gap' which was met by food aid and commercial purchases. If the country is to become self-sufficient in foodgrain production in the next 20 years, foodgrain output will have to grown at an unprecedented annual rate of over 4 per cent annually.

Planning Exercises

Major government efforts to increase agricultural production started in the early 1960s. The importance of improved water control to agricultural development was recognized from the outset. Among the first organizations established were those that were later to become the Bangladesh Water Development Board (BWDB) and the Bangladesh Agricultural Development Corporation (BADC).

In 1964, a Master Plan for water resource development was developed which envisaged the development of 50 flood protection and drainage projects (some with gravity irrigation) covering about 5.8 Mha of land. Three types of polders were envisaged: polders with gravity drainage, tidal sluice drainage and pump drainage. Projects included in 1964 Master Plan which have been completed include: the Coastal Embankment Project (949,000 ha) completed in 1980; the Brahmaputra Right Flood Embankment (226,000 ha) completed in 1968; the Dhaka-Narayanganj-Demra irrigation scheme (4,000 ha) completed in 1968; the Ganges-Kobadak Project (Phases 1 and 2), providing wet season irrigation to 141,000 ha, which was completed in 1970; the Manu River Project (22,500 ha); the Teesta Right Embankment Project (39,000 ha); the Barnal-Salimpur-Kolabasukhali Project (25,500 ha); the Chalan Beel Project (53,000 ha); the Tarail-Pachuria Project (20,000 ha); the Satla-Bagda Project (29,000 ha); and Chandpur Irrigation project, an FCDI project completed in 1979.

In 1972, soon after Independence, the World Bank published a comprehensive Land and Water Sector Study. this study was based on the experience gained in the 1960s and on new hydrological and agro-ecological data that had become available. The recommendations of the study, called for a significantly different pattern of development from that envisaged in the 1964 Master Plan. A rapid increase in flood production through the implementation of small, quick-yielding projects, including minor irrigation and small scale FCD projects, became the main focus of government water development policy. BADC, IRDP (later BRDB), BKB and commercial banks became largely responsible for expanding medium and large-scale multipurpose schemes in more deeply flooded areas (over 2 m) to small-scale schemes emphasizing minor irrigation, most of which are located in more shallowly flooded areas (less than 1 m).

Through the 1970s and 1980s, there was a rapid expansion in minor irrigation and in the use of high-yielding varieties and fertilizers. the area under irrigation increased from 1.2 Mha in 1973 to 2.4 Mha in 1988; that under HYV paddy, more than tripled from 1.1 Mha to 3.4 Mha over the same period. In 1985, the area under minor irrigation (excluding traditional) was 1.74 Mha, that protected by FCD projects 1.30 Mha.

In the mid-1980s, a second master planning exercise was undertaken which resulted in the National Water Plan, published in 1986. The NWP concentrated on the development of water resources for agriculture, but also took into account the needs of other water users including industry, urban areas, water transport, fisheries and salinity control. The NWP recommended the continuing expansion of minor irrigation in the short and medium terms plus an increase in the area under flood control and drainage. In the longer term, the NWP recommends the development of the major rivers through barrages and regional schemes. Flood control and drainage projects were considered in the NWP, with emphasis on irrigation.

Flood Control and Drainage Projects

Flood control and drainage projects are implemented for a number of reason, including protection from main river floods, from flash floods in the east of the country, from saline intrusion in the lower delta and for drainage improvement. Projects include:

- Major projects such as the Coastal Embankment Project (949 000 ha), the Manu River Project (22 500 ha), the Teesta Right Embankment (39 000 ha), the Ganges-Kobadak Project (141 600 ha), the Brahmaputra Right Bank (226 000 ha), the Chandpur Irrigation Project (54 000 ha), and the chalan Beel Project (53 000 ha). These project include extensive embankments and water control structures and, in the case of Meghna-Dhonagoda, Manu, Muhuri and Ganges-Kobadak projects, gravity irrigation covers about 185 000 ha in addition to flood protection. In Bhola, Chandpur and Karnafuli, low lift pump irrigation is available along with flood control for an area of 140 000 ha located in moderately to deeply flooded areas.

- Medium-scale projects such as the Satla-Bagda, Chenchuri Beel and Barnal-Salimpur-Kolabasukhali projects implemented under the Drainage and Flood Control Projects (DFC I to DFC IV) financed by the World Bank. These projects typically benefit area of 10 000 to 30 000 ha and involve flood control and drainage with limited irrigation development.

- Small-scale projects such as those implemented under the Early Implementation Project (supported by Netherlands and Sweden), the Small-scale Irrigation Project (Asian Development Bank) and Small-scale Drainage and Flood Control Project (World Bank/Canada). These started mainly as flood control and drainage projects in shallowly flooded areas. Projects were designed to be implemented quickly, to have low investment coasts per hectare and to benefit areas of 1 000 ha to 10 000 ha. In recent years, upazilas have started to implement very small-scale projects (generally under 500 ha) with the assistance of the Local Government Engineering Bureau.

Flood control and drainage projects have accounted for about half of the funds spent on water development projects since 1960. Despite this, the benefits have been less than planned. There are a number of reasons for this, including cost and time overruns (due to a number of factors such as land acquisition) and problems in the operation and maintenance of projects. There is a tendency to see projects as being finished when the physical works are complete. Insufficient attention is paid to ensuring adequate water control. Problems in the operation and maintenance of projects have also been common, as is indicated by the frequent cutting of embankments by the public and malfunctioning of regulators and water control structures.

There have been few adequate evaluations of flood control and drainage projects and it is thus difficult to estimate their impact on production, incomes and employment, or to assess the operational and other problems involved and the best ways to overcome these. Evaluations of the Chandpur Irrigation Project (a major FCDI project) and the Early Implementation Projects indicate that benefits can be considerable, but are often less than planned because of operational problems. Big projects like Chandpur, however, often bring indirect benefits (eg to transport, culture fisheries) which are difficult to quantify.

Minor Irrigation Development

The development of irrigation started in the 1950's with the exploitation of surface water using low lift pumps which irrigate, on average, about 10 ha. By the mid-1970's, 0.6 M ha were irrigated with surface water and most of the easily accessible resources had been developed. Groundwater development started with deep tubewells (capable of irrigating about 30 ha) in the 1960s and continues today. The further exploitation of groundwater with individually owned shallow tubewells (irrigating, typically, 4 ha each) started in the late 1970s and by 1985 this had become the dominant mode of groundwater irrigation. In 1985, shallow tubewells (STW) irrigated an estimated 0.520 Mha, deep tubewells (DTW) 0.420 Mha, low-lift pumps (LLP) 0.720 Mha, major FCDI project 0.32 Mha and traditional methods 0.350 Mha. Over the last 20 years, the area under traditional irrigation declined from about 0.6 Mha to 0.4 Mha; that under mechanized/modern irrigation increased from under 0.1 Mha to nearly 1.8 Mha.

In the mid-1980s, there was concern about an apparent falling off in demand for new irrigation equipment and in the average area irrigated by tubewells and low-lift pumps. Following the 1987 and 1988 floods, however, demand for irrigation, especially STWs, has increased as farmers tried to compensate for the crop losses.

Irrigation has been the leading factor in agricultural development. Irrigation has made possible expansion of HYV boro and wheat cultivation. In low-lying areas, HYV boro often replaced broadcast aman leading to increases in yields per ha from 1.5 tonnes to 4.5 tonnes; on higher land, HYV boro often replaced broadcast aus or transplanted aus with increases in yields from 1.2 tonnes to 4.0 tonnes/ha. Irrigated development also typically led to increases in agricultural employment of 30 to 50 per cent per hectare. Post-project evaluations indicate that rates of return on minor irrigation projects are generally in the region of 20 to 40 per cent.

Potential for Growth

The UNDP Agriculture Sector Review emphasizes the considerable potential for agricultural growth in Bangladesh. Yields of rice, wheat and other main crops are lower than in other countries in the region, as are the use of irrigation, fertilizer and other inputs. Bangladesh has generally productive, alluvial soils and a climate that permits plant growth throughout the year. With improved water control (through flood control, drainage and irrigation), the development of new varieties of rice and other crop varieties suited to the varied agro-ecological conditions in the country, and improved input supply and extension, Bangladesh can greatly increase agricultural output.

Scope for Intensification of Dry Season Agriculture

There is considerable scope for the further expansion of dry season agriculture over the next 15 to 20 years. The National Water Plan estimates that the area under irrigation could be almost tripled from about 1.8 Mha to 5.5 Mha (from 26 to 76 per cent of the net cropped area). This would be achieved through:

- groundwater development to irrigate 2.3 Mha with shallow tubewells, deep-set shallow tubewells and deep tubewells;

- surface water development to irrigate and additional 1.2 Mha by low-lift pumps, withdrawal schemes from main rivers and major irrigation schemes (FCDI).

The expansion in the area under irrigation would greatly increase HYV boro production.

After the end of the National Water Plan period in the year 2005, a further development of 1.5 Mha of irrigation would be possible but would involve major projects, such as barrages, to divert stream flow from the main rivers and possibly the integrated development of major FCDI projects.

Scope for the Intensification of Wet Season Agriculture

There is also considerable scope for the further development of wet season agriculture through more widespread supplementary irrigation of monsoon season crops (especially transplanted aman), the development of new FCD and FCDI project and by the more effective operation of existing and future major schemes. The National Water Plan estimated that an extra 1.8 Mha of protection (raising the total net benefited area to 3.1 Mha) could be implemented by the year 2005, but this did not take account of possible major project along main rivers which would increase this potential area considerably.

As was noted earlier, flood control and drainage projects make possible increased agricultural production by reducing the risk associated with monsoon agriculture and, where pump or tidal drainage can be provided, making the cultivation of HYVs possible in monsoon season as well as, with irrigation, in the dry season. Although there have been few evaluations of FCD and FCDI projects, there are a number of examples of projects that have been successful. The Chandpur Irrigation Project, for example, is a successful tidal and pump drainage project that made possible the cultivation of HYV aman on land where lower-yielding broadcast aman was previously grown; the Early Implementation Projects are examples of generally successful smaller schemes which reduce the risk to farmers associated with the cultivation of traditional cropping patterns.

To achieve potential changes in cropping patterns and increases in production from flood control and drainage projects, such projects need to be designed carefully and operated effectively. Likely problems in scheme operation need to be anticipated during planning and taken into account in project design (eg, inclusion of ring bending within polders to protect lower land; provision of minor irrigation on higher land). Projects also need to be operated and maintained effectively which requires participation of the beneficiaries and possibly local government in management. Effective operation and maintenance will be even more important in the case of the compartments envisaged for 'controlled flooding'.

Attention must also be given in project design and operation to reducing any negative social impacts of FCDI and FCD projects and to minimize any negative environmental impacts. Land acquisition procedures need to be improved and provision made for occupational training and skill development for those whose land is acquired. Projects should be designed, as far as practicable, to minimize any negative impacts on fishing or boating households and on the fisheries resource.

Floodplains are critical for capture fisheries in Bangladesh. Floodplains and beels provide the habitat for a highly productive open water capture fishery and spawning grounds for major river fisheries. The productivity is dependent, however, upon natural hydrologic patterns which can be disrupted by flood control and drainage projects. Care needs to be taken in the planning and operation of such projects to minimize any detrimental impacts on fisheries (eg through the provision of structures to allow entry and exit of fish at appropriate times).

To make the most effective development of agriculture based on flood control, drainage and irrigation projects (including minor irrigation), improvements in farming technology, including more intensive use of inputs, will be essential. A number of new varieties of paddy and other crops are under development and may be ready for introduction within the next decade. These include improved varieties of deepwater aman and new varieties of HYV aman which are photoperiod-sensitive and suitable for late transplanting after the recession of floodwater. Privatization of input distribution (eg fertilizers, pesticides) has improved overall access to them by farmers, though problems still remain in more remote areas. Over the next two decades, a high priority must be given to agricultural research, extension and input supply. The UNDP-financed Agricultural Sector Review discusses these issues. There should also be a closer integration of activities of agencies concerned with the development of flood control and drainage and those concerned with agricultural, fisheries and livestock development in the planning and implementation of FCD and FCDI projects.

Overview By Region

The north-west (NW) region has high potential for the further development of surface water irrigation from the rivers Brahmaputra, Teesta, Darla and Dudhkumar. Besides, the region has further potential of groundwater development by tubewells, particularly on the extensive areas of F0 and F1 land. This would be for HYV boro and aman cultivation on impermeable soils and for irrigated rabi crops (eg wheat) on permeable soils. Flood control along the Brahmaputra-Jamuna is essential to the securing of kharif crops: aus, jute and transplanted aman in the north and center, deepwater aman is the south. The same is true along the Ganges. To gain full benefits from such major embankments, improved water control through smaller projects behind the main embankments is needed. In the Chalan Beel, flood control embankments to protect the HYV boro crop could be further developed, through the problem of evacuating rainwater in the rainy season requires a solution.

The north-central (NC) region needs flood protection, especially in the south and center of the region, for further improvement of crop production in the rainy season. Controlled flooding would increase security of aus, deepwater aman and jute crops, which might encourage farmers to invest more in fertilizers and improved seeds. Like the northwest region, this region offers high potential for surface water irrigation from the Brahmaputra and its distributaries. In addition, development of groundwater irrigation is envisaged for HYV boro cultivation and for supplementary irrigation of aman, especially in the north.

The north-east (NE) region is hydrologically the most difficult region to develop due to the very high rainfall in the region and surrounding hill areas, deep flooding in the haors in the Sylhet basin and low river gradients below the exit from the region. Controlled flooding as in the NC region may be feasible in the western part of the old Brahmaputra floodplain. Elsewhere, further development of submersible embankments (in the center and east) and embankments against flash flooding (in the north and east) will be needed to secure the boro crop and, in places, the aus and transplanted aman crops.

The south-west (SW) region requires channel improvements to relieve drainage congestion in interior areas and embankments with sluices to control flooding by saline water in coastal areas so as to secure the transplanted aman crops and expand shrimp farming. The region has potential for further irrigation development for dry land rabi crops on permeable ridge soils and HYV boro on depression clays. There is significant potential for surface water irrigation development from the Ganges in this region which would at the same time ensure some relief from salinity to the industries in Khulna in the dry season. Supplementary irrigation of aus, aman and jute would also be beneficial.

The south-central (SC) region also experiences severe hydrological problems (see para. 13, above). In the north, controlled flooding following protection from the ganges and Padma floods would secure jute and mixed aus and aman crops. In the interior parts of the north and center, channel improvements together with embankments (perhaps submersible) could increase the area suitable for secure HYV boro, dryland rabi crops, jute, aus and aman crop production. In the south, embankments will sluices are needed to provide protection against tidal flooding, occasional storm surges and, in the extreme south, saltwater flooding. Such facilities would help secure transplanted aman and HYV boro/aus cultivation. The region offers excellent potential for surface water irrigation development through diversion of Ganges waters. There is also further potential to expand tubewell or low-lift pump irrigation in most areas in order to increase HYV boro/aus production and, in the north, high-yielding dryland rabi crops.

The south-east (SE) region is the most intensively cultivated region of Bangladesh. It has potential for further irrigation development. Surface water irrigation development could be achieved with new projects similar to the Chandpur Irrigation Project and with water diversion from the middle Meghna river through the existing network of man-made khals. Controlled flooding on land adjoining the middle Meghna would bring benefits similar to those in the NC region. Embankments to protect against flash floods would benefit boro, aus and aman crops in the east of the region in Comilla, Noakhali and Chittagon districts.

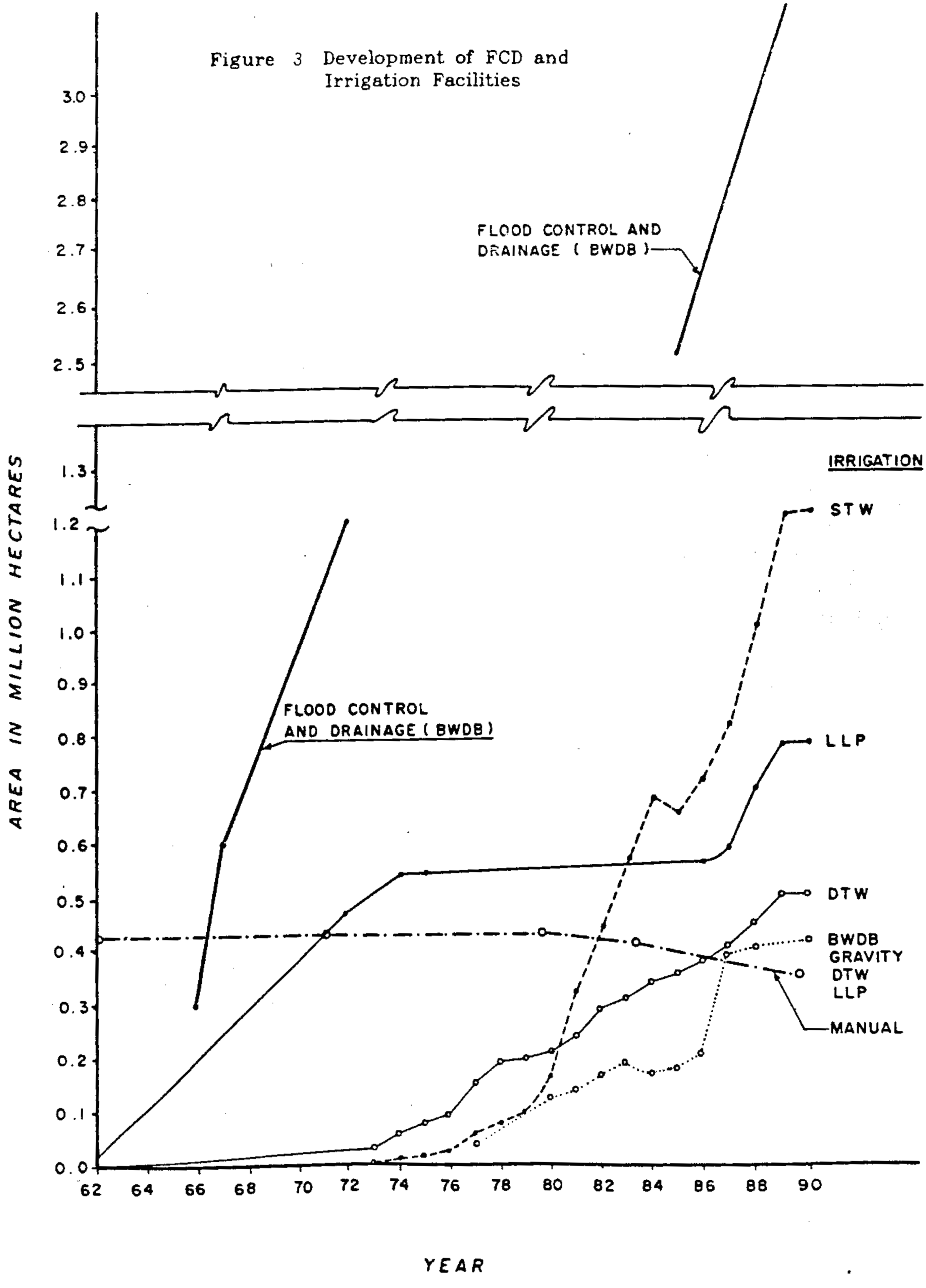
Growth of Flood Control, Drainage (FCD) and Irrigation Facilities.

The area provided with flood control and drainage facilities has grown steadily since mid 1960's to about 3.37 M ha through the construction of 7,555 kms of embankment (coastal embankment 3,674 kms, and embankment in upland areas 3,881 kms), 7,907 hydraulic structures which includes sluices and regulators and 1,082 river closures by Bangladesh Water Development Board. In addition BWDB has constructed 3,204 kms of drainage channels, 4,620 kms of irrigation canals and 4,095 bridges and culverts. BWDB has developed irrigation facilities for 0.44 M ha and has reclaimed 0.1 M ha of land. Please refer to Figure 3 for graphical representation of development of FCD and irrigated areas in Bangladesh.

Development of FCD facilities by BWDB for 3.37 M ha is equivalent to 24 percent of total area and 39 percent of net cultivable area. The FCD facilities were developed steadily from mid-sixties to present time.

Irrigation facilities exist for 3.10 M ha (32 percent of NCA) of which BWDB developed 0.44 M ha (14 percent of irrigated area or 4.5 percent of NCA)

Figure 3 Development of FCD and Irrigation Facilities



Performance of FCD Projects

Flood Control and Drainage Projects are designed "to establish conditions for the adoption of HYV - fertilizer technology" 1/ FCD projects, in other words, are to be designed to establish conditions in which the annual depth, timing and duration of flooding are within a definite range with a high degree of probability. This will enable the farmer to shift to local improved and high yielding varieties of transplanted Aman in these areas. In other words, the FCD Projects will increase the cropping intensity and production of Aman and Aus rice.

Though there were major increase in the development of FCD areas (Figure 3) in Bangladesh, it appears from Table 1 and Figure 4 that there was no noticeable increase in the area and production of aman rice during the period 1973 and 1988. Production of aus has declined during this period. The boro production has increased greatly over this period mainly due to development of irrigation facilities.

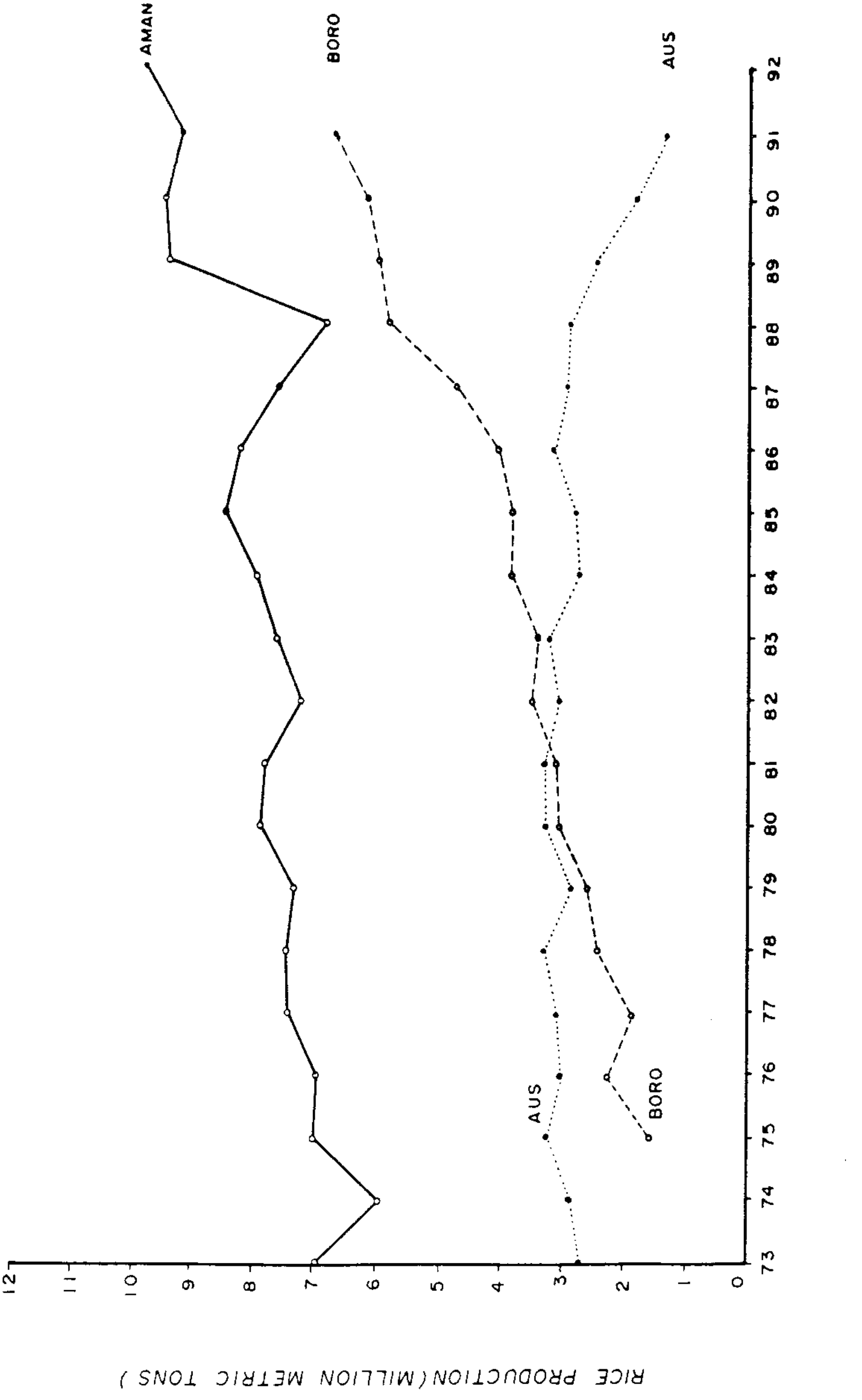
1/ National Water Plan, Summary Report, December, 1986, Master Plan Organization.

Table 1

Rice Production in Bangladesh

	Aus		Aman		Boro	
	Area (Mha)	Production (Million Metric Ton)	Area (Mha)	Production (Million Metric Ton)	Area (Mha)	Production (Million Metric Ton
1973 - 74	3.11	2.80	5.72	6.70	1.05	2.22
74 - 75	3.18	2.86	5.45	6.00	1.16	2.25
75 - 76	3.42	3.23	5.76	7.04	1.15	2.29
76 - 77	3.22	3.01	5.81	6.91	0.85	1.65
77 - 78	3.16	3.10	5.77	7.42	1.09	2.24
78 - 79	3.24	3.29	5.81	7.43	1.07	1.93
79 - 80	3.04	2.81	5.98	7.30	1.15	2.43
80 - 81	3.11	3.24	6.04	7.84	1.16	2.59
81 - 82	3.15	3.22	6.01	7.10	1.30	3.10
82 - 83	3.16	3.02	6.00	7.48	1.43	3.49
83 - 84	3.14	3.22	6.01	7.60	1.40	3.46
84 - 85	2.94	2.78	5.71	7.93	1.57	3.89
85 - 86	2.85	2.83	6.02	8.54	1.53	3.79
86 - 87	2.91	3.13	6.06	8.27	1.65	4.08
87 - 88	2.97	2.99	6.59	7.68	1.94	4.80
88 - 89		2.86		6.86		5.83
89 - 90		2.48		9.47		6.00
90 - 91		2.00		9.50		6.30
91 - 92		1.86		9.30		6.07
92 - 93				10.00		

Source: Bangladesh Bureau of Statistics.


RICE PRODUCTION (MILLION METRIC TONS)
0
1
2
3
4
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6
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12
73
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75
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83
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91
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YEAR
AMAN
BORO
AUS
AUS
BORO
SOURCE: BBS

Figure- 4

From the above information, can we suggest that FCD projects in Bangladesh did not produce noticeable benefit.

If so, what could be the reasons ?

During the major floods in 1987 and 1988, large part of the BWDB's FCD infrastructure were damaged. Official estimates of losses and damage to FCD infrastructure by floods in 1987 and 1988 are summarized below (Table 2).

Table 2

Official Estimates of Losses and Damage by Floods in 1987 and 1988.

Item	Loss/damage	
	1987	1988
Flood embankments	1,279 km	1,990 km
Irrigation / drainage canal	222 km	283 km
Irrigation / drainage control structures	541	1,465
Roads truck 1/	1,523	3,000
rural	15,107	10,000
bridges / culverts	1,102	898

Though it may safely be assumed that many of the FCD facilities were not functioning in 1989 and 1990, the Aman rice production was significantly higher in these years (Figure 4 and Table 1). The aman production has been very good in subsequent years also. What could be the reason for such bumper aman crops after 1989 ?

1/ Roads are used as flood embankments in may cases.
SOURCE: Mainly BWDB.

The farmers in Bangladesh are opposed to total elimination of flooding, they welcome 'some' flooding. Farmers welcome deposition of silt by shallow flooding and firmly believe that such silt deposition increases the fertility of soil which could be due to micro-nutrient content in silt. While embankments may provide protection against flooding, it can also cause waterlogging and drainage congestion if adequate number of drainage structures are not provided. With lots of breaches in embankments, most probably the drainage during monsoon and post-monsoon was good in some years. The whole matter needs serious attention and thorough investigation.

Causes For Inadequate Return From FCD Projects

In the following sections, there are discussion about causes for poor return from FCD projects.

It is very difficult to keep the polders water tight. There are breaches in the embankments due to river erosion and public cuts which causes flooding in the project area.

River-bank erosion provides the main threat to embankments; (in passing, it may be added that it also provides a major hazard for towns situated along the major rivers, such as Serajganj and Chandpur). The Brahmaputra right-bank embankment (BRE) and embankments along most of the eastern rivers have breached several times, allowing serious flooding to occur periodically on adjoining 'protected' land; (this has occurred again in 1990, first by early flash floods along rivers in the north-east; and later in the Kazipur and Serajganj sections of the BRE). More than half the 217 km length of the BRE has been retired since it was completed in 1968, some sections several times, one section (near Belkuchi in the south) eight times. Some retired sections are now located about 2 km inland from the alignment of the original embankment, which was built at a distance of 1.6-2.4 km from the 1968 river bank. Retirements have also been needed along sections of the Chandpur Irrigation Project embankment and embankments along some of the eastern rivers.

The Brahmaputra-Jamuna has a marked tendency to erode its right bank, but some sections on the left bank have also been seriously eroded in recent years. Attempts to stabilize the channel near Serajganj town by means of revetment and groynes have been very costly and not totally successful. The Brahmaputra has a braided channel in which unpredictable changes in flow and direction can take place, sometimes rapidly. Coleman measured lateral changes in bank configuration of up to 850 m within a single year 1/ The lower Meghna adjoining Chandpur town and the Chandpur Irrigation project also has a braided channel, which has caused serious erosion problems. Erosion here and in the south-west of the neighbouring Meghna-Dhonagoda project is aggravated by exposure to waves generated in the 4-5 km wide Meghna channel during the monsoon season. Along the eastern rivers, bank erosion takes place mainly at meander bends, which is more easily predictable. The Ganges and Padma rivers also have mainly meandering courses where prediction of bank erosion is simpler (though not necessarily less costly to prevent).

Public cuts in embankments occur in two situations: where people living on the river side of an embankment seek to relieve high flood levels threatening their homes or land by releasing water into adjoining protected land (which is usually ineffective); and where run-off from heavy rainfall accumulates on the inside of an embankment and submerges farmers' crops. This problem has been most serious in embankments facing the Atrai river in the Chalan Beel project; public cuts have also occurred occasionally in the BRE and in embankments along some of the eastern rivers. It is a problem which could become more serious in future as population pressure grows on land lying outside major embankments and especially if confinement of such rivers by embankments along both their banks increases river flood levels.

1/ J.M. Coleman, Brahmaputra river: Channel processes and sedimentation. Sedimentary Geology, 1969, vol. 2/3, pp 129-239.

Poor design has generally not been a reason for failure of major embankment, except in so far as the selection of embankment alignments too close to eroding river bends on some eastern rivers and inadequate consultation with local residents in designing the Chalan Beel project may have been unwise. The major embankments along the Brahmaputra-Jamuna and Ganges rivers were designed to withstand a 1 in 100 year flood, and none was overtopped during the exceptionally high 1987 and 1988 floods. Failures occurred in embankments built to lower design standards, including inadequate crest width, side slopes and berm width between embankment and borrow pit.

Review reveals two other design problems, mainly relating to small-scale projects.

(a) One is the lack of up-to-date topographic maps. The existing 1:15, 850 and 1:7, 920 irrigation planning maps with 1-foot contours were made about 30 years ago, since then there have been significant changes in some places in settlements, roads, river courses and even in land levels (due to sedimentation and possible tectonic subsidence or uplift). Incorrect topographic detail can produce incorrect designs for structures which may lead to ponding of water and then public cuts or breaching.

(b) Different run-off criteria are used for designing structures on different projects, even within BWDB:

- 10-day 1 in 10 years rainfall for the World Bank's FCD-III and Second Small Schemes FCDI projects;

- 10-day 1 in 5 years rainfall for the World Bank's Small Schemes projects;

- 5-day 1 in 5 years rainfall for the Dutch Delta Development projects;

- 3-day 1 in 5 years rainfall for the Teesta projects;

Additionally, the maximum allowable flood depth within protected areas varies between 50 cm and 150 cm in the pre-monsoon season and between 150 cm and 400 cm in the monsoon season, both rates calculated for 72 hours duration. Whilst, to some extent, these differences reflect regional differences in physiography, rainfall and cropping patterns, there would seem to be scope for closer standardization.

Poor operation and maintenance represents an important threat to embankment project. Generally, the problem arises from inadequate fund allocations rather than from lack of skills or diligence on the part of staff responsible for operating projects. This can result in:

(i) lack of embankment inspections to identify potential problems;

(ii) inadequate attention to routine maintenance and repairs including rat holes;

(iii) inability to prevent encroachment of settlers on the embankment;

(iv) inability to acquire land for constructing new embankment sections when existing embankments are threatened by advancing river erosion;

(v) inadequate patrolling during floods; and

(vi) inability to take emergency actions during floods.

Public opposition to land acquisition for realigning an embankment section can also be a problem, as well as the public cuts described above.

In some polders, design assumptions are not followed during project operation: e.g., that no flow will enter from outside; that all regulators/sluices will operate simultaneously. Unplanned developments (such as new roads, drains, urban development) can also upset hydraulic design assumptions. These changes can subject structures to flows or pressures greater than they were designed to withstand, leading to risk of failure.

<u>Submersible embankments.</u> These are low embankments built around floodplain basins to prevent early flash floods (and sometimes local rainwater floods) from damaging boro rice and sometimes young aus and deepwater aman crops. They are especially used in the <u>haor</u> areas in the north-east (Regions 20, 21 and 22) where very deep flooding prevents any crop from being grown in the monsoon season. Regulators and spillways are provided in some embankments, and internal low bunds are sometimes added to retain water and provide protection to lower-lying land in case of the main embankment breaching. The embankments and structures are submerged during the monsoon flood season.

Submersible embankments are vulnerable to breaching by early flash floods. Damage is particularly liable to occur when embankments are overtopped at a time when there is a big difference in levels between floodwater outside the embankment and land on the inside. Such floods may also damage regulator structures when flows exceed their design capacity. Maintenance of these embankments in remote areas provides a problem, and inadequate maintenance often is a contributory factor to embankment and regulator failures.

<u>Water Controls to satisfy the water requirements for lands of different elevations in a polder</u> are difficult to achieve, Bangladesh's flood plains are almost flat. The highest floodplain elevation in the north-west corner of Bangladesh is about 90 m above mean sea-level, giving an overall gradient over the 500 km to the coast of less than 20 cm/km; south of Dhaka, the average gradient is only 1.6 cm/km. However, Bangladesh floodplains are not absolutely flat (see Figure 5); nor are they uniform in relief throughout the country.

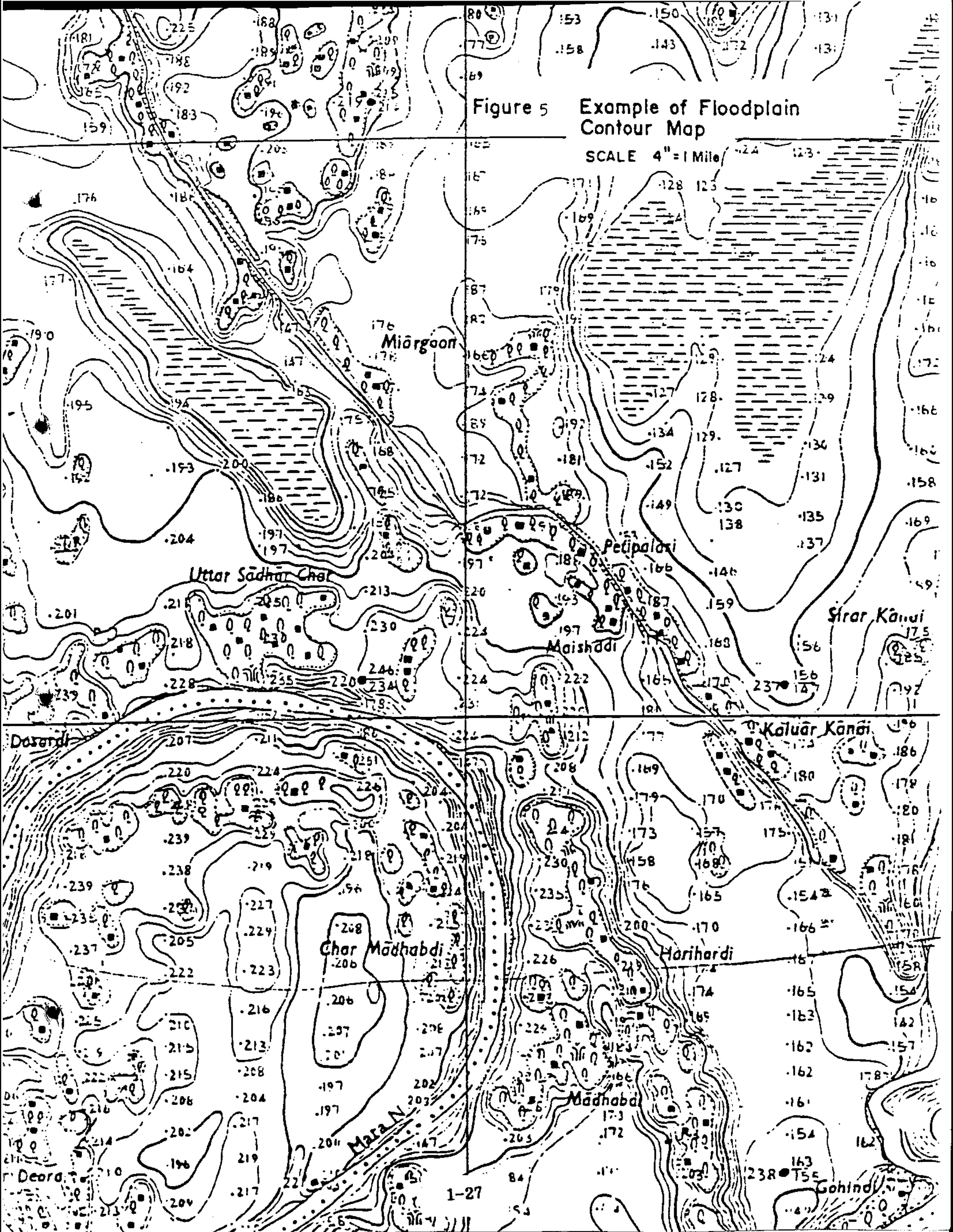


Figure 5 Example of Floodplain Contour Map

SCALE 4" = 1 Mile

Even in a small polder, there may be considerable variation in the land elevation. Cropping pattern is dependent on the elevation and that's why MPO developed land classification based on flood depth.

With watertight embankments in a polder the low (not very low) and medium low lands are benefitted because of elimination of flooding, but the medium high and high lands suffer due to lack of water. Prior to construction of flood embankments, the high and medium high lands were getting the benefit of the water standing on low and medium low lands. The benefits in poldered areas are shifted from higher to lower lands and most probably the net benefit is not noticeable.

The present concept of flood control by only embankments and some drainage regulators may not be adequate. Improved water control and irrigation facilities are required to meet the water requirement of high, medium and low lands.

PHYSIOGRAPHY

Bangladesh is generally described as a delta or as a flat alluvial plain, with about half the surface of the country below the 7.62 meters contour line (Figure 6). The physiography, however, presents considerable local and regional variety. Geologically speaking, the land can be divided into three broad categories of physiographic regions. These are the Tertiary Hills, the Pleistocene Uplands, and the Recent Plains.

The Tertiary Hills are located in the Chittagong Hill Tracts region in the South East, the only region in the country that experienced upheaval contemporaneous with the Himalayan Orogeny. Formed mainly of sand-stones and shales, the average height of the hills is around 300 m., the highest peak being Mowduk Mual (1,004 m) on the Bangladesh-Burma border.

The Pleistocene Uplands comprise the Madhupur Tract north of the City of Dhaka and the Barind Tract to the North West of the country. The altitude of the Madhupur Tract, a continuous block of about 3900 sq.km., varies between 9 m to 18 m. The Tract rises as an island above the flood plains all around, its elevation being due, perhaps, to regional uplift and erosion of adjacent areas that were subsequently filled in with newer alluvial deposits. The Barind, although discontinuous at present, extends over a much longer area. It attains a height of about 40 m in places. One notable outlier of pleistocene formation is the Lalmai Hills near Comilla.

Other areas of the country contain flood plains made up of continental deposits, deltaic plains built up by both marine and continental deposits, and the tidal plains composed predominantly of marine deposits. These plains are all of very low elevation with considerable areas lying below the 3 m contour line, and, taken together, they occupy about 90 percent of the area of Bangladesh. A narrow strip along the coast of Chittagong represents a coastal plain. In general, the alluvial plains of Bangladesh, formed by the Ganges and Brahmaputra rivers, rise with very low gradients from the Bay of Bengal in the south to the foot hills of the Assam hills in the northeast and the Himalayas, a few kilometres outside

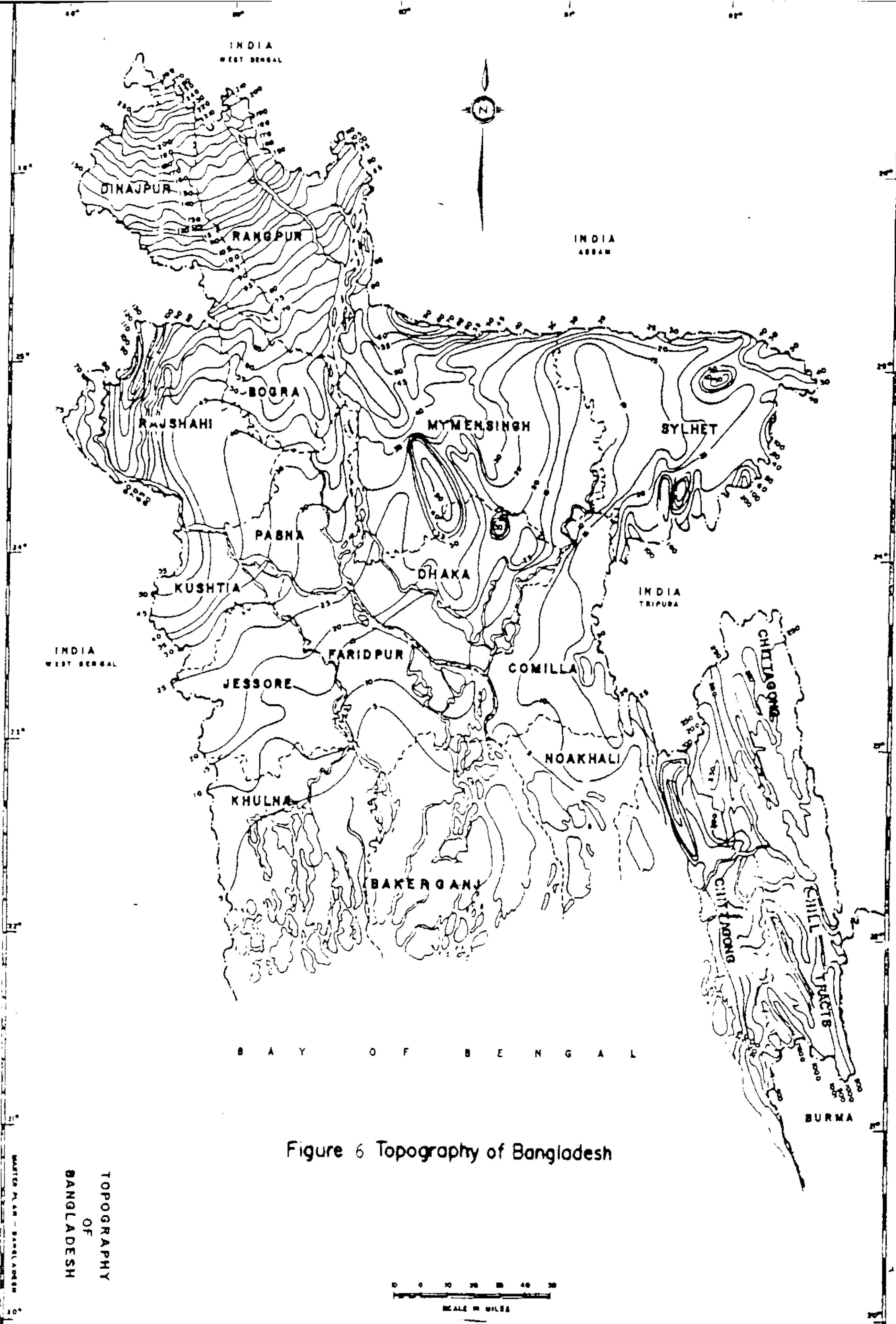


Figure 6 Topography of Bangladesh

TOPOGRAPHY
OF
BANGLADESH

MASTER PLAN - BANGLADESH

PLATE II - 2

Bangladesh, in the north and the northwest. In the northeast, elevations rarely exceed 15 m, and in the far northwest they reach to almost 90 m.

This overall picture disguises considerable variations in local relief. Changes of the river courses over time, tectonic disturbances, and differential erosion and sedimentation characteristics have created a number of depressions and slightly elevated areas in this vast plain. The extensive coastal tidal plain lies only a few meters above sea level, and the Sylhet depression in the northeast is of comparable elevation. Few areas are truly flat except for the most recently added coastal plain. The smoothest areas are on the old Meghna flood-plain and the Ganges tidal floodplain, where differences in elevation between adjoining ridges and basins are generally less than 1.5 m. Human habitation is crowded on higher ground which is not flooded at all or only to shallow depths. Wherever there is flooding, the villages are built on mounds. During the rainy season, these mounds stand like small islets, and country boats become the only means of travel. These complex local differences in elevation are important in determining the flooding and cropping patterns of the land and in planning for irrigation and drainage, especially as they are usually associated with important differences in soil permeability and associated crop suitability.

CLIMATE

The climatic elements such as rainfall, hail storms, temperature, humidity and solar radiation all affect crop growth and yield. The critical aspects of climate in relation to crops and cropping patterns are:

(a) the occurrence and reliability of the pre-monsoon rains and the onset of the monsoon;

(b) the reliability, amount, and distribution of the end of monsoon rains;

(c) the occurrence of storms and cyclones that damage standing crops and;

(d) the rise, duration and recession of floods associated with the monsoon rains.

All of these climatic elements have to do with rainfall which is the main source of variation in the crop environment of Bangladesh.

Rainfall is a very significant component of the water cycle and water resources planning in Bangladesh because it provides major part of crop water requirements during monsoon and early monsoon periods, determines the magnitude of drainage requirements and recharges the groundwater reservoir.

There are three distinct climatic seasons in Bangladesh. The monsoon (rainy) season from May to October experiences more than 90 percent of the total annual rainfall. It is characterized by high temperature, high humidity and low solar radiation. The dry, cool season from November to February receives negligible rainfall and is characterized by low temperature, low humidity and high solar radiation. The pre-monsoon, hot season from March to April receives some rainfall in occasional heavy thundershowers and hailstorms and is characterized by the highest temperatures and evaporation rates.

The seasonal and regional variation of rainfall in Bangladesh is given in Table 2. Mean annual rainfall ranges from about 1400 mm in Rajshahi District (Northwest

Region) to about 6000 mm in Sylhet District (Northeast Region) (Figure 7). In general, the Northwest Region except Rangpur and the Southwest Region except Patuakhali and Barisal are the low rainfall zones and are considered as the drought prone areas of the country. Following the monsoon, the availability of soil moisture declines and falls short of the crop demand during the Rabi season. Then during the pre-monsoon season, erratic distribution of rainfall also causes soil-moisture deficits for crops. Potential soil moisture deficiencies over 6-7 months seriously limit crop production in Bangladesh.

Crop environment during the monsoon (Kharif) season is not favourable for achieving full potential yields because of uneven distribution of rainfall, flooding at variable depths, low solar radiation, and high temperature and humidity during this season. On the other hand, high solar radiation, low humidity and wide variation in day and night temperature during the winter (Rabi season) are very favourable for achieving high potential yields, but the lack of adequate soil moisture seriously limits expansion of cropped area. Therefore, both drainage and irrigation are essential modes of development for increased crop production.

Table 2

Mean monthly and annual rainfall

Name and Station	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Patuakhali	13.1	24.0	61.1	111.1	269.3	595.2	659.3	585.6	405.3	279.2	50.6	12.7	3066.9
Barisal	15.9	19.0	48.1	104.5	196.8	423.4	451.9	395.7	285.3	190.4	42.3	9.9	2183.2
Khulna	14.6	22.3	40.9	90.6	173.9	318.3	359.4	309.8	213.8	138.0	27.6	6.3	1715.5
Jessore	11.1	25.0	43.4	92.4	185.6	305.8	329.0	301.6	206.6	127.1	19.7	4.7	1652.0
Kushtia	10.6	17.7	37.3	75.5	178.4	293.3	282.5	283.9	222.0	129.0	20.6	3.6	1555.2
Rajshahi	13.6	15.7	28.6	42.5	131.1	266.6	315.0	260.7	237.3	110.9	13.0	3.0	1438.0
Pabna	8.9	20.0	42.7	712.4	175.8	288.3	267.8	273.7	224.3	141.8	16.2	4.0	1534.9
Bogra	12.2	16.9	28.1	58.7	198.2	324.5	337.1	332.7	275.8	153.9	15.6	2.7	1756.4
Dinajpur	10.1	12.3	20.1	53.4	166.2	341.8	398.5	327.6	305.0	133.1	9.5	0.8	1778.4
Rangpur	11.3	14.8	28.2	84.6	298.0	492.6	411.3	333.1	317.4	166.6	10.0	1.4	2169.3
Tangail (Gopalpur)	12.6	16.4	17.6	126.3	195.5	343.5	332.4	306.4	192.1	176.6	12.3	0.7	1732.4
Faridpur	6.8	9.3	38.1	148.1	205.2	353.7	303.7	308.1	234.1	160.1	33.3	13.3	1813.8
Dhaka	5.6	5.3	46.5	89.9	247.9	328.6	334.0	339.8	235.1	152.3	108.8	34.4	1928.2
Mymensingh	11.7	16.3	46.5	113.3	296.7	456.4	388.4	399.8	314.7	172.0	13.7	2.0	2231.5
Kishorganj	18.0	20.8	55.9	169.9	306.6	467.4	406.9	424.3	321.3	193.0	32.7	13.8	2430.2
Comilla	8.5	16.0	45.4	179.8	244.6	457.1	453.2	376.0	232.3	204.8	57.0	24.0	2298.7
Ctg. H. T.	17.4	28.8	58.6	134.8	251.3	493.3	543.1	469.9	308.9	158.3	63.6	38.1	2566.1
Chittagong	14.3	15.4	41.1	131.3	224.6	598.8	578.6	602.9	270.2	202.7	61.7	21.8	2914.4
Noakhali	6.5	20.1	60.3	103.9	266.8	603.5	645.8	572.0	395.1	220.4	56.2	27.4	2978.0
Sylhet	19.6	47.4	81.8	315.0	494.2	973.7	435.7	600.0	425.4	176.5	35.3	15.0	3923.2

Source : Mañalo, E.M., Agr-climatic Survey of Bangladesh, BRRI/IRRI, 1976.

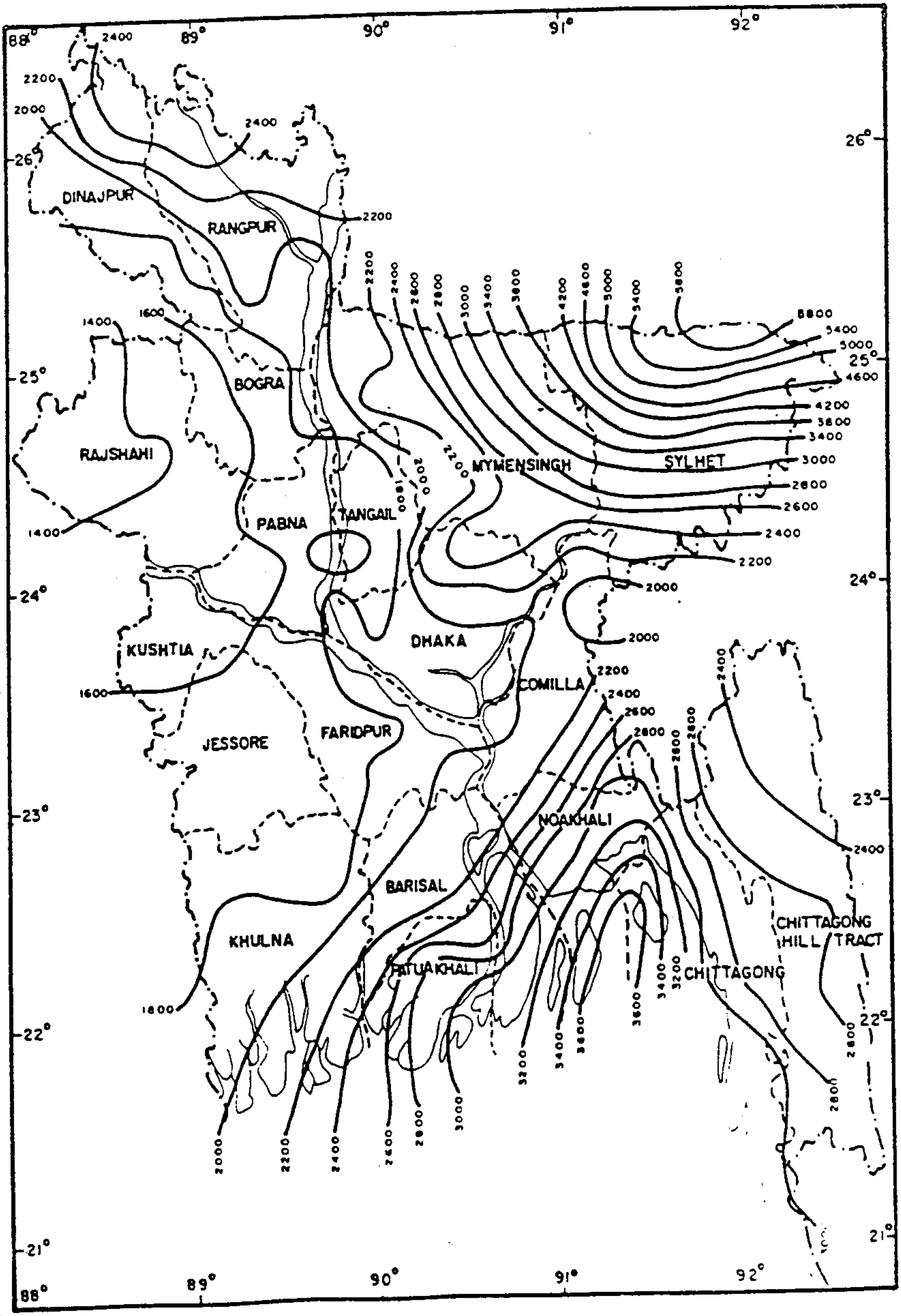


Figure 7. Mean Annual Rainfall (in mm)

A number of constraints are inherent in the monsoon rainfall and climate pattern. The 5 - 6 months dry season restricts the cultivation of perennial and annual dry land crops to soils with superior moisture holding capacity, found only in limited areas, or to irrigated land. Uncertainty of pre-monsoon showers causes variations in aus and jute acreage and yields and cloudiness imposes limitations in monsoon season production, encouraging various crop diseases and making it difficult to harvest, dry, store and transport crops. Finally monsoon season cloudiness reduces the yield potential of most rice varieties below what can be achieved in the sunny dry season.

SOIL

Bangladesh's 21 General Soil Types are differentiated at the highest level into three physiographic groups: floodplain soils, hill soils and terrace soils. Floodplain (including piedmont) soils occupy 80 percent of the country's land area, hill soils 12 percent and terrace soils 8 percent. Figure 8 presents a highly generalized map showing the description of floodplain, terrace and hill soils.

Floodplain Soils

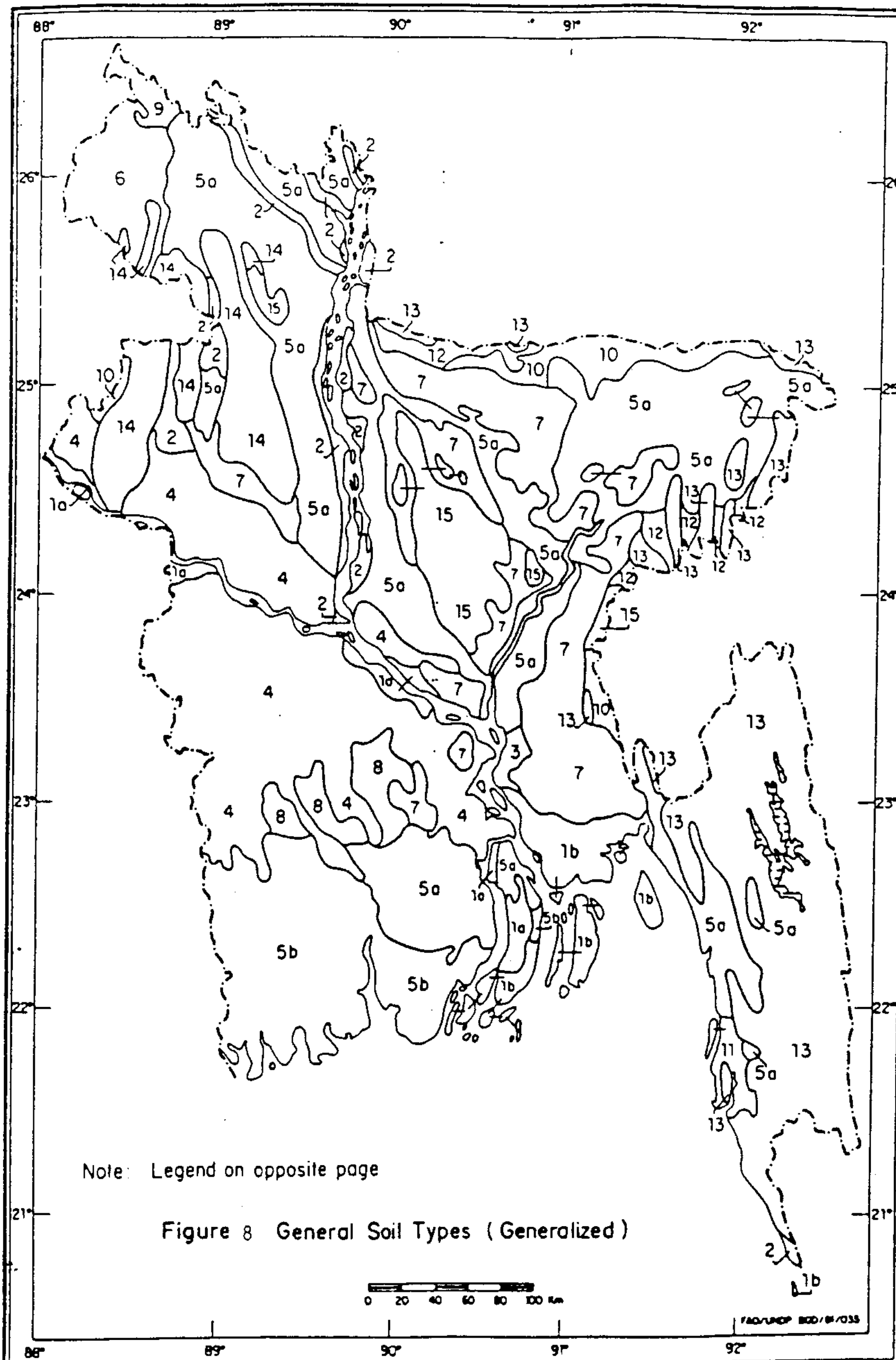
These soils have formed in river and piedmont alluvium ranging from very recent to several thousand years old. Except in the youngest soils, the original alluvial *stratification* generally has been broken up to a depth of 20-50 cm or more, and three main layers (horizons) can be recognized:

- the topsoil which generally has been disturbed by cultivation and may have a compact ploughman at the base;

- the subsoil in which alluvial stratification is well broken up or is not recognizable, and in which soil properties such as structure, porosity and mottles have developed;

- the substratum comprising raw or stratified alluvium; in some soils, the substratum consists of one or more buried older soil profiles.

There is a general pattern of silt loam or sandy loam soils on the highest parts of floodplain ridges grading through silty or sandy clay loams on intermediate sites to silty clays or clays in basins. However, the relative proportions of these different textures vary considerably between physiographic units and sometimes between different areas within such units.



Note: Legend on opposite page

Figure 8 General Soil Types (Generalized)

Legend for Figure 8

General Soil Types

1a	Calcareous Alluvium (non-saline)
1b	Calcareous Alluvium (seasonally-saline)
2	Noncalcareous Alluvium
3	Calcareous Grey Floodplain Soils
4	Calcareous Dark Grey Floodplain Soils
5a	Noncalcareous Grey Floodplain Soils (non-saline)
5b	Noncalcareous Grey Floodplain Soils (seasonally saline)
6	Noncalcareous Brown Floodplain Soils
7	Noncalcareous Dark Grey Floodplain Soils
8	Noncalcareous Dark Grey Floodplain Soils and Peat
9	Black Teras Soils
10	Acid Basin Clays
11	Acid Sulphate Soils
12	Grey Piedmont Soils
13	Brown Hill Soils
14	Shallow and Deep Grey Terrace Soils
15	Deep Red-Brown Terrace Soils

Note : Only the principal General Soil Type of the unit is indicated. A more detailed soils legend is given on the Agroecological Regions and Subregions Map in Report 2, of land Resources Appraisal of Bangladesh for Agricultural Development by UNDP and FAO, 1988.

All except the highest soils are subject to seasonal flooding. Flooding is mainly by accumulated rainwater or the raised groundwater-table. Flooding with silty water occurs mainly on land close to major river channels, in hill-foot areas and on unembanked parts of tidal and young estuarine floodplains. Depth of flooding varies both regionally and locally : mainly shallow in the north, west, east and south of the country, becoming deeper towards the centre; and relatively deeper in basins than on adjoining ridges. Flooding starts in April-June in basins and in June-July on ridges. Flood-levels normally are highest in July-August except on the Ganges River Floodplain where peak levels normally are reached in August-September. Floodwater drains rapidly from most sites in September-November, but generally basin centres stay wet for part or all of the dry season.

Permeability is low in basin clays and in most loamy ridge soils which are puddled for cultivating transplanted paddy. Exceptions include cracking clays on the Ganges River Floodplain, loamy soils on the Old Himalayan Piedmont Plain which are used for transplanted paddy, and other loamy ridge soils which are not puddled for transplanting paddy: these soils have moderate or rapid permeability. Moisture-holding capacity ranges from high in deep silt loams on the Teesta Meander Floodplain and the Old Meghna Estuarine Floodplain to low in most basin clays, some sandy ridge soils, shallow soils overlying sand, and most soils whose topsoils are puddled for transplanting paddy and which are underlain by a ploughman.

Soil organic matter contents range from low in recent alluvium and in most ridge soils to moderate (locally high) in basin soils and in Black Teras Soils in the north of the Old Himalayan Piedmont Plain. Topsoils generally are slightly to very strongly acidic except in some calcareous Ganges River Floodplain and Young Meghna Estuarine Floodplain soils and in recent alluvium. Subsoils mainly are between slightly acidic and moderately alkaline, but they are more strongly acidic in some piedmont and basin soils and are more alkaline in calcareous layers of Ganges River Floodplain soils. Soils near the coast become somewhat saline in the dry season, and there are some areas of toxic Acid Sulphate Soils in the same zone.

By tropical standards, most floodplain soils can be regarded as fertile: i.e., they either have a high natural fertility, or they respond satisfactorily to the application of fertilizers. This fertility is attributable to both mineral and biological factors:

- reactive clay minerals such as illite and, in Ganges River Floodplain soils, montmorillonite;

- a high content of easily weatherable sand minerals (such as biotite and feldspars), except in piedmont alluvium derived from hill rocks and soils;

- in seasonally flooded soils, nitrogen added by blue-green algae (and other organisms) together with organic matter added by weeds and crop residues decomposing under water; (new alluvial deposits themselves and little quickly-available fertility).

LAND

Per Capita Availability

Land is a key element of crop production and is a scarce and limited natural resource in Bangladesh. The amount and quality of its land resource determine the crop production potentials of the country. There is little or no scope for bringing any new land under cultivation, and a very unfavourable land-man ratio already exists in Bangladesh. Except for limited natural and man-made accretion of new lands in the coastal area, nearly all available arable land is presently cropped. Culturable waste accounts for only 2.6 percent of total net cultivable area. The land-man ratio will decline alarmingly even at present cropping intensity because of increasing population on one hand and static or decreasing cultivable land area on the other. Improvement of land productivity is the only way to increase farm income and minimize the potentially growing food production gap. Improvements in production per unit area and higher cropping intensities are the means to increase land productivity.

The reduction in cropped area per person over the three decades is given in Table 3. Also see Figure 9 for reduction in net cultivable area per person.

Table 3
Cropped area per person
(1950 to 1985)

Year	Cropped Area Per Person (ha)
1944 - 50	0.23
1969 - 70	0.19
1978 - 79	0.15
1984 - 85	0.13

Land Holdings and Landlessness

The Census of Agriculture and Livestock of 1983-84 provides valuable information on who the farmers are. Out of 13.8 million rural households, 10 million owned at least 5 decimals (0.05 acre) of land and were classified as " farm households". Of the 27.3% of non-farm households who owned no cultivated land (not counting homestead land), some are actually tenant farmers or livestock owners. More than two-thirds of the rural households are fully or "functionally" landless: about 9% had no land, around 20% owned a homestead but no cultivated land, and about 40% had a homestead and up to 1 acre. These figures refer to <u>operated</u> land. About 22% per cent of the rural households owned 2.5 acres of land or more, and only farms of that minimum size are supposed to be large enough to provide sufficient income large enough to support a family using traditional farming methods. Farmers with between 1 and 2.5 acres of land may not be functionally landless, but they must find supplementary income to survive.

Between the 1960s and 1980s, owing mainly to pressure of population, average farm size declined from 1.45 ha to 0.90 ha and the proportion of households that are landless rose from about 35 percent to 45 percent. Over the next twenty years, it is likely that the average farm size will drop to 0.6 ha and the proportion of rural households without land will increase to nearly 60 per cent 1/.

One form of supplementary income is wage labour for other farmers. Almost two-thirds of all "non-farm" households are counted as agricultural labour households. 41% of the small farm households, i.e. with less than 2.5 acres of land, seek wage income. Even some of the farmers with farms above 2.5 acres take outside jobs as agricultural labourers. As the demand for labour in crop farming cannot be expected to increase by more than 200-300,000 man-years per year at best, even on optimistic assumptions, it is likely that the proportion of households which will find work as agricultural labour may decrease.

1/ Bangladesh Action Plan for Flood Control, World Bank, December, 1989, Pg. 34.

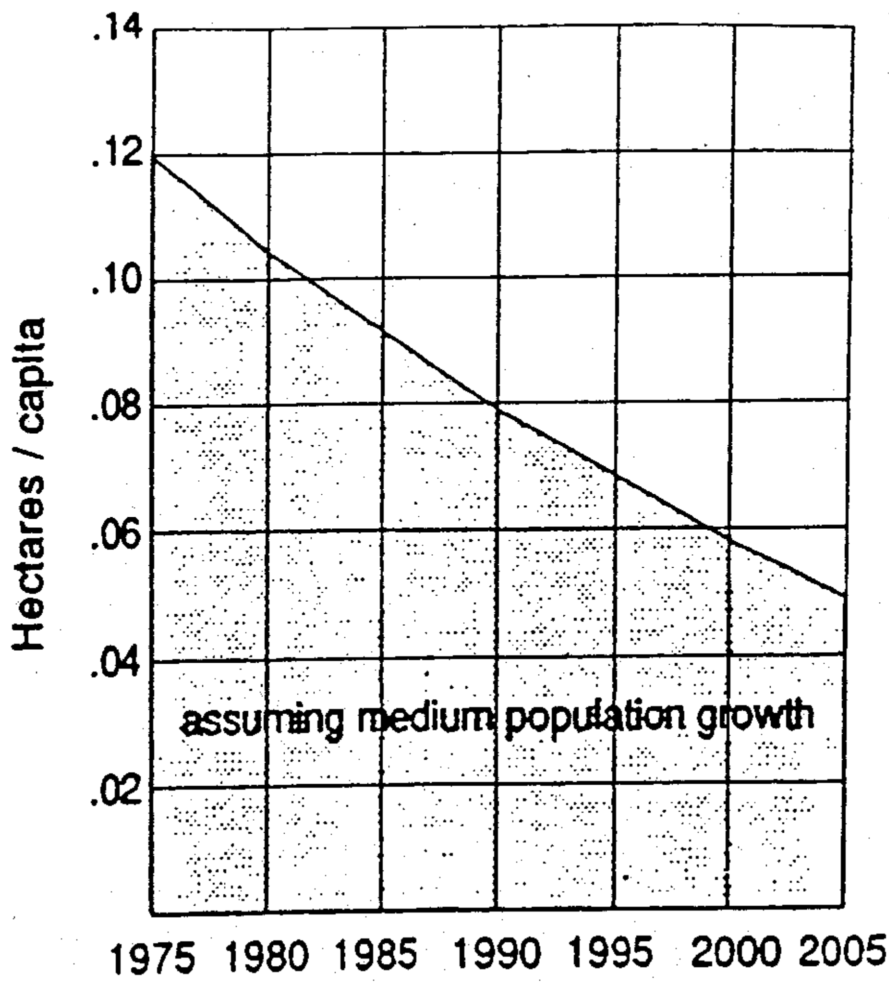


Figure 9 Net Cultivable Area Per Capita

The growth of population is so large and rapid that it has had a profound effect on the availability of land and land ownership. The agricultural censuses show that the average size of all categories of farm holdings have been declining. Also the number of large and medium-sized farms is shown to have fallen, in spite of purchases of land by large farmers from the small land owners. Population growth has also led to overcrowding of the homestead and, a decline in the horticultural and bamboo area surrounding homesteads and also has led to ownership problems with fish ponds. Increased human population has also put pressure on the livestock population. Many holdings are falling below viable size and less feed is available for livestock. Landlessness and "functional" landlessness is also increasing because small farmers more frequently are obliged to sell parts of or their entire land to larger farmers in order to meet basic expenditure or to buy inputs for cultivation.

Small farmers often lose their land also by mortgaging it because of their continuing poverty or because of natural calamities such as river bank erosion or flooding. To the number of landless and near-landless in rural areas must be added the large number of landless who have moved to towns in an often vain search for employment.

Land Utilization

Current land utilization is given in Table 4 and Figure 10 for the five regions of Bangladesh and for the whole country. Currently 67 percent of the land area of the nation is cultivated and 15 percent is in forest. As the substantial growth of population and urbanization continue the fraction not available for cultivation, now about 17 percent, will grow. This will add to the need to increase productivity of the agricultural land.

Table 4

Land Utilization by Region, 1984/85

(Thousand hectares)

		Northwest Region	Northeast Region	Southeast Region	South Central Region	Southwest Region	Active Flood Plain	Bangladesh
1.	Total land area	3,054	3,450	3,125	1,387	2,575	676	14,267
2.	Not available for cultivation	523	630	457	315	353	127	2,405
3.	Forest	13	170	1,303	30	552	-	2,068
4.	Culturable waste	67	77	52	17	4	16	233
5.	Current fallow	92	170	131	20	70	30	513
6.	Net cropped area	2,359	2,403	1,182	1,005	1,596	503	9,048
7.	Net cultivable area [a]	2,451	2,573	1,313	1,025	1,666	533	9,561

[a] Net cropped area and current fallow land added to obtain net cultivable area based on Upazila maximum cultivable area in any one year during the 8 year period (1974/75-1982/83).

SOURCE : MPO computation based on BBS Upazila Statistics.

Distribution of Land

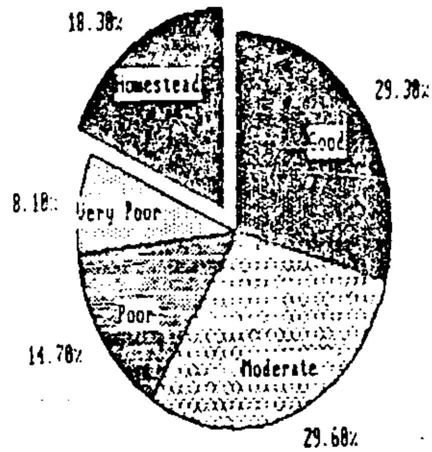
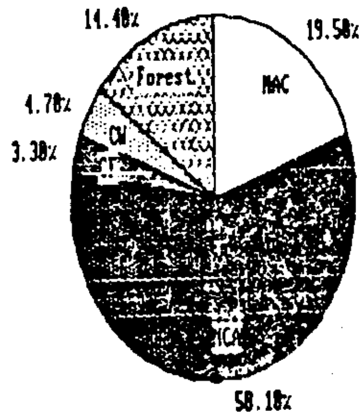


Chart by Shamsuddin Ahmed

Data Source: The Reconnaissance Soil Survey Reports

Land Utilization



Net Cropped Land
Distribution

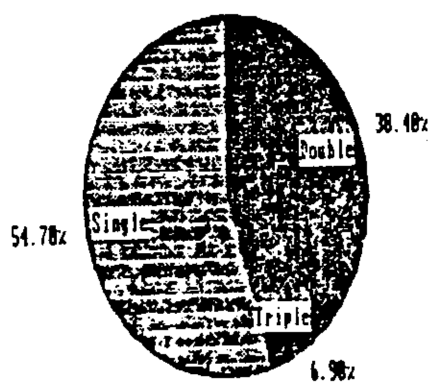


Chart by Shamsuddin Ahmed
Data Source: BBS 1984-85 Annual Statistics

Figure : 10

CROPS

Introduction

With a climate favourable for the cultivation of a wide variety of crops, nearly 100 different crops are presently grown in Bangladesh. Thirty two of those which cover 96 percent of the total cropped area have been considered by MPO in the NWP. These have been grouped into 16 crops (7 rice and 9 others) based on their importance and similarity in terms of input requirements and water regime. The 16 crops are: Broadcast Aus and Aman rice (B. Aus, B. Aman), high yielding varieties of summer rice (HYV Aus, Aman), winter rice (HYV Boro), high yielding wheat (HYV wheat), local varieties of transplanted rice (L.T Aman, L.T. Boro), potato, jute, sugarcane, pulses, oilseeds, spice, minor crops, and orchards.

Rice Varieties

There are three important types of rice-Aman, Aus, and Boro-grown in the country. Aman is mainly transplanted, but approximately a third is broadcast. It is sown in May-June and harvested in November to January. It occupies about 50 percent of rice land and contributes about 44 percent of the crop. The yield per hectare is about 1,350 kilogram (kg). Aus is transplanted onto higher ground as well as broadcast onto medium highland. It is sown in March-April and harvested in late June through August. It is sown on about 26 percent of the rice land and it contributes about 18 percent of the crop. The yield per hectare is 1,063 kg. Recently, Boro, particularly high yielding varieties, has become a very important rice crop in the country. It is sown in November-January on low lying areas or in marshes and swamps and the crop is ready for harvesting in April-May. About 24 percent of the rice land is under Boro. It contributes 38 percent of the rice crop of the country and the yield per hectare is about 2,400 kg. Rice crop calender is provided in Figure 11.

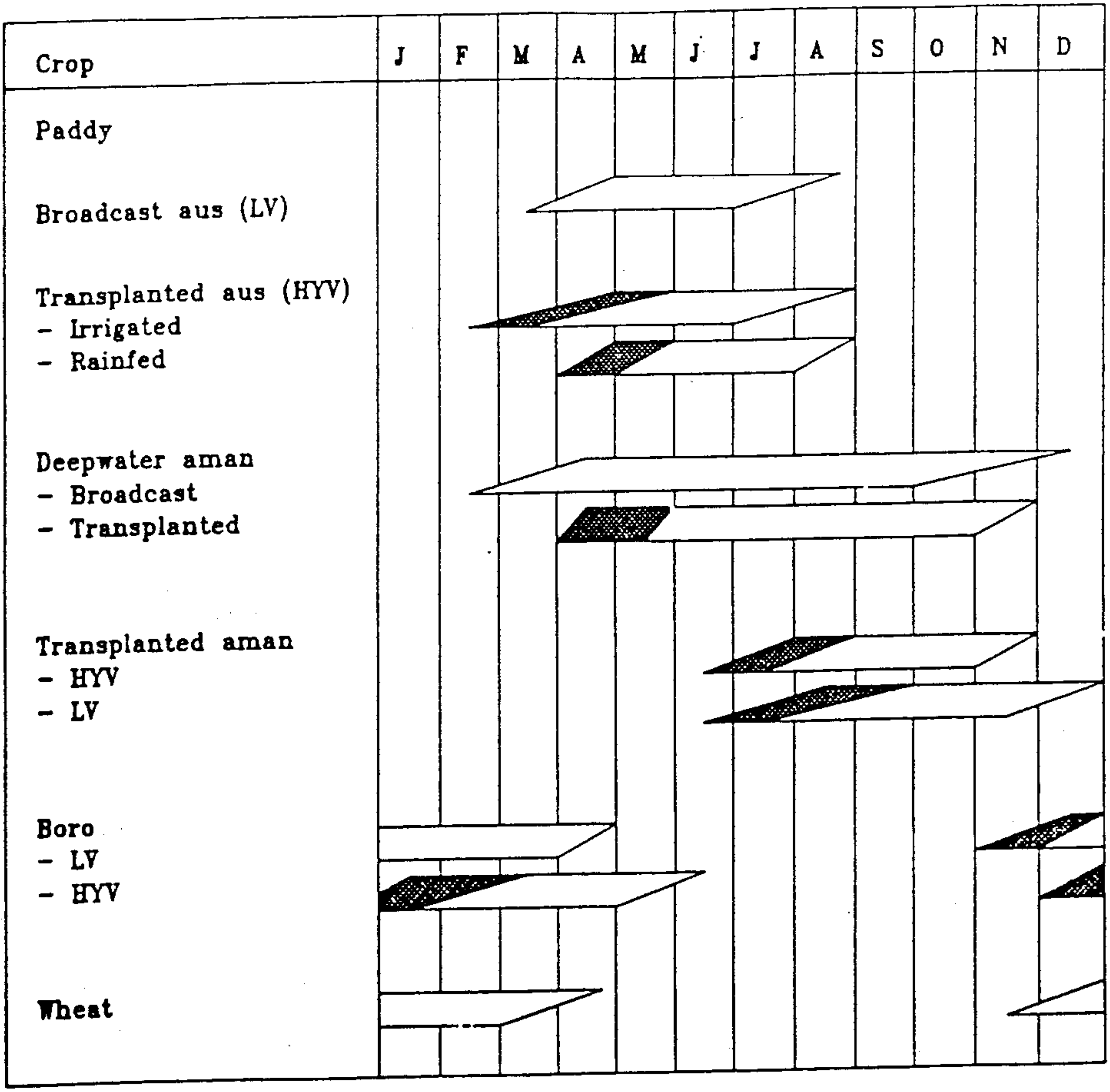


Figure 11 Cereals Crop Calendar

N.B. LV = local variety. HYV = high yielding variety.
Range of sowing dates Seedbed period Range of harvesting dates
Broadcast aus, deepwater aman and transplanted aman are mainly grown without irrigation.
50 percent of wheat and over 90 percent of boro are grown with irrigation.

Other Crops

In recent years wheat has become an increasingly important crop in the country. It is grown mostly in the northern districts. Some wheat is also grown in the northern area of the Southwest region in Kushtia and Faridpur. It accounts for only 3 percent of the volume of annual foodgrain harvest at the present time, but its potential for growth is felt to be very large. According to one recent survey of the agricultural situation in Bangladesh, increased use of dry season wheat growing "is one of the greatest challenges and promises of agriculture in Bangladesh."

Jute is the most important commercial crop of the country. Jute and jute-based products continue to supply most of the export earnings of Bangladesh. Jute is grown from late February to September. There are eight leading jute producing districts, which together form the jute belt of the country; Mymensingh, Rangpur, Comilla, Dhaka, Faridpur, Rajshahi, Pabna, and Bogra. Bangladesh once had almost a monopoly on production of jute, but now its contribution to the total world jute production is much reduced.

Other commercial crops are sugarcane, tobacco, tea, and cotton. The major sugarcane areas are the northern districts of Rajshahi, Dinajpur, Rangpur, and Pabna in the NW region, Kushtia and Faridpur in the SW Region, and Mymensingh and Dhaka in the NE region. Sugar processing factories are also located in these areas.

Tobacco is an important crop. Besides Rangpur, tobacco grows in Mymensingh, Sylhet, Dhaka, Faridpur, Barisal, and Chittagong. Tea is grown on the hilly slopes and uplands of the northeastern district of Sylhet. Since the middle of the nineteenth century it has formed the basis of a substantial industry in the country. It has been extended to Comilla, Chittagong, and Chittagong Hill Tracts.

Once, cotton was an important crop in the country, but production has declined. It used to be produced in Chittagong Hill Tracts. Now cotton is being cultivated in the northern districts of Rangpur, Dinajpur, Bogra of the NW region, Jessore, and Kushtia of the SW region and also in Comilla and Dhaka.

In addition, betelnut and coconut grow widely in the coastal districts of Barisal, Patuakhali, Khulna, Comilla, Noakhali, and Chittagong. Some also grow in Jessore and Kushtia.

Moisture Regime for Major Crops

A number of factors determine the land resource utilization of an area for crop production. The most important of these are:

- Flood depth and duration during the monsoon season;
- Soil moisture storage capacity, particularly during the dry season;
- Capillary rise of groundwater to the soil profile;
- Local relief, soil texture, and permeability; and
- Frequency of sudden rise of water, flash flood, and storm surge.

The depth and nature of flooding determine the crops that can be grown in an area during the monsoon season. The permeability or internal drainage of the soil, the soil moisture available and soil moisture storage capacity largely determine which crops can be grown during the pre-monsoon and Rabi seasons. For instance, Broadcast Aus is mainly grown in high to medium high land (F_0 and F_1) that are not flooded deeper than 90 cm before harvesting in July/August.

Transplanted Aman is planted in poorly drained high to medium high lands (F_0 and F_1) where flooding depth does not exceed 30 cm at the time of transplanting during July/August/September. Broadcast Aman is the main crop in medium low to low land (F_2 and F_3) where flooding is as high as 180 cm during the period of flood. Boro is planted in poorly drained soil or where irrigation can be provided and where no flooding will occur before the harvest of the crop.

Rabi crops are restricted to areas where either residual soil moisture is adequate or irrigation is available. The number of crops grown during the Rabi season is larger than in the Kharif. Prominent crops are Boro rice, wheat, mustard, groundnut, pulses, spices, sugarcane, vegetables, melons, potato, and millet.

Dry land Kharif crops and Rabi crops are planted in areas where soil moisture is adequate, drainage is good and flooding does not occur before the harvest of the crop. The perennial and annual crops, like orchards and sugarcane, are mainly grown on high land (Fo).

Rice, pulses, oilseed, spices and vegetable crops are commonly grown in all areas. These crops have their specific position in the annual cropping systems in the different land types. Of these crops, rice is grown in all three crop seasons though not necessarily in the same area. Jute is grown under conditions where broadcast aus is grown and thus competes with broadcast aus for land.

Soil salinity reduces the yields of transplanted aman in coastal areas such as Khulna and parts of Barisal, Patuakhali, Noakhali and Chittagong. It also limits the area planted to aus and dry land Rabi crops in these areas.

Areas Under Major Crops

The regional land use by the 16 major crops is given in Table 12. Rice is the major crop among the 16 crops; it is grown in all crop seasons and occupies about 82 percent of the total cropped area. The foodgrains, rice and wheat, together occupy about 87 percent of the total cropped area. Among the different groups of rice, L.T. Aman is the largest with 36 percent of the net cultivable area (NCA) followed by B. Aus 25 percent. HYV rice covers 27 percent of (NCA). Forty-nine percent of HYV rice is irrigated including Boro 36 percent and Aus 5 percent. About 68 percent of Boro rice is in high yielding varieties.

MPO cropping intensities for the 60 planning areas are less than the BBS cropping intensities. There are two reasons for this.

First, about 0.38 million hectares under minor crops are not included in the 32 crops. Secondly, MPO's estimated NCA is higher than the BBS reported NCA for any particular year.

Rice, wheat, jute, and tea are the major cash crops of the country. Rice and wheat occupy about 85 percent of the cropped area, yet the country has to import about 10 percent of its cereal requirement.

Table 12

Regional Land Use By 16 Major Crops (1988/89) 1/

(Area in Million Hectares)

Sl. No.	Crop	Northwest Region	Northeast Region	Southeast Region	Southcentral Region	Southwest Region	Total	Percent of NCA
1.	B. Aus	0.621	0.617	0.231	0.314	0.386	2.169	24.0
2.	HYV Aus	0.120	0.173	0.169	0.095	0.038	0.585	6.4
3.	B. Aman	0.221	0.437	0.204	0.203	0.303	1.368	15.1
4.	L.T. Aman	1.224	0.755	0.277	0.547	0.432	3.235	35.8
5.	HYV Aman	0.215	0.322	0.315	0.108	0.078	1.038	11.4
6.	L.Boro	0.017	0.297	0.016	0.016	0.016	0.362	4.0
7.	HYV Boro	0.249	0.454	0.180	0.051	0.076	1.010	11.2
8.	HYV Wheat	0.291	0.071	0.073	0.037	0.134	0.606	6.7
9.	Potato	0.039	0.036	0.021	0.002	0.006	0.103	1.1
10.	Jute	0.160	0.160	0.031	0.047	0.140	0.544	6.0
11.	Sugarcane	0.072	0.022	0.006	0.015	0.040	0.155	1.7
12.	Pulses	0.088	0.035	0.020	0.046	0.075	0.264	2.9
13.	Oilseeds	0.060	0.058	0.038	0.011	0.027	0.194	2.1
14.	Spices	0.039	0.028	0.032	0.025	0.021	0.145	1.6
15.	Minor crop	0.061	0.058	0.050	0.031	0.040	0.243	2.7
16.	Orchard	0.043	0.038	0.032	0.019	0.022	0.154	1.7
	Total	3.523	3.566	1.685	1.567	1.834	12.175	

1/ Projected land use in 60 Planning Areas based on past land use trend.

SOURCE: MPO Technical Report No. 14, Agricultural Production System, 1986.

Crops are selected in each season because of soil-plant-water conditions. Due to high soil moisture or flooding during the monsoon, rice is the predominant crop; many other crops suitable for a high temperature do not tolerate excessive soil moisture. Jute, sugarcane, vegetables and fruits are also grown during the Kharif season. Since more water is available the cropped area is highest during this season. The area occupied by field crops during the rabi and Kharif seasons (1984/85) are shown in Table 6. The area under Kharif crops is 9.26 million hectares as compared to 3.23 million hectares under the Rabi crops. During the Rabi season, 35.7 percent of the total cultivable area (9.56 million hectare) was in production. During the Kharif season, 102.6 percent of the total cultivable area was in production. Of this, 8.40 Mha was in rice (other than boro).

Production Costs for Major Crops

The production costs of major crops are provided in Table 13 and 14. These informations were reproduced from the feasibility study of Horai River Sub-Project 1/

1/ Horai River Sub-Project, Feasibility Report, Northwest Hydraulic Consultants Ltd. IDA Credit 1870-BD, 1990

Table 13

COST OF PRODUCTION FOR CROPS

(Taka/ha)

Crop	Labour		Bullock		Seed		Pesticide		Urea		TSP		MP		Irriga-tion	Total Tk
	person-days	uc	paird/ha	uc	kg/ha	uc	kg/ha	uc	kg/ha	uc	kg/ha	uc	kg/ha	uc	uc	
b aus	146	40	44	39.15	100	9.24	0	438.48	25	6.64	10	10.15	0	8.18	0	9667
lt aus	146	40	40	39.15	30	8.88	0	438.48	40	6.64	10	10.15	0	8.18	0	8039
HYV aus	170	40	46	39.15	30	8.88	1.5	438.48	100	6.64	60	10.15	20	8.18	0	1096
lt aus	140	40	42	39.15	30	8.88	1.0	438.48	40	6.64	20	10.15	10	8.18	0	8500
HYV aman	165	40	44	39.15	30	7.92	1.5	438.48	100	6.64	60	10.15	20	8.18	0	10655
HYV boro	190	40	46	39.15	30	8.88	1.5	438.48	120	6.64	90	10.15	40	8.18	3000	15363
pulses	50	40	30	39.15	30	21.75	0	438.48	0	6.64	0	10.15	0	8.18	0	3827
sweet potato	180	40	46	39.15	1000	7.40	0.5	438.48	30	6.64	20	10.15	10	8.18	0	17104
vegetables	250	40	50	39.15	0.3	348.0	0.5	438.48	60	6.64	40	10.15	20	8.18	0	13249
spices	150	40	40	39.15	3	522.0	1.8	438.48	40	6.64	25	10.15	25	8.18	0	10645

Table 14

LABOUR INPUT FOR CROPS (PERSON DAYS PER HECTARE)

Crops	Prepare Land	Apply Fertilizer	Seed or Transplant	Weed	Apply Pesticide	Irrigate	Harvest	Post Harvest	Total
b aus	40	2	2	67	0	0	20	15	146
lt aus	40	0	40	26	0	0	30	10	146
HYV aus	40	2	40	51	2	0	20	15	170
lt aman	30	2	40	20	1	0	30	17	140
HYV aman	40	2	40	36	2	0	30	15	165
HYV boro	40	6	30	57	2	10	25	20	190
pulses	20	0	0	0	0	0	20	10	50
sweet potato	60	2	40	38	2	0	30	8	180
vegetables	80	2	26	70	2	0	50	20	250
spices	40	2	35	33	0	0	25	15	150

Land Classification Based on Flood Depth

The main factors determining cropping patterns and crop yields on the floodplains of Bangladesh are the monsoon climate, soil properties, the depth, timing and duration of floods, salinity (in coastal areas), the presence or absence of irrigation, access to markets and farm management levels. Local variations in soils, in the timing and duration of seasonal flooding and in the provision of irrigation often lead to complex cropping patterns. Crop yields and production vary from year to year depending on the sufficiency of pre-monsoon and post-monsoon rainfall for non-irrigated crops, the incidence of untimely or high floods and, for dryland rabi crops, the time of recession of floodwater from the land and the incidence of rainfall during the dry season.

Six million ha are subject to annual inundation ranging from 30 cm to in excess of 2 m. The land resources of Bangladesh have been classified into five land types on the basis of flood depth (Table 15).

Table 15

Land Types Defined On The Basis Of Flood Depth

Land Types	Description	Flood Depth	Nature of Flooding
F0	High land	Not flooded	Intermittent or flooded up to 30 cm
F1	Medium	30 to 90 cm high land	Seasonal
F2	Medium	90 to 180 cm lowland	Seasonal
F3	Lowland	Over 180 cm	Seasonal (<9 months)
F4	Low/very lowland	Over 180 cm	Season(>9 months) or perennial

Cropping Pattern

Table 16 shows the main cereal cropping patterns by depth of flooding and land types (under normal flooding) and the changes that can occur with irrigation. The impact of flood control and drainage is to reduce the flooding depth and to make it possible to grow higher yielding cropping patterns.

Table 16

Main Cereal Cropping Patterns

Land type	Early kharif	Late kharif	Rabi
Non-Irrigated			
F0	B Aus	- [1]	Wheat
	HYV Aus	LV T Aman	-
F1	B Aus	LV T Aman	-
	HYV T Aus	LV T Aman	-
F2	B Aus	mixed with B Aman	Wheat
	B Aus	mixed with B Aman	-
F3	-	B Aman	-
F4	-	-	LV Boro
Irrigated			
F0	-	HYV T Aman	Wheat
	-	HYV T Aman	HYV Boro
F1	-	LV T Aman	Wheat
	-	LV T Aman	HYV Boro
F2	-	T Aman [2]	HYV Boro
	-	-	HYV Boro
F3	-	-	HYV Boro
F4	-	-	LV Boro

Notes: [1] Fallow. [2] Transplanted deep water aman.
Abbreviations: B = broadcast; T = transplanted; LV = Local variety; HYV = high yielding variety.

On non-irrigated land, paddy yield and overall output generally decline with increasing depth of flooding (Table 17). Yields of transplanted rice normally exceed those of broadcast rice. Farmers on F0 and F1 land with permeable soils generally grow broadcast aus or jute followed by rabi crops; on impermeable soils, they grow local or HYV transplanted aman, preceded in wetter parts of the country by broadcast or transplanted aus. On F2 land, mixed aus and aman or jute is grown followed by rabi crops (eg wheat, oilseeds, pulses). On F3 land, farmers grow deepwater broadcast aman which may be followed by rabi crops on the higher parts. Local boro is the only crop grown on F4 land. Two or three crops a year are grown on F0 and F1 land in the wetter eastern and central districts, on F2 land and on the higher part of F3 land.

Table 17

Gross Value of Output per hectare by land Type [1] (Taka)

Land type	Irrigated	Non-irrigated
F0	35 500	19 100
F1	29 100	20 200
F2	28 100	15 500
F3	24 100	8 600
F4	12 900	11 300

Note: [1] National Water Plan estimates in 1989 prices. Average of two main cropping patterns on each land type, irrigated and non-irrigated.

On irrigated land, boro paddy is the principal crop on impermeable soils on all lands types (F0 to F4). High yielding varieties of rice (HYVs) are grown, except in some depressions subject to early flooding, where traditional boro varieties continue to be grown. On impermeable F0 and F1 soils, HYV boro is generally followed by transplanted aman (HYV or local, rainfed or irrigated). Elsewhere, irrigated boro has displaced some rainfed aus and jute on F1 and F2 land and deepwater aman on F2 and F3 land. On permeable F0 to F2 soils, wheat, potato, vegetables and sauces are the principal crops grown with irrigation.

Floodplain cropping patterns are often complex. Most floodplain villages include several soil and land types and farmers' fragmented holdings lie scattered across them. Additionally, farmers generally grow several crop varieties in order to suit local micro-environments, to reduce risks of crop damage and to spread their labour use and market opportunities.

Flood control and drainage can be provided for two alternative purposes. Where pump or tidal drainage can be provided, the objective is to reduce the depth of flooding so as to convert the greater part of the protected area to F0 and F1 land and thereby enable intensive cropping practices to be used in the rainy season (and, with irrigation, in the dry season also). Elsewhere, with 'controlled flooding', the objective is to eliminate high and untimely floods so as to provide greater security of crop production in the rainy season under 'normal' flooding conditions. Embankments (including submersible embankments) are needed in some areas to protect boro paddy from early floods.

The main cropping patterns on floodplain land types are presented in Table 18.

Table 18

Main Cropping Patterns On Floodplain Land Types

Land/Soil Type	Main Cropping Patterns Without Irrigation	 With Irrigation
F0 (not flooded + <30 cm flooding)		
- Permeable	(Not flooded): tree crops, sugarcane: kharif vegetables, spices, fruits. B.aus-rabi crops. Jute/mesta-rabi crops	Similar to non-irrigated, but greater emphasis on rabi cash crops(vegetables,spices, tobacco, cotton); also wheat
	(1-30 cm flooding):ditto, but no tree crops	Ditto, but no tree crops
- Impermeable	HYV/LV t.aman HYV/LV t.aus-HYV/LV t.aman	HYV boro/aus-HYV aman
F1 (30-90 cm) flooding		
- Permeable	B.aus/jute-rabi crops B.aus/jute LV t. aman	Similar to non-irrigated, but more emphasis on wheat & rabi cash crops (as on permeable Fo soils)
- Impermeable	HYV/LV t.aman HYV/LV t.aus-HYV/LV t.aman	HYV boro/aus-(non-irrigated) LV t.aman
F2 (30-180cm) flooding		
- Permeable	Mixed b.aus+b.aman - rabi crops Jute - rabi crops	Similar to non-irrigated, with more emphasis on wheat,spices, potato
Impermeable	Ditto, but less jute	HYV boro, locally preceded by non-irrigated mustard and/or followedby transplanted deep-F3 water aman.
(180->300 cm flooding for <9 months)		
-Permeable+ -impermeable	B.aman - rabi pulses, oilseeds B.aman - rabi follow	HYV boro
F4 (flooded for >9 months, mainly >300 cm deep)	LV boro (in perennially wet depressions) Grassland/reed swamp	Boro (mainly LV)

NB: See table 9 for definitions of FO-F4 land types.

HYV	=	high yielding varieties	LV	=	local varieties
B	=	broadcast	t.	=	transplanted
kharif	=	rainy season	Rabi	=	dry season
/	=	or			

Flooding Depth Tolerance of Rice and Jute Crops

Different crops and varieties are sensitive to different depths of flooding at different growth stages, as shown in Table 19 Broadly speaking, regulators and sluices need to be managed so that flooding depths on relevant land types do not exceed the depths indicated for individual crops in particular months. Where such control cannot be effected, it may be impossible to grow a particular crop or crops reliably on the land type(s) indicated without further improvement of drainage.

The achievement of sustained high crop yields on land subject to controlled flooding will require the adoption of recommended agronomic practices, especially balanced use of fertilizers. Provision of supplementary irrigation for kharif crops would be advantageous, especially on relatively higher land which may be less deeply flooded than formerly or may remain unflooded under protected conditions. Irrigation would enable kharif crops to be sown at the optimum time and would eliminate the risk of yield reductions due to drought in the pre-monsoon, monsoon or post-monsoon seasons. Eventually, it may become necessary to provide pump drainage in addition to flood protection and irrigation in order to maximise production from low-lying floodplain land.

Table 19

Maximum Flooding Depth Tolerance by Rice and Jute

Crops by Land Types and Month

		Month/maximum flooding depth (cm)						
Land Type	Crop	Apr	May	Jun	Jul	Aug	Sep	Oct
Medium Highland	HYV boro	30	30	30				
	LV boro	50	50					
	Broadcast aus	0-15	0-30	90	50			
	Transplanted aus	15	15-30	30	30	30		
	Jute	0-15	0-30	90	90			
	HYV t aman				15	15-30	30	50
	LV t aman				30	30	30-50	60
Medium Lowland	Broadcast aus	0-15	30	90	50			
	Jute	0-15	15	90	90			
	Deepwater aman	0-15	30	90	90-180	90-180	90-120	90
	HYV boro	30	30	30				
	LV boro	50	50					
Lowland + Very Lowland	Deepwater aman	0-15	30	90	90-300+	90-300+	90-300+	90-180
	HYV boro	30	30	30				
	LV boro	50	50					

N.B: 1. Depth tolerances are general : individual rice and jute varieties have different flood tolerances.

2. Land for broadcast aus, aman and jute should not be submerged at the time of sowing and emergence.

3. Flood tolerance of broadcast aus, jute and deepwater aman in May and June depends on sowing date in March-April (even May), which is determined by the time of onset of pre-monsoon rainfall.

4. Many varieties of deepwater aman exist, suitable for flooding depths between 90 cm and 4m. Floodwater should not rise by more than 20cm/day in May-August, and depth should not increase further after end-August.

5. Flood tolerance of HYV and LV transplanted aman in July-September depends on date of transplanting (HYV July-August; LV July-September).

Regional Distribution of Cultivable Land Types

The country has been divided into six regions based on differences in flooding characteristics (see Figure 12 Table 20 indicates the considerable differences which exist between regions in the distribution of land of different depth of flooding types. These regional differences, together with differences between them in annual rainfall, the length of the rainy season and the length of the cold winter season, considerably influence regional cropping patterns and potentials. Important differences between regions are outlined below.

Table 20

Distribution of Cultivable Land Types and Region
(Area in Mah)

Region	Cultivated Area	Fo	F1	F2	F3	F4
North West	2,451 (100)[1]	1,307 (53)	0,797 (33)	0,194 (8)	0,153 (6)	--
North Central	0,909 (100)	0,308 (34)	0,267 (18)	0,168 (18)	0,160 (1)	0,006
North East	1,664 (100)	0,481 (29)	0,301 (18)	0,361 (22)	0,508 (30)	0,013 (1)
South West	1,666 (100)	0,545 (33)	0,713 (43)	0,281 (17)	0,120 (7)	0,007 (0)
South East	1,313 (100)	0,386 (29)	0,474 (36)	0,300 (23)	0,134 (10)	0,019 (2)
South Central	1,026 (100)	0,234 (23)	0,599 (58)	0,128 (13)	0,032 (3)	0,033 (3)
Active Flood Plain	0,533 (100)	0,253 (47)	0,137 (26)	0,126 (24)	0,017 (3)	--
Bangladesh	9,562 (100)	3,514 (37)	3,288 (34)	1,558 (16)	1,124 (12)	0,076 (1)

Note: [1] Figures in parentheses indicate percentage of cultivable area in the region.

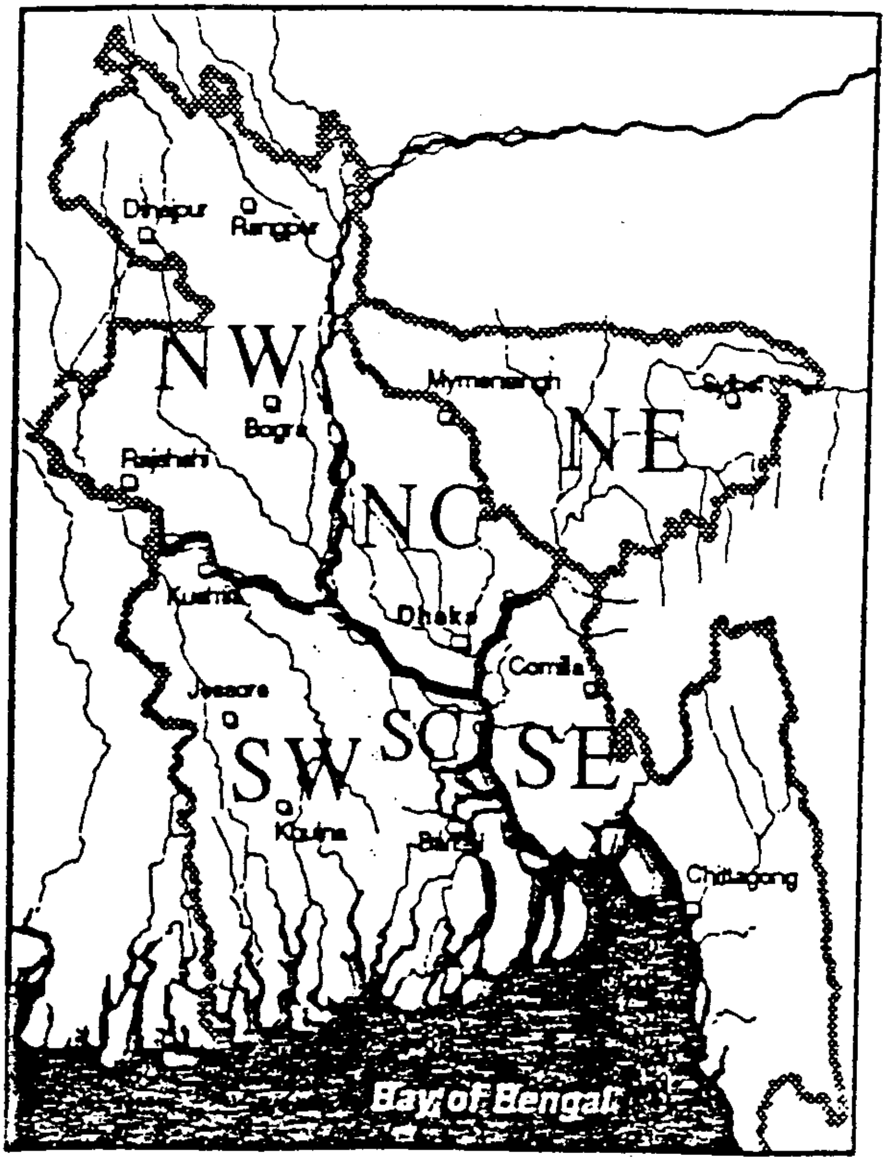


Figure 12. Regional Units

The north-west (NW) region comprises the area between the Brahmaputra-Jamuna and Ganges rivers. It includes 13 sub-regions: the active Teesta, Brahmaputra and Ganges floodplains; the high and low Ganges meander floodplains; the Teesta and Karatoya-Bangali floodplains; the old Himalayan piedmont plain; the lower Atrai basin (Chalan Beel); and the Barind Tract. About 86 per cent of the cultivable land is Fo and F1 land. The main flooding problems in the NW are caused by river floods on the active Teesta, Brahmaputra-Jamuna and Ganges floodplains, by the backing up of water in the low-lying Atrai basin and by flash floods on the old Himalayan piedmont plain and the Teesta and Atrai floodplains. Breaching of the Brahmaputra right bank embankment can also cause sever flooding on the Karatoya-Bangali floodplain and parts of the Teesta floodplain. Transplanted aman is the principal crop affected, but boro, aus and deepwater aman paddy and jute are occasionally damaged by early floods. About 29 per cent of the NW region is irrigated, predominantly by tubewells.

The north-central (NC) region comprises the area between the Old Brahmaputra, Jamuna and meghna rivers. It includes nine sub-regions; the active and meander floodplains of the Jamuna, the Old Brahmaputra and Ganges rivers; the low lying Arial beel; and the predominantly upland Madhupur Tract. FO and F1 land together occupy about 63 per cent of the cultivable land and F3 and F4 land 19 per cent. the main areas subject to damaging floods are the active river floodplains, the Ganges meander floodplain and the southern part of the Jamuna floodplain. Transplanted aman is the principal crop affected by floods in north; in the center and south, aus, deepwater aman and boro paddy and jute are affected by early floods about 40 per cent of the NC region is irrigated, partly by low-lift pumps and traditional methods, partly by tubewells.

The north-east (NE) region comprises three main subregions: extensive piedmont floodplains in the north and east; the low-lying sylhet basin in the center; and part of the Old Brahmaputra floodplain in the west. F0 and F1 land occupy 47 per cent of the cultivable land, F3 and F4 31 per cent. The main flooding problems are caused by flash floods in the north and east, deep and sometimes early flooding of the sylhet basin, and both river and rainwater floods on the Old Brahmaputra floodplain. Flash floods can damage boro, aus, deepwater and transplanted aman, depending on the season in which they occur. Early floods damage boro paddy in the Sylhet basin and on the Old Brahmaputra floodplain. About 22 per cent of the NE region is irrigated, mainly by low-lift pumps and traditional methods.

The south-west (SW) region comprises the area south of the Ganges and Padma between the western border and the Arial Khan and Swarupkati rivers. It includes five subregions: parts of the Ganges active, meander and tidal floodplains and of the Gopalganj-Khulna beels. F0 and F1 together comprises 76 per cent of the cultivable land in the region, and F2 land 17 per cent. Flooding problems are caused mainly by drainage congestion in interior floodplain areas and over-bank spill from the Ganges. Additional problems are provided by dry-season salinity and exposure to cyclones and storm surges in the south, by perennially wet peat areas in the Gopalganj-Khulna beels, and by relatively droughty conditions in the north. Transplanted aman is the main crop affected by floods. About 16 per cent of cultivable land in the SW region is irrigated, partly by gravity canals in the Ganges-Kobadak project, elsewhere by low-lift pumps or tubewells.

The south-central (SC) region comprises the area west of the Padma and low Meghna rivers as far as the Arial Khan and Swarupkati rivers. It includes six sub-regions; parts of the Ganges active, meander and tidal floodplains, of the old and young Meghna estuarine floodplains and of the Gopalganj-Khulna beels. F0 and F1 land together comprise 81 per cent of the region, and F2 land 13 per cent. Flooding problems are caused mainly by river flooding along the Ganges-Padma channel and by drainage congestion in the interior. Additional problems are caused by exposure to cyclones and storm surges in the south, dry-season salinity in the extreme south and by perennially wet peat areas in the Gopalganj-Khulna beels. About 10 per cent of cultivable land in the SC region is irrigated, mainly by low-lift pumps in the south (eg., in Barisal Irrigation Project) and to some extent by tubewells in the north.

The south-east (SE) region comprises the area east of the middle and lower Meghna rivers to the eastern border and the coast. It includes eight sub-regions: old and young Meghna estuarine floodplains; middle and lower Meghna river floodplains; the complex Chittagong coastal plain; piedmont plains; and the eastern hills. Excluding the hill areas, F0 and F1 land occupy 65 per cent of the cultivable area and F2 land 23 per cent. The main flooding problems are caused by flash floods in areas adjoining the eastern hills, which may damage boro, aus or transplanted aman depending on their time of occurrence. River floods occasionally damage aus and deepwater aman on the Meghna river floodplains and local flooding damages crops on the Meghna estuarine floodplain. Storm surges occasionally damage transplanted aman, more rarely aus, in coastal areas. About 40 per cent of the cultivable land in the region is irrigated, mainly by tubewells, but partly by low-lift pumps in the Chandpur Irrigation project and on the Chittagong coastal plain and by gravity irrigation in the Meghna-Dhonagoda Project.

Cropping Intensity

In general, cropping pattern indicate the crops grown in sequence on a piece of land in a year. Thus B. Aus-L.T. Aman-Pulse cropping pattern means that on a piece of land B. Aus was grown during kharif-I season and after harvesting B. Aus, L.T. Aman was grown in Kharif-II season and thereafter pulses was grown in the rabi season on the same piece of land harvesting L.T. Aman. This is a triple cropping pattern as it involves three crops in a year. Double cropping pattern include two crops grown in sequence in a year on a piece of land, viz B. Aus-L.T. Aman meaning that B. Aus was grown in Kharif-I season and after harvesting B. Aus, L. T. Aman was grown in kharif-II season on that land which remained fallow during the rabi season. Single cropping pattern indicate the name of the crop grown during the year on a piece of land, e.g. B. Aman.

Irrigation increases the cropping intensity in all categories of land (F0 to F4), see Table 21. It is essential to get acquainted with the following terminology which are closely associated with cropping pattern.

Mixed Cropping : Seeds of two crops are sown together on a piece of land. Harvesting of each crop is done separately when the individual crop attain maturity, e. g. mixed Aus-aman. Seeds of aus and B. Aman are sown together in March-April. Aus rice attain maturity in about 100 days and is harvested during June-July. The Photosensitive Aman rice is harvested in November.

Inter Cropping : Seeds of the succeeding crop are sown before harvest of the previous crop, e.g. Pulses in T. Aman. Pulse seeds are sown in T. Aman field during reproductive phase. Germination and seedling emergence of pulse take place when the T. Aman is still in the field. The T. Aman is harvested after which pulses continue to grow.

Cropping Intensity : Cropping intensity expressed in percent indicate the number of crops grown throughout an year and is computed using the following equation;

$$CI = \frac{TCA}{NCA} * 100$$

Where CI = Cropping Intensity

TCA = Total Cropped Area.

NCA = Net Cultivated Area.

Table 21

Cropping Pattern of Bangladesh (1984-85)

Based on Land Category

Existing Cropping Pattern in Percentage

Land Type		1 B.L Aus	2 HYV Aus	3 B. Aman	4 L.T Aman	5 HYV Aman	6 L. Boro	7 HYV Boro	8 HYV Wheat	9 Potato	10 Jute	11 S.Cane	12 Pulses	13 Oil Seed	14 Spices	15 Minor Crop	16 Orchard	Intensity %	Net Cultivable (ha)
Fo	Non-irrigated	39	4	0	38	12	0	0	2	0	6	4	2	1	1	1	5	115	2649930
Fo	irrigated	5	31	0	29	52	0	32	19	3	1	2	2	2	3	2	0	183	611740
F1	Non-irrigated	36	3	8	60	8	0	0	7	1	10	1	4	3	2	2	0	145	2603900
F1	Irrigated	3	24	0	54	31	0	49	19	5	2	0	1	1	3	5	0	197	547360
F2	Non-irrigated	22	0	55	8	0	0	0	8	1	6	0	4	4	0	1	0	109	1126270
F2	Irrigated	0	0	28	32	0	6	81	8	4	0	0	0	0	1	1	0	161	305670
F3 and F4	Non-irrig.	0	0	51	0	0	15	0	2	0	0	0	2	2	0	0	0	72	711140
F3 and F4	Irrigated	0	0	20	0	0	63	37	0	0	0	0	0	0	0	0	0	120	472690

SOURCE : MPO Technical Report No. 14, 1985.

MPO estimate based on BBS crop and irrigation statistics and BBS and SODAP land area and flood phase information.
39% of Non-irrigated Fo land is BAUS.

DRAINAGE

Introduction

In a delta environment, drainage is extremely important in influencing the water regime and agricultural production.

On medium (F_2) and deeply flooded (F_3) lands, drainage is a key factor influencing the annual crop calendar. The rate of drainage affects crop choice during the post-monsoon period (early rabi-wheat-vegetables) and the planting dates for Boro rice. The risk to Boro and Kharif-I (Aus) crops during the pre-monsoon period is a function of rainfall and drainage capacity.

Highlands and medium highlands (Fo) generally have light textured soils. Often they are over drained, a condition that is frequently worsened when the drainage of lower lands generally makes these lands particularly susceptible to moisture deficits.

Reduction in flood depth converts deeply to moderately flooded land into shallowly flooded land where farmers can replace B. Aman by T. Aman. Likewise the scope of growing HYVs in kharif season are made possible by reducing flood depth on shallowly flooded land. In the moderately to deeply flooded land, delayed entrance of flood water ensure safe harvest of boro rice.

Drainage is an important aspect for crop production. The maintenance of desired moisture regime through drainage is essential for successful crop production. The process normally involve timely removal of rainfall run-off or accumulated water so that soil moisture level remain within tolerance limit of the crop. Drainage requirements vary depending on season and crops and brief discussion for that is given below.

Pre-Monsoon Drainage

Crops grown during this season include B. Aus, Jute, Mixed Aus-Aman and B. Aman. These crops are sown under dryland condition and prefer soil moisture regime around field capacity. As such all water in excess of field capacity need to be drained out at sowing. As the crops grow, their tolerance limit increases. Deep Water Rice (B. Aman) acquire the ability to elongate along with gradual rise of water level in about 6 weeks from sowing and as such need to be protected through drainage during this initial 6 weeks. L. Boro and HYV Boro, planted in rabi season, continue into the pre-monsoon season when they need protection from drainage congestion.

Drainage requirement of different crops during the pre-monsoon season are presented in the Table 22.

Table 22

Pre-Monsoon Season Drainage Requirements

	Desired Moisture Level		
Crops	At Sowing	Early stages	Late Stages
B.Aus	Field capacity	10-30 cm	50-90 cm
Jute(c.olitorius)	Field capacity	Field Capacity	Field Capacity
Jute(c.Capsularis)	Field capacity	Field Capacity	90-150 cm
Mixed Aus-Aman	Field Capacity	30-50 cm	Aus is harvested & the aman elongates with gradual rise of water level upto 4 m.
B. Aman	Field Capacity	30-50 cm	Elongates with gradual rise of water level upto 4 m.
HYV Boro	---	--	30-50 cm upto end May
Local Boro	--	--	50-90 cm upto end April.

Monsoon Season Drainage

Monsoon season drainage, dependent on the water level at out-fall point, is important for the Paddy and is given in Table 23.

Table 23

Monsoon Season Drainage

Tolerable depth of water

Crop	At Transplantion	Later stage	Remarks
HYV Aman	15 - 20 cm	30 - 50 cm	Saturated
L.T. Aman	20 - 30 cm	50 - 90 cm	condition is preferable.

Post-Monsoon Drainage

Post-monsoon season drainage determine the scope of growing rabi crops. Rabi crops namely HYV wheat, potato, pulses, oilseeds, spices, tobacco, vegetables etc. are grown under dry-land condition and prefer moisture regime around field capacity. In order to grow these crops, all water in excess of field capacity must be removed early in the rabi season for timely sowing of seeds. Throughout their growing period also, all surface water must be removed within 24 hours. Local Boro and HYV Boro planted in the rabi season require saturated condition and can even be transplanted in shallow depth of water.

CROP WATER REQUIREMENTS

Introduction

Water is essential for crop production. Crop water requirements include evapotranspiration, water used for land preparation and in the case of puddled field, water lost through percolation by maintaining ponded condition.

Evapotranspiration (ET) includes the total water evaporation occurring from the soil and plant surfaces and the plant transpiration. In addition to ET, plants use small amounts of water in tissue building. The sum of ET and water use in tissue building is called crop consumptive use. Because of the fact that water removed in plant tissue is usually very small compared to ET, the terms consumptive use and evapotranspiration are commonly used interchangeably.

Evapotranspiration of a crop is computed by multiplying reference crop evapotranspiration (ETo) by the crop co-efficient (Kc).

$$ETc = ETo \times Kc$$

Reference crop evapotranspiration (ETo) is defined as "the rate of evapotranspiration from an extensive surface of 8 to 15 cm tall green grass cover of uniform height, actively growing, completely shading the ground and not short of water". ETo values for different locations are given in Table 24.

Crop Co-efficient

The effect of crop characteristics on crop water requirement is given by the crop co-efficient (Kc) which presents the relationship between reference crop evapotranspiration (ETo) and crop evapotranspiration (ETo), i.e.

$$ETc = ETo \times Kc$$

TABLE 24

Average Monthly Evapotranspiration (ETo mm/day)

	Station	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1.	Barisal	2.73	3.60	4.54	5.34	5.44	3.93	3.88	3.68	3.84	3.68	3.26	2.63
2.	Bogra	2.80	3.81	5.27	6.29	5.85	4.46	4.28	4.09	4.10	3.06	3.19	2.63
3.	Chittagong	2.35	4.02	4.93	5.93	5.70	4.42	4.78	4.56	4.52	4.02	3.49	2.99
4.	Comilla	2.75	3.79	4.80	5.49	5.70	4.43	4.36	4.29	4.10	3.79	3.20	2.59
5.	Cox's Bazar	3.99	4.48	5.55	6.23	6.26	4.67	4.46	4.26	4.50	4.39	4.27	3.72
6.	Dhaka	2.87	3.92	5.44	6.25	6.06	4.42	4.66	4.52	4.29	3.87	3.31	3.02
7.	Dinajpur	2.76	3.43	4.71	5.13	5.50	4.33	4.21	4.04	4.05	3.42	2.93	2.36
8.	Faridpur	2.73	3.42	4.70	5.81	5.89	4.06	4.42	4.41	4.23	4.03	4.31	2.62
9.	Ishurdi	2.67	3.61	5.10	6.61	6.30	4.54	4.20	4.11	4.92	3.52	3.15	2.52
10.	Jessore	2.98	3.90	5.41	7.12	6.97	4.65	4.49	4.42	4.09	4.07	3.33	2.73
11.	Khulna	2.85	3.82	4.83	5.39	5.53	3.83	3.82	3.66	3.74	3.86	3.43	2.84
12.	Mymensingh	2.58	3.49	4.47	5.16	5.26	3.99	4.20	4.07	3.99	3.64	3.17	2.51
13.	Noakhali	2.86	3.79	4.77	5.59	5.54	4.15	4.37	4.21	4.18	3.89	3.30	2.68
14.	Rangamati	3.51	4.43	5.73	6.36	6.05	4.48	4.55	4.37	4.48	4.25	4.35	3.14
15.	Rangpur	2.32	3.32	4.36	5.65	5.43	4.44	4.32	4.16	4.10	3.55	2.95	2.35
16.	Sylhet	2.57	3.67	4.71	4.86	4.72	3.55	3.83	3.70	3.40	3.54	3.10	2.57
17.	Satkhira	2.68	3.65	4.72	5.35	5.77	3.96	3.82	3.61	3.74	2.38	2.38	3.30
18.	Narayanganj	2.87	3.83	5.51	5.69	5.86	4.39	5.57	4.60	4.25	4.22	4.44	2.95

SOURCE : Karim and Akand 1982; See also MPO Second Interim Report, Vol-Vi, June, 1984.

Crop co-efficient (Kc) vary from crop to crop and from growth stage to growth stage in individual crops. Crop co-efficients for some irrigated crops at different growth stages are given Table 25.

Table 25

Crop Co-efficients for Some Crops

Crops	Initial Stage	Mid season	Late season
HYV Aus	1.10	1.25	1.0
L.T. Aman	1.10	1.05	0.95
HYV Aman	1.10	1.05	0.95
L. Boro	1.10	1.25	1.0
HYV Boro	1.10	1.25	1.0
HYV Wheat	1.0	1.05	0.75
Potato	1.0	1.05	0.95
Tobacco	1.0	1.2	1.0

Percolation Losses

Downward movement of water through the soil profile take place from ponded field till field capacity condition is reached. Thus percolation losses need to be considered in crops grown under wetland condition while it does not happen in case of crops grown under dryland condition.

The rate of percolation vary depending on soil properties, the most important of which is soil texture. Approximate rate of percolation in different soil textures are presented in Table 26.

Table 26

Percolation Rates

Soil Texture	Percolation rate (mm/day)
Clay	0.89
Silty clay	1.74
Sandy clay	1.75
Silty clay loam	1.75
Clay loam	2.00
Sandy clay loam	2.25
Silty loam	3.00
Sandy loam	4.00
Loam	5.0
Loamy sand	5.7
Sand	13.7

RAINFALL

Dependable Rainfall

Crop water needs can be fully or partly met by rainfall. Rainfall for each period will vary from year to year and therefore, rather than using mean rainfall data, a dependable level of rainfall should be selected. One in five year (80%) rainfall data is normally considered while computing crop water requirements.

Effective Rainfall

Not all rainfall is effective and part may be lost by surface runoff, deep percolation or evaporation. Only a portion of heavy and high intensity rains can enter and be stored in the root zone and the effectiveness is consequently low. Frequent light rains intercepted by foliage with full ground cover are close to 100 percent effective. With a dry soil surface and little or no vegetative cover, rainfall upto 8 mm/day may all be lost be evaporation; rains of 25 to 30 mm may be only 60 percent effective with a low percentage of vegetative cover.

Irrigation Requirements

To determine agricultural water demands, crops have been divided into the following three categories:

(a) Irrigated crops are grown during periods in which the total water available from rainfall and soil mixture is usually insufficient to meet evapotranspiration requirements. The required additional water supplies are referred to as <u>primary irrigation requirements</u> because the crops being considered would not be grown without these supplies.

(b) Rainfed crops are normally grown without irrigation with the expectation that rainfall and soil moisture will be sufficient to meet crop water needs. However, because rainfall is highly variable during the growing season and from year to year, there are periods in which rainfall and moisture available to the crop are less than required. These water deficits induce stress in the plant which reduce yield. The amount of water required to off-set these deficits is called supplemental or secondary irrigation requirement.

(c) Transition season crops such as wheat may be grown during periods of low rainfall because the moisture stored in the soil, the contribution from the water table by capillary flux, and the small quantities of rainfall are frequently sufficient to meet crop water needs and produce an acceptable level of yield. These crops represent a transition between the two groups defined above and could be considered from either point of view.

Examples of these three types of crops are given in Table 27.

Table 27

Representation of Types of Irrigation

Type of Irrigation	Crop Characteristics	Crops
Primary	Dry season wetland	HYV Aus L. Boro HYV Boro
Supplemental	Pre-Monsoon and Monsoon Crops	B. Aus L. T. Aman HYV Aman
Transition	Dryland and Rabi	Wheat

Irrigation Requirement Computation

Irrigation requirement computations for HYV Boro (primary irrigation) is provided in Table 28.

Table 28.

Irrigation Requirement of HYV Boro

Month/ Decade	ETo (mm)	Kc	ETC (mm)	Per (mm)	Eff. rain (mm)	Net req (mm)	Field req (mm)	Area (ha)	Irrign.Requirement (cumec) Land prep.	Crop req	Total req
Dec.											
21-31								1400	5.74	--	5.74
Jan.											
01-10	20	1.10	22	18	-	40	89	3000	7.22	1.44	8.66
11-20	20	1.10	22	18	-	40	89	3975	4.40	3.09	7.49
21-31	22	1.15	25	19	-	44	98	3975	--	4.10	4.10
Feb.											
01-10	31	1.20	37	18	-	55	122	3975	--	5.61	5.61
11-20	31	1.25	39	18	-	57	126	3975	--	5.79	5.79
21-28	25	1.25	31	14	-	45	100	3975	--	5.75	5.75
Mar.											
01-10	45	1.25	56	18	1	73	162	3975	--	7.45	7.45
11-20	45	1.25	56	18	1	73	162	3975	--	7.45	7.45
21-31	50	1.25	63	19	5	77	171	3975	--	7.15	7.15
Apr.											
01-10	52	1.25	65	18	5	78	173	3975	--	7.96	7.96
11-20	52	1.20	62	18	10	70	155	3975	--	7.13	7.13
21-30	52	1.10	57	18	21	54	120	3975	--	5.52	5.52
May.											
01-10	53	1.10	58	18	29	47	104	2575	--	3.10	3.10
1	2	3	4	5	6	7	8	9	10	11	12

Col 2.	Potential evapotranspiration (ETo) has been taken from Table 24.
Col 3.	Crop Coefficient data has been taken from Table 25
Col 4.	Col 2 multiplied by Col 3
Col 5.	Based on the soil texture, percolation rate has been taken as 1.75 mm per day.
Col 6.	One in five year rainfall of Rajbari, Baliakandi and Pangsha stations have been considered for calculating effective rainfall. Water requirement for land preparation has been taken as 175 mm for HYV Boro.
Col 7.	Net Water requirement = Col 4+ Col 5 - Col 6.
Col 8.	Field Water requirement = Net Water requirement ÷ overall irrigation efficiency (considered to be 45 percent in this case).
Col 9.	Obtained from field survey.
Col 10.	For land preparation, irrigation requirement =

$$Q \times 60 \times 60 \times 24 \times 11 \times 0.45 = 1400 \times 100 \times 100 \times \frac{175}{1000}$$

$$Q = 5.74 \ m^3/s$$

The above computation assumes the following:

i) Overall irrigation efficiency of 45 per cent.

ii) Water application of 24 hours a day.

Water requirement computation for 1-10 January period:

1400 ha from preceding decade will be provided with crop water requirement whereas additional 1600 ha will need water for land preparation. Application of 89 mm of water on 1400 ha to satisfy the crop water requirement for 10 days is equivalent to 1.44 m^3/s over the whole period.

Similarly application of 175 mm of water on 1600 ha for land preparation over a period of 10 days is equivalent to 7.22 m^3/s with an overall irrigation efficiency of 45 per cent.

The computation for water requirement of T. Aman is given in Table 29.

Table 29

Irrigation Requirement of T Aman

Month/ Decade	ETo (mm)	Kc	ETC (mm)	Per (mm)	Eff. rain (mm)	Net req (mm)	Field req (mm)	Area (ha)	Irrign.Requirement (cumec) Land prep. required	Crop req	Total req
Jul.											
01-10	40	1.10	44	18	40	22	49	450	0.99	-	0.99
11-20	40	1.10	44	18	40	22	49	1000	1.21	0.25	1.46
21-31	44	1.10	48	19	37	30	66	1625	1.25	0.69	1.94
Aug.											
01-10	40	1.10	44	18	41	21	46	1625	-	0.86	0.86
11-20	40	1.10	44	18	50	12	26	1625	-	0.49	0.49
21-31	43	1.10	47	19	42	24	53	1625	-	0.91	0.91
Sep.											
01-10	40	1.10	44	18	32	30	66	1625	-	1.24	1.24
11-20	40	1.10	44	18	31	31	69	1625	-	1.29	1.29
21-30	40	1.10	44	18	46	16	35	1625	-	0.66	0.66
Oct.											
01-10	36	1.05	38	18	24	32	71	1625	-	1.33	1.33
11-20	36	1.05	38	18	5	51	113	1625	-	2.12	2.12
21-31	36	1.05	38	18	5	51	113	1625		1.93	1.93

Yield Loss for Lack of Supplemental Irrigation

Weighted average values along with range of yield loss and supplemental water demands for B. Aus, transplanted (L.T.) Aman and HYV Aman by region are presented in Table 30. The estimated percent yield loss due to moisture stress occurs when the supplemental water need is not satisfied. The water needed is that required to reduce the yield loss to zero. These water needs are not total irrigation requirements since they do not include application and conveyance losses.

Flooding is a key factor limiting yield loss due to moisture stress in L.T. Aman and HYV Aman planted on lower lands especially during July and August but in areas that drain rapidly after the rainy season or in FCD projects that tend to overdrain highland and medium highland areas, moisture stress builds rapidly in September and especially in October frequently causing yield reduction.

Since HYV Aman is restricted to lands with shallow inundation of less than 30 cm to 50 cm, early drought stress is less likely to be offset by flooding; and the rates of loss given in Table 24 are probably indicative of actual conditions. An additional factor that may offset part of these yield losses is capillary flux from high water tables during the monsoon season. This factor will have a more pronounced and persistent effect from early October on lower land phases.

The volumes of secondary water required given in Table 30 include only crop evapotranspiration and percolation losses. These values can be converted to farm gate requirements by dividing by the over all irrigation efficiency.

Table 30

Yield Loss Due to Drought and Supplemental Crop Water Needs

Region	B. Aus			L. T. Aman			HYV Aman		
	Yield Loss Due to Drought	Water Requirement: Month	Water Requirement: Season	Yield Loss Due to Drought	Water Requirement: Month	Water Requirement: Season	Yield Loss Due to Drought	Water Requirement: Month	Water Requirement: Season
	%	(mm/mo)	(mm)	%	(mm/mo)	(mm)	%	(mm/mo)	(mm)
NW	15	60	221	12	25	99	17	32	117
	(1-33)	(47-103)	(175-395)	(1-44)	(18-55)	(75-222)	(1-35)	(2645)	(97-166)
NE	5	24	72	4	24	87	7	32	119
	(4-15)	(10-69)	(36-251)	(1-11)	(5-44)	(20-175)	(2-18)	(16-56)	(59-406)
SE	7	33	121	8	14	55	8	28	101
	(1-13)	(12-85)	(43-312)	(1-23)	(3-84)	(11-337)	(2-28)	(16-79)	(59-291)
SC	15	68	250	3	8	33	3	17	61
	(3-27)	(29-113)	(106-415)	(1-46)	(10-76)	(41-302)	(1-12)	(14-45)	(52-166)
SW	16	60	222	5	17	69	15	40	147
	(1-32)	(21-92)	(75-338)	(1-54)	(5-91)	(19-362)	(6-20)	(23-52)	(84-191)
Bangladesh	12	48	171	8	20	78	13	63	145
	(1-33)	(10-113)	(36-415)	(1-54)	(3-91)	(11-362)	(1-35)	(14-79)	(52-406)

NOTE: 1. Weighted percent yield loss under non-irrigated condition with existing cropping pattern defined in relation to normally expected yields without drought stress (NWP, Vol. I, Chapter-3).
2. Water requirement is average irrigation requirement to reduce yield loss to zero and does not include application and conveyance losses.
3. Upper figure is the weighted average and the lower figure shows the range.
4. Does not includes active floodplains and PA 59 (Sundarban Forest).

SOURCE : MPO National Water Plan, 1986.

Soil

Excerpted from 'Irrigation Practice and Water Management
by FAO

Soil

Soil is a complex system made up of solid, liquid and gaseous materials. The mineral portion consists of particles of various sizes, shapes and chemical composition. These particles are classified according to the size of the grains, as sand, silt, and clay, which essentially determine the texture of the soil. The organic fraction consists of both plant and animal matter, some of which are alive, while others are in different stages of decomposition. The accumulation of partly decomposed organic matter is the humus fraction of the soil.

The liquid portion of a soil consists of the water, dissolved minerals, and soluble organic matter which fills part, or most, of the spaces between the solid particles. This water is absorbed by plant roots and must be periodically replenished by rain or irrigation for the successful production of crops. Thus the soil serves as a reservoir for moisture. This moisture reservoir and a knowledge of its capacity are principle factors governing the frequency and amount of irrigation water to be applied to the land.

The gaseous or vapour portion of the soil occupies that part of the spaces between the soil particles not filled with water. This is an important phase of the soil system as most plants require some aeration of the root system, with the exception of aquatic plants such as rice. Irrigation practice is important in maintaining a reasonable balance between the soil moisture and air.

Soil Texture

The texture of a soil is determined by the size and distribution of its soil particles, i. e. the proportions of coarse, medium, and fine particles which are termed sand, silt and clay, respectively. Various combinations of these fractions are used to classify soil according to its texture. The textural class of a soil can be accurately determined in the laboratory by mechanical analyses. Sand, silt and clay are size groupings of soil particles. Sands range from 2 to 0.05 mm; silt from 0.05 to 0.002 mm; and clays less than 0.002 mm in diameter. These three particle sizes mixed together in various proportions constitute the soil class according to the diagrammatic textural triangle given in Figure 13. For example, a soil containing 13 percent clay, 41 percent silt and 46 percent sand would be a loam soil as located at point "A" on the triangle where the dashed lines intersect. In the triangle an area is marked off showing the limits of mixtures of sand, silt and clay with characteristics of a loam. Surrounding the loam are modifications of it, depending upon the relative proportions of other size fractions. For instance, an increase in silt changes a loam to silt loam. An increase in clay changes a loam to clay loam.

The texture of a soil is closely related to its water holding capacity or the reservoir of moisture available for plant use. It also determines, to a considerable extent, the quantity of water to be applied as well as the frequency of irrigation. Therefore it is necessary to determine the textural class of soil for practical irrigation. This can be done by the "feel" of the soil when rubbed between the thumb and fingers or in the palm of the hand. The smoothness of the soil when wet and "slick out" gives a fair idea of the amount of clay present. The sand particles are gritty. The silt has a floury or talcum-power feel when dry. The following description will help to identify a soil according to its texture.

i. <u>Sand</u>

Sand is loose with single grains. The individual grains can be seen or felt. Squeezed in the hand when dry it will fall apart when the pressure is released. Squeezed when moist, it will form a cast, but will crumble when touched.

ii. <u>Sandy Loan</u>

Sandy loam is a soil containing a large portion of sand, but which has enough silt and clay to make it slightly cohesive. The individual sand grains can readily be seen and felt. Squeezed when dry, it will form a cast which will readily fall apart, but if squeezed when moist, a cast can be formed that will bear careful handling without breaking.

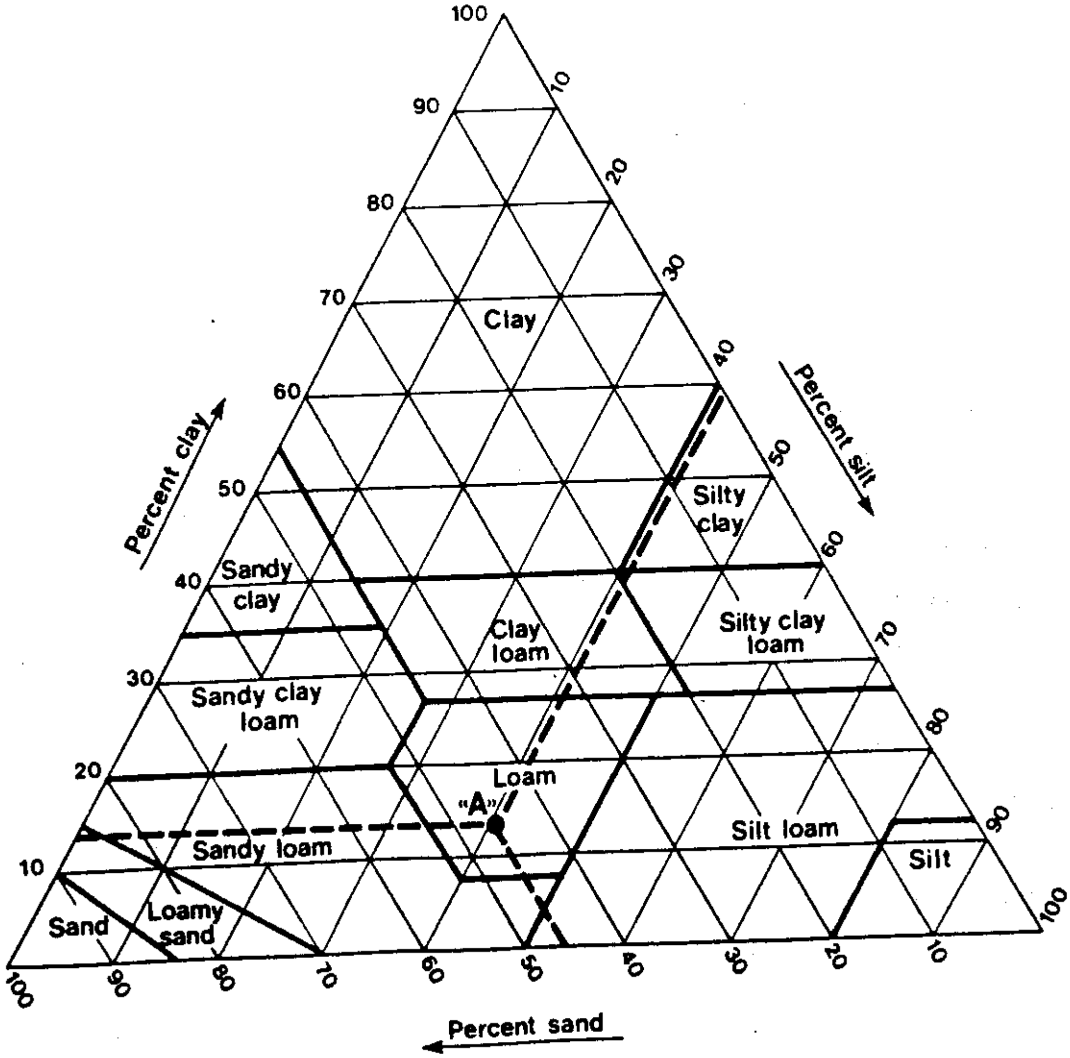


Fig.13 The soil textural triangle

iii. Loam

Loam is a soil having a mixture of the different grades of sand, silt and clay in such proportions that no characteristic predominates. It is mellow with a somewhat gritty feel, and when moist is slightly plastic. Squeezed when dry, it will form a cast that will bear careful handling, while the cast formed by squeezing the moist soil can be handled quite freely without breaking.

iv. Silt Loam

Silt loam is a soil having a moderate amount of the fine grades of sand and only a small amount of clay, over half of the particles being of the size called "silt". When dry, it may appear quite cloddy, but the lumps can be readily broken and when pulverized it feels smooth, soft and floury. When wet, the soil readily runs together. Either dry or moist it will form a cast that can be freely handled without breaking, but when moistened and squeezed between thumb and finger it will not "ribbon" but will have a broken appearance.

v. Clay Loam

Clay loam is a fine-textured soil which usually breaks into clods or lumps that are hard when dry. When the moist soil is pinched between the thumb and finger it will form a thin "ribbon" which will break readily, barely sustaining its own weight. The moist soil is plastic and will form a cast that will bear much handling. When kneaded in the hand it does not crumble readily, but tends to work into a heavy compact mass.

vi. Clay

Clay is a fine-textured soil that usually forms very hard lumps, or clods, when dry and is quite plastic and usually stickily when wet. When the moist soil is pinched out between the thumb and finger it will form a long flexible "ribbon". Clay soil, having a high percentage of clay, is some times referred to a "adobe soil" and is extremely plastic and sticky when wet.

Soil Structure

The structure of a soil refers to the arrangement of the soil particles and the adhesion of smaller particles to form large ones or aggregates. On the surface, soil structure is associated with the tilth of the soil. The permeability of a soil for water, air and the penetration of roots is influenced primarily by the soil structure. Soils without definite structure may be single grain types, sands, or massive types such as heavy clay. Crumbly or granular structures are desirable for good irrigation management and crop growth.

Soil structure, unlike soil texture, can be changed. It can be improved or maintained by good farm practice, such as the rotation of crops and timely cultural practices. Cycles of wetting and drying improve soil structure. On the other hand, the cultivation of soils when the moisture content is too high tends to destroy soil structure and at the other extreme - destroy the structure. An unfavourable concentration of sodium salts in the soil, resulting in an alkali condition, causes a deterioration of structure by a breaking down or dispersion of the clay aggregates. The result is a soil having a low infiltration rate, and in some cases it may become nearly impermeable to water.

Storage of Soil Moisture

Soil is a porous material composed of particles of many different sizes touching each other, but leaving spaces in between. The space not occupied by the particles is known as "pore space" and for most soils constitutes 40 to 60 percent of its volume. Water is stored in the pore space. This stored water or soil moisture is used by the plant and in low rainfall areas must be replenished by irrigation. The following discussion covers the water-soil-plant relationships.

Saturation

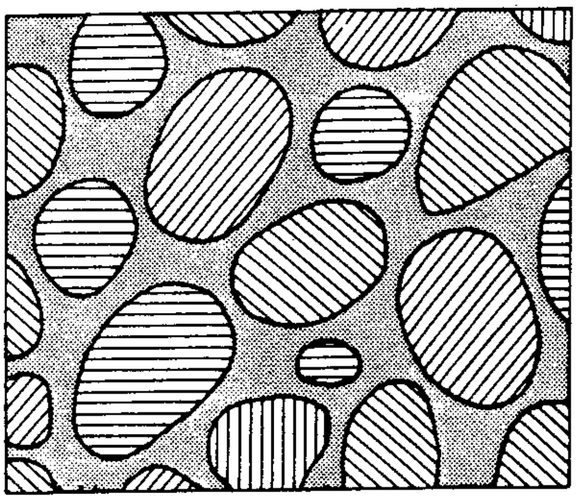
During and immediately following surface irrigation, the soil below the water surface is nearly saturated. All the pore space or small openings between the soil particles are almost completely filled with water as illustrated in Figure 14a. There is little air present in the saturated soil. Since plants, with the exception of rice, need air as well as water, some of the water from the larger pores must move out in a reasonable length of time to prevent damage to the crop. If the soil is well drained, part of the water will move downward by gravity, and to a limited extent laterally by capillarity. The water moving downward by the forces of gravity is called gravitational water or free water.

Field Capacity

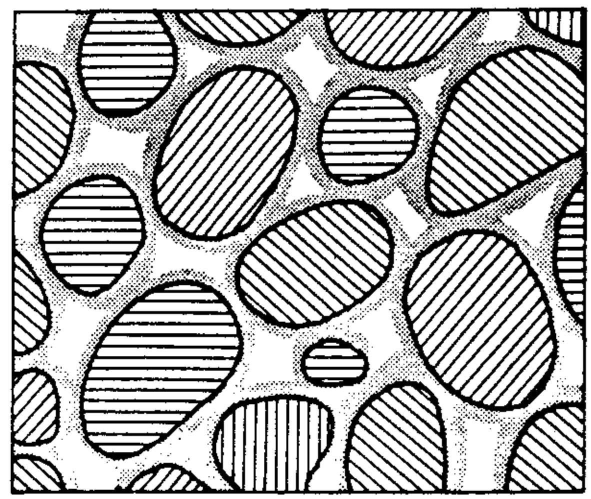
The amount of water retained after drainage of saturated soil is called field capacity. At field capacity, each soil particle is completely surrounded with a relatively thick film of water. However, most of the water is located in the form of wedges between the soil particles as indicated in Figure 14b. It is from these wedges that plants obtain most of their water. The moisture held in the soil against gravity may be described in terms of moisture tension. Tension values may be expressed in equivalent atmospheres or height of water column in centimetres. In converting soil moisture tensions to equivalent atmospheres 1 atmosphere is approximately equal to a suction or a negative pressure of 1000 cm height of water column. At field capacity a loam or clay soil retains moisture

under a tension of about 1/3 atmosphere, or a water column height of 300 cm, whereas sands may be as low as 0.1 atmosphere or a water column height of 100 cm.

(a) Saturation-pore space filled with water



(b) Field capacity-water held by soil particles after surplus has drained by gravity



(c) Permanent wilting point-water surrounds soil particles, but so tightly it cannot be removed by plants

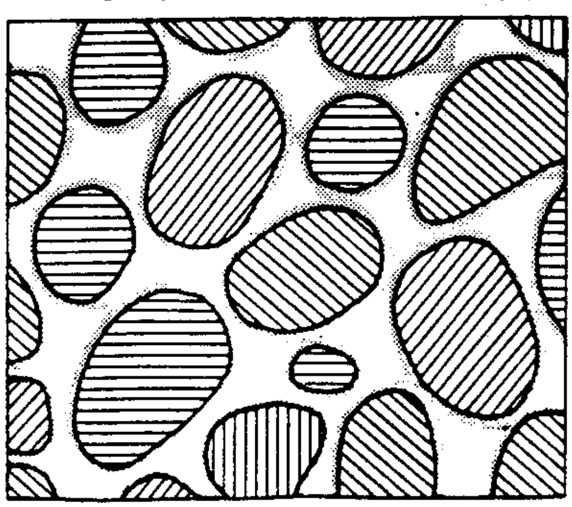


Fig. 14 Soil moisture conditions

The volume of soil wetted to field capacity by an irrigation will depend upon the dryness of the soil, its texture, structure and the amount of water applied. The moistened portion of a drained soil of uniform texture and structure reaches its field capacity two or three days after a rain or irrigation. This time period is increased if there are layers of soil which hinder the downward movement of water, or with a very fine textured soil such as clay. The field capacity of a soil depends upon the soil texture or size of the soil particles. For example, assume we have several 1 cm cubes, each containing soil of a different texture. Also assume all the soil particles are spheres arranged in an orderly manner. Table 31 shows how the total surface area of the soil particles varies with the texture of the soil.
For Table 31 it can be noted that the surface area of the soil becomes larger and the number of points of contact between the soil particles is greatly increased as the size of the particles becomes smaller. therefore, a fine-textured soil will retain more water than one of coarse texture.

The very large surface area, or internal surface of soil, can be illustrated as follows: 1 m^3 of a silt loam has an internal surface area of 70 000 m^2. Therefore a maize plant rooting 120 cm deep has an internal soil area of 7.0 ha from which moisture and plant nutrients may be obtained.

At field capacity, a cubic metre of a typically sandy soil will hold about 135 litres of water, a loam soil about 270 and a clay about 400. The addition of organic matter, such as manures, straw, or plant residue, will only slightly increase the field capacity and usually only in the surface soil (this does not apply to the nutrient value of these materials or their influence on structure and permeability of the soil).

Table 31

RELATIONSHIP OF PARTICLE SIZE TO SURFACE AREA

Diameter of particles in cm	Number of particles in 1 cm^3 of soil	Surface area of particles in cm^2
1	1	3.14
1/2	8	6.28
1/16	4 096	50.23
1/1000	1 000 000 000	3 141.60

Permanent Wilting Point

Removal of water from the soil by plant roots causes the water film surrounding the soil particles to become thinner and thinner and most of the water in the wedges between the soil particle to disappear. Finally, a condition is reached where the water is held so tightly by the soil particles that the roots cannot remove it at a sufficiently rapid rate to prevent the leaves from wilting. When this condition is reached, the soil is at the permanent wilting point (PWP), as illustrated in Figure 14c.

The soil moisture tension has now reached about 14 to 15 atmospheres or equal to a suction or negative pressure of a water column 1.5 x 106 cm high. At this level of soil moisture the removal of only a small amount of water, about half to one percent decrease in soil moisture content, will greatly increase the negative pressure, to 30 or more atmospheres, with the ultimate death of most plants.

Wilting or drooping of the leaves is the most common symptom that the PWP has been reached. Some plants will not wilt, but show other signs such as decreased plant or fruit growth or a change in appearance, such as leaf colour.

The PWP is influenced by texture in the same general way as the field capacity, i. e. the fine-textured soils have a higher PWP moisture content than soils of coarse texture.

The wilting or dropping of leaves in mid-afternoon, or when the temperature is the highest, is a sign that soil moisture has been reduced close to the PWP. If this wilted condition is still noticeable the following morning, for most soils, this means that the PWP has been reached in that part of the soil which contains the major portion of the root system. Under these conditions the normal activities of the plant are limited.

Available Moisture

This is the moisture above the PWP. Considerable soil moisture is present below the PWP but is held so tightly by the soil particles that plant roots cannot absorb it rapidly enough to prevent wilting. The water in the soil above the PWP throughout the entire range of moisture content up to field capacity (FC) can be used by plants and is considered the available moisture.

The ratio of field capacity to the PWP is not constant. The ratio for 15 agricultural soils listed in the order of increasing field capacity is given in Table 32. some of these soils, admittedly, were selected to indicate the extreme ranges in the ratio of field capacity to the PWP. For example, compare the clay loam soil (number 14) with a ratio of 1.21 with soil number 8, a silty clay loam, having 4.35. Experience has indicated that for many soils about half of the total water at field capacity is available for plant use, thus giving a ratio of approximately 2. Therefore, if either the field capacity or the PWP is known the other one can be roughly estimated for field work.

Table 32

RELATION OF SOIL TYPE TO MOISTURE CHARACTERISTICS BASED ON DRY WEIGHT OF FIFTEEN SOILS

No.	Soil Type	Location	Field Capacity	PWP %	Ratio Field Capacity / PWP	cm of available water per 30 cm depth
1	Fine Sand		3.3	1.31	2.40	0.86
2	Sand		4.8	3.2	1.50	0.66
3	Sandy Loam		9.7	4.2	2.2	2.03
4	Sandy Loam		11.1	3.1	3.60	3.31
5	Fine Sandy Loam		16.8	8.9	1.9	3.20
6	Silt Loam		17.3	8.2	2.10	3.76
7	Sandy Loam		18.8	6.6	2.8	5.06
8	Silty Clay Loam		21.7	5.0	4.3	6.93
9	Silty Loam		23.4	6.1	3.8	7.35
10	Clay Loam		24.5	11.5	2.1	5.02
11	Clay		27.3	12.5	2.2	6.00
12	Silty Clay Loam		28.3	12.5	2.2	6.43
13	Clay		30.4	16.0	1.9	6.00
14	Clay Loam		31.1	25.7	1.2	1.81
15	Loam		37.9	19.0	2.0	7.82

ANNEX 2

Irrigation Requirement for Rice

Excerpted from FAO Publications.

Cannel (1962) has given an average for six years for a representative group of gravity projects operation by the United States Bureau of Reclamation on a total irrigated area of 1 300 000 acres under semi-arid conditions. Mean conveyance losses were 13.3 percent in the main canals, 17.4 percent in the laterals, and 10.8 percent in the minor distribution systems; thus 58.5 percent of water released at the reservoir was delivered to the farmers. Cannel showed further that water-use efficiency on the farms averaged 50 percent, so that the overall project water-use efficiency was some 29 percent. The gross irrigation requirement is expressed thus:

gross irrigation requirement = net irrigation requirement + conveyance losses (loss and waste of water in transit from sources of supply to the irrigated fields) (See Fig. 15).

Duty of Water

The duty of water is the relationship between irrigation water and area irrigated. It is often expressed as the number of acres on which a crop can be irrigated by a continuous flow of water at the rate *of 1 cubic foot* per second. It is a term commonly used in India, Pakistan, Burma and Malaysia.

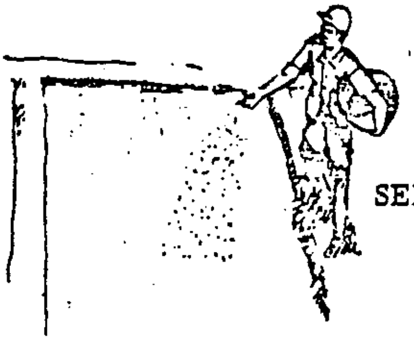
The base of a duty is the time during which the flow is continued. If 1 cubic foot per second running continuously for four months will mature 120 acres of crop, the duty in that case is said to be 120 acres to the cusec to the base of four months. In Vietnam and Indonesia, it is generally said that so many litres of water are required per second to irrigate a hectare. One unit expressed as 1/ha./sec. means a continuous 24-hour a day flow of one litre per hectare per second. Net duty = farm duty; gross duty = diversion or head-rate duty).

1/ha/sec	acres per cusec	acre - inches per acre per 24 hours	1 inch per acre takes hours:
0.58	120	0.195	120
0.70	100	0.235	100
1.00	70	0.336	70

1 litre per second	=	0.035 acre-inch per hour
1 000 litres per minute	=	0.583 " " " "
1 cusec	=	1.0 " " " "
1 000 U.S. gallons per minute	=	2.21 " " " "
100 Imperial gallons per minute	=	2.65 " " " "

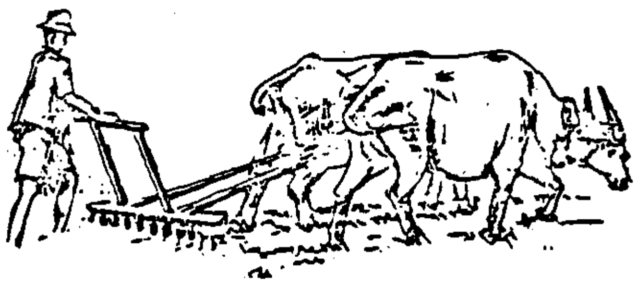
1 acre = 4 047 square metres = 43 560 square feet
1 hectare = 10 000 square metres
1 cubic metre = 1 000 litres
1 inch = 25 mm
1 mm (millimetre) of water per square metre is 1 litre
1 mm of water per acre = 4 cubic metres
1 inch of water per acre = 100 cubic metres
1 cubic foot = 28.3 litres
1 acre foot = 43 560 cubic feet
If 1 acre is irrigated at a rate of 1 cusec it will take 1 hours to supply 1 acre-inch of water.

Fig.15 - IRRIGATION REQUIREMENT FOR RICE



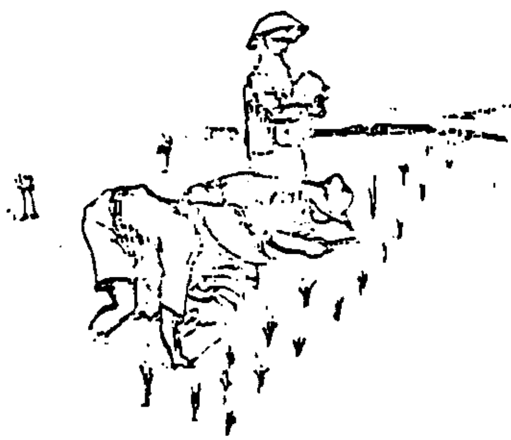
SEED NURSERY $\frac{400 \text{ MM.}}{10} = 40 \text{ MM.}$

+



LAND PREPARATION 200 MM.

+



FIELD IRRIGATION 1,000 MM.

TOTAL 1,240 MM.

Midseason drainage is practised at the middle of maximum tillering stage up to the following panicle primordia development stage when the paddy fields are drained of surface water for the purpose of bringing back the soil from reductive into oxidative status. The water requirement of rice at this period is at its lowest and therefore no ill effects are incurred.

When the paddy field is left in the submerged state for a rather long period, organic matter is decomposed by the action of micro-organisms. With the progress in decomposition, all the free oxygen and even oxygen combined with iron, manganese or in other compounds is consumed and the paddy soil is rapidly brought into a state of reduction. Various acids (such as acetic and butyric), CO_2, CH_4 and hydrogen-sulfids are produced and are accumulated in the soils. These toxic substances retard root development, inhibit nutrient absorption and normal aerobic respiration, and cause root rot. Under ordinary conditions, the development of roots coincides with the formation of tillers and new root development reaches its peak at the maximum tiller stage and begins to decline and cease at about the heading stage. During the tillering stage, root damage due to toxic substances can easily be compensated for by the new root formation and the plant can minimize the effect of the injuries by production of new roots. Therefore, it is necessary to maintain the physical activity of roots at as high a level as possible to promote yield. Nitrogen is essential for the rice plant to build up vegetative and reproductive organs. The requirement for nitrogen decreases in the later stages of growth. Excessive nitrogen exerts an adverse effect on the ripening of grains. Midseason drainage is effective to remove the remaining nitrogen by changing NH_4-N into NO_3-N and also to prevent the further supply of nitrogen caused by mineralization of organic nitrogen in the soil. As a result, the formation of noneffective tillers is suppressed, and the sterility percentage is lowered. Midseason drainage also helps to prevent lodging. It is also partly effective in the removal of excessive nitrogen and in the supply of oxygen to promote the elongating internodes of the culms.

Desirable water depth in different growth stages is recommended as follows in areas where irrigation and drainage facilities are completed.

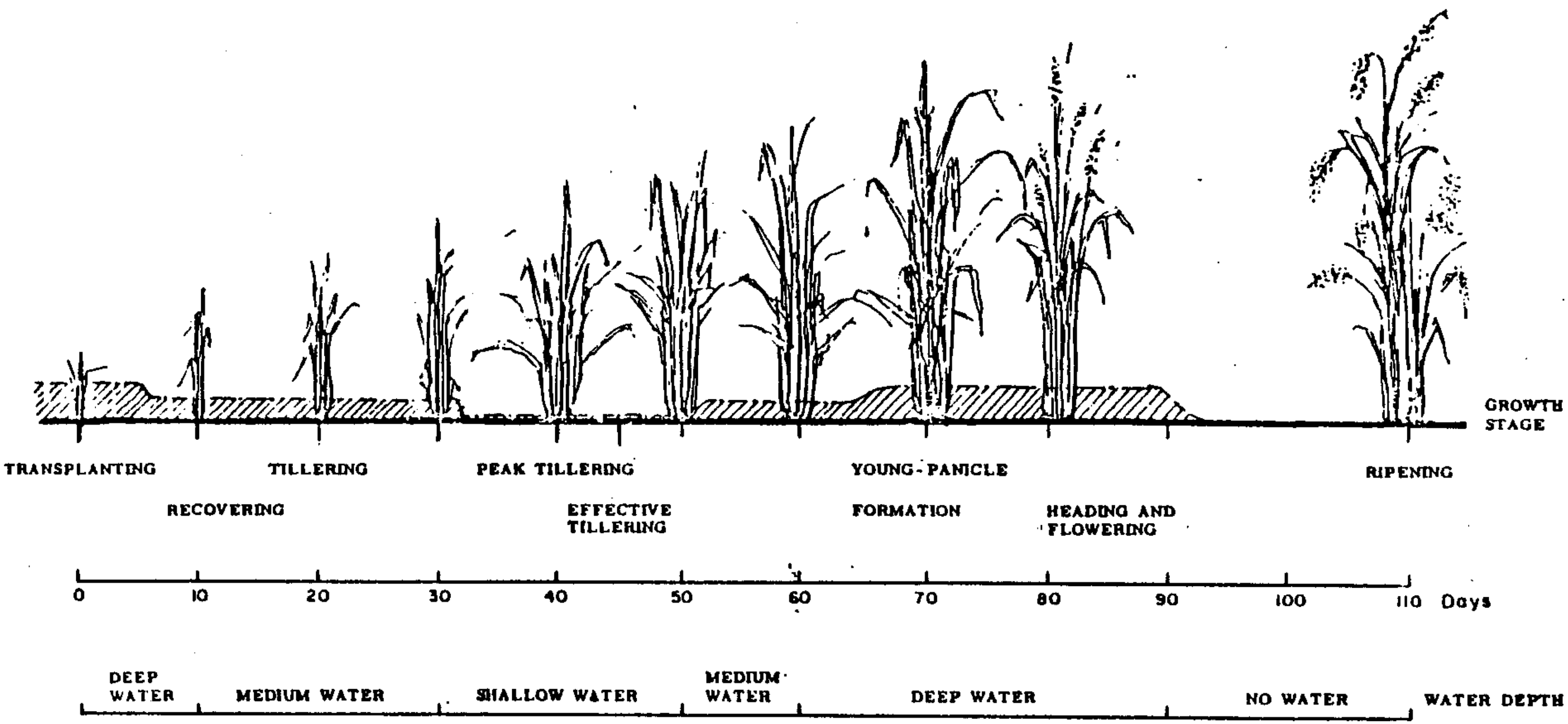
<u>During transplanting</u>. The puddled field should be covered with water about 2 - 3 cm in depth for transplanting the seedlings. Deeper water is not desirable, since this causes the growers to transplant the seedlings too deep. Deep planting will delay the development of new root systems.

<u>After transplanting</u>. Water must be maintained at a depth of 5 - 8 cm immediately after transplanting to facilitate the reestablishment of the seedling and to stimulate rapid growth of new roots. Midseason drainage is practised at the beginning of the maximum tillering stage. Water depth can be reduced gradually by draining. The fields are exposed to dry conditions. Top dressing of fertilizers, weedings and intertillages take place at the period. Fresh water is flooded in only when the soil surface is about to crack.

<u>Panicle primordia development stage</u>. From the beginning of panicle primordia development stage up to the full flowering stage the field must be flooded to a depth of 5 - 8 cm as a water shortage during this and later stages will bring about sterility and cause a big loss in production.

<u>After full flowering</u>. The paddy field must be drained gradually after the flowering stage, or 15 - 20 days after the full crop and to prepare the land for the succeeding crops in places where double cropping is practised (See Fig. 16).

Fig. 16 - CONTROLLED WATER DEPTH IN PADDY FIELD



Effective Rainfall

Excerpted from FAO Publications.

Effective rainfall:

Not all rainfall is effective and part may be lost by surface runoff, deep percolation or evaporation. Only a portion of heavy and high intensity rains can enter and be stored in the root zone and the effectiveness is consequently low. Frequent light rains intercepted by plant foliage with full ground cover are close to 100 percent effective. With a dry soil surface and little or no vegetative cover, rainfall up to 8 mm/day may all be lost by evaporation; rains of 25 to 30 mm may be only 60 percent effective with a low percentage of vegetative cover.

Effective rainfall can be estimated by the evapotranspiration/precipitation ratio method, Table 33 (USDA, 1969). The relationship between average monthly effective rainfall and mean monthly rainfall is shown for different values of average monthly ETcrop. At the time of irrigation the net depth of irrigation water that can be stored effectively over the root zone is assumed equal to 75 mm; correction factors are presented for different depths that can be effectively stored. Data in Table 33 do not account for infiltration rate of the soil and rainfall intensity; where infiltration is low and rainfall intensities are high, considerable water may be lost by runoff which is not accounted for in this method.[1/]

1/ A more detailed prediction of effective rainfall is available in FAO Irrigation and Drainage Paper No. 25, Effective Rainfall. N.G. Dastane, 1975.

Table 33 Average Monthly Effective Rainfall as Related to Average Monthly ETcrop and Mean Monthly Rainfall

(USDA (SCS), 1969)

Monthly mean rainfall	mm	12.5	25	37.5	50	62.5	75	87.5	100	112.5	125	137.5	150	162.5	175	187.5	200
		Average monthly effective rainfall in mm*															
Average monthly ETcrop mm	25	8	16	24													
	50	8	17	25	32	39	46										
	75	9	18	27	34	41	48	56	62	69							
	100	9	19	28	35	43	52	59	66	[illegible]	[illegible]	[illegible]	[illegible]	100			
	125	10	20	30	37	46	54	62	70	76	85	92	98	107	116	120	
	150	10	21	31	39	49	57	66	74*	81	89	97	104	112	119	127	133
	175	11	23	32	42	52	61	69	78	86	95	103	111	118	126	134	141
	200	11	24	33	44	54	64	73	82	91	100	109	117	125	134	142	150
	225	12	25	35	47	57	68	78	87	96	106	115	124	132	141	150	159
	250	13	25	38	50	61	72	84	92	102	112	121	132	140	150	158	167

* Where net depth of water that can be stored in the soil at time of irrigation is greater or smaller than 75 mm, the correction factor to be used is:

Effective storage	20	25	37.5	50	62.5	75	100	125	150	175	200
Storage factor	.73	.77	.86	.93	.97	1.00	1.02	1.04	1.06	1.07*	1.08

EXAMPLE:

Given:

Monthly mean rainfall = *100 mm*; ETcrop = *150 mm*; *effective storage* = *175 mm*

Calculation:

Correction factor for effective storage = 1.07

Effective rainfall 1.07 x 74 = 79 mm

6.8 ESTIMATING EFFECTIVE RAINFALL

Effective rainfall is that portion of rainfall that contributes to meeting the ET requirement of a crop (Hershfield, 1964). This differs diametrically from the hydrologic definition which describes effective rainfall as that portion of the total rain that produces runoff. Thus, rain water that neither leaves as surface runoff nor contributes to excess subsurface drainage may be effective precipitation in the context of irrigation water management. An extensive review of models for estimating effective rainfall from measured rainfall has been published by the Food and Agriculture Organization of the United Nations (Dastane, 1974).

Rain water retained by the plant canopy contributes to the satisfaction of the meteorological evaporative demand. This results in a consequent reduction in use of soil moisture. However, some engineers discount each rainfall event by a small amount, say 2 mm (0.08 in.), in situations where vegetative cover is incomplete or where prevailing ET rates are otherwise less than potential.

Estimates of effective precipitation should take local conditions into account. Rainfall of high intensity or large amounts that produce runoff should be considered to be of reduced value. Similarly, rainfall on an already wet soil profile is ineffective to the extent that subsurface drainage exceeds leaching requirements. Soil moisture accretion after the crop reaches physiological maturity is nonbeneficial unless it is stored in the soil for use by a crop during the next growing season.

Heermann and Shull (1976) upon analyzing seasonal, monthly, daily and hourly occurrence and dissipation of different rainfall amounts concluded that daily ET is increased after a rainfall during the early development of the crop (alfalfa). Frequent irrigations and rainfall increased the total seasonal ET as compared with infrequent rainfall and irrigation. Small rainfall amounts are important, not only in the amount of water received, but because of the associated decrease in potential ET due to cloudy, humid conditions. Techniques are available (Jensen, 1974; Jensen et al., 1971; Ritchie, 1972) to account for increased evaporation immediately after an irrigation or rainfall.

Two of the simple models of estimating effective rainfall from measured rainfall are presented here. The first method is very simple and was apparently developed by the U.S. Bureau of Reclamation for monthly water resource calculations. Stamm (1967) makes the following comments about its use. The method is intended for the arid and semi-arid areas of the Western United States. To be conservative the method should be applied to the driest 5 consecutive years in the growing season only. The latter requirement has

TABLE 34. EFFECTIVE PRECIPITATION BASED ON INCREMENTS OF MONTHLY RAINFALL (U.S. BUREAU OF RECLAMATION METHOD)

Precipitation increment range			Effective precipitation accumulated - range	
mm	in.	Percent	mm	in.
0.0- 25.4	0-1	90-100	22.9- 25.4	0.90-1.00
25.4- 50.8	1-2	85- 95	44.4- 49.5	1.75-1.95
50.8- 76.2	2-3	75- 90	63.5- 72.4	2.50-2.85
76.2-101.6	3-4	50- 80	76.2- 92.7	3.00-3.65
101.6-127.0	4-5	30- 60	83.8-107.9	3.30-4.25
127.0-152.4	5-6	10- 40	86.4-118.1	3.40-4.65
Over -152.4	Over 6	0- 10	86.4-120.6	3.40-4.75

often been ignored. Table 34 shows factors used to estimate monthly effective rainfall from measured rainfall.

A second commonly used method in the United States of estimating effective rainfall from field measurements was developed by the Soil Conservation Service. The method, which is described in more detail by Dastane (1974), is based on a soil moisture balance performed for 22 stations using 50 years of data. The method recognizes both monthly ET estimates and monthly precipitation measurements. In addition the method indicates that effective rainfall defined for irrigation purposes by the depth of irrigation water applied is directly related to irrigation frequency. The monthly effctive rainfall may be estimated for a 75-mm irrigation application using Table 35. If the irrigation application differs from 75 mm the effective rainfall may be corrected by an appropriate factor selected from Table 36.

TABLE 35 . AVERAGE MONTHLY EFFECTIVE RAINFALL AS RELATED TO MEAN MONTHLY RAINFALL AND MEAN MONTHLY CONSUMPTIVE USE (USDA, SCS)

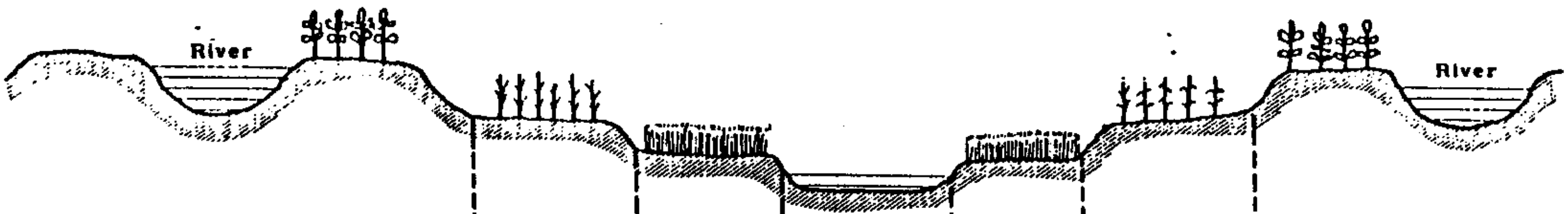
Monthly mean rainfall mm	Mean monthly consumptive use mm													
	25	50	75	100	125	150	175	200	225	250	275	300	325	350
	Mean monthly effective rainfall mm													
12.5	7.5	8.0	8.7	9.0	9.2	10.0	10.5	11.2	11.7	12.5	12.5	12.5	12.5	12.5
25.0	15.0	16.2	17.5	18.0	18.5	19.7	20.5	22.0	24.5	25.0	25.0	25.0	25.0	25.0
37.5	22.5	24.0	26.2	27.5	28.2	29.2	30.5	33.0	36.2	37.5	37.5	37.5	37.5	37.5
50.0	25	32.2	34.5	35.7	36.7	39.0	40.5	43.7	47.0	50.0	50.0	50.0	50.0	50.0
62.5	at 41.7	39.7	42.5	44.5	46.0	48.5	50.5	53.7	57.5	62.5	62.5	62.5	62.5	62.5
75.0		46.2	49.7	52.7	55.0	57.5	60.2	63.7	67.5	73.7	75.0	75.0	75.0	75.0
87.5		50.0	56.7	60.2	63.7	66.0	69.7	73.7	77.7	84.5	87.5	87.5	87.5	87.5
100.0		at 60.7	63.7	67.7	72.0	74.2	78.7	83.0	87.7	95.0	100	100	100	100
112.5			70.5	75.0	80.2	82.5	87.2	92.7	98.0	105	111	112	112	112
125.0			75.0	81.5	87.7	90.5	95.7	102	108	115	121	125	125	125
137.5			at 122	88.7	95.2	98.7	104	111	118	126	132	137	137	137
150.0				95.2	102	106	112	120	127	136	143	150	150	150
162.5				100	109	113	120	128	135	145	153	160	162	162
175.0				at 160	115	120	127	135	143	154	164	170	175	175
187.5					121	126	134	142	151	161	170	179	185	187
200.0					125	133	140	145	158	168	178	188	196	200
225					at 197	144	151	160	171	182				
250						150	161	170	183	194				
275						at 240	171	181	194	205				
300							175	190	203	215				
325							at 287	198	213	224				
350								200	220	232				
375								at 331	225	240				
400									at 372	247				
425										250				
										at 412				
450	25	50	75	100	125	150	175	200	225	250				

TABLE 36 . MULTIPLICATION FACTORS TO RELATE MONTHLY EFFECTIVE RAINFALL VALUE OBTAINED FROM TABLE 6.13 TO NET DEPTH OF IRRIGATION APPLICATION (d)

d mm	factor	d mm	factor	d mm	factor
10.0	0.620	31.25	0.818	70.0	0.990
12.5	0.650	32.5	0.826	75.0	1.000
15.0	0.676	35.0	0.842	80.0	1.004
17.5	0.703	37.5	0.860	85.0	1.008
18.75	0.780	40.0	0.876	90.0	1.012
20.0	0.728	45.0	0.905	95.0	1.016
22.5	0.749	50.0	0.930	100.0	1.020
25.0	0.770	55.0	0.947	125.0	1.040
27.5	0.790	60.0	0.963	150.0	1.060
30.0	0.808	65.0	0.977	175.0	1.070

Flood Depths and Submergence Tolerance of Rice

Fig. 17 - CROP DISTRIBUTION AND PROPOSED CROPPING PATTERNS UNDER DIFFERENT TOPOGRAPHICAL CONDITIONS



LAND CLASS	Levee	Upland	Paddy	Low Depression	Paddy	Upland	Levee
WATER MANAGEMENT	Pumping Irrigation	Pumping or Gravity Irrigation	Gravity Irrigation and Drainage	Drainage			
CROP DISTRIBUTION	Fruits, Perennial crops and Vegetables	Upland Crops	Rice	Floating Rice			
PROPOSED CROPPING PATTERNS	a. Fruits b. Perennial Crops c. Vegetables d. Mixed Farming	a. Single Upland Crop b. Double Upland Crop	a. Single Rice b. Rice-Upland Crop c. Double Rice	a. Single Rice b. Double Rice			

X. SUBMERGENCE TOLERANCE OF RICE

Excessive water depth in the paddy field for a prolonged period will significantly reduce yield. Prolonged periods of submergence may be caused by floods in cases where the capacity of the drainage system is not enough to drain excess water. The allowable submergence depth and time differ greatly according to the crop growing stage. For example, the greatest flood damage occurs at the panicle formation stage (submergence for more than 2 days seriously affects crop growth). Table 37 shows the relationship between flooding conditions and yield of rice.

Table - 37. Submergence damage of rice at different growth stages (Tsutsui, 1972 in Irrigation and Drainage Paper, Farm Water Management Seminar, FAO).

	Clear Water				**Muddy Water**			
	Days of Submergence							
	1-2	**3-4**	**5-7**	**7**	**1-2**	**3-4**	**5-7**	**7**
	% Yield Reduction							
20 days after transplanting	10	20	30	50				
Young panicle formation partly inundated _1_/	10	30	65	95-100	20	50	85	90-100
Young panicle formation, completely inundated.	25	45	80	80-100	70	80	85	90-100
Heading stage	15	25	30	70	30	80	90	90-100
Ripening stage	0	15	20	20	5	20	30	30

1/ Partly means leaves (9-15 cm. long) remain above water surface.

CHAPTER TWO

BASIC HYDRAULICS

H. R. Khan

BASIC HYDRAULICS

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BASIC ELEMENTS OF OPEN CHANNELS

Free surface flow, or open-channel flow, include all cases of flow in which the liquid surface is open to the atmosphere. Thus, flow in a pipe is open-channel flow if the pipe is only partly full.

A uniform channel is one of constant cross section. It has uniform flow if the grade, or slope, of the water surface is the same as that of the channel. Hence, depth of flow is constant throughout. Steady flow in a channel occurs if the depth at any location remains constant with time.

The discharge Q at any section is defined as the volume of water passing that section per unit of time. It is expressed in cubic feet per second, cfs, and is given by

$$Q = VA \qquad \text{....(1)}$$

where V = average velocity, fps

A = cross-sectional area of flow, sq ft

When the discharge is constant, the flow is said to be continuous and, therefore,

$$Q = V_1 A_1 = V_2 A_2 \qquad \text{....(2)}$$

Where the subscripts designate different channel sections.
Equation (2) is known as the continuity equation for continuous steady flow.

In a uniform channel, varied flow occurs if the longitudinal water surface profile is not parallel with the channel bottom. Varied flow exists within the limits of backwater curves, within a hydraulic jump, and within a channel of changing slope or discharge.

Depth of flow Y is taken as the vertical distance, ft, from the bottom of a channel to the water surface. The wetted perimeter is the length, ft, of a line bounding the cross-sectional area of flow, minus the free surface width. The hydraulic radius R equals the area of flow divided by its wetted perimeter. The average velocity of flow V is defined as the discharge divided by the area of flow,

$$V = \frac{Q}{A} \qquad \ldots.(3)$$

The velocity head E_v, ft, is generally given by

$$E_v = \frac{V^2}{2g} \qquad \ldots.(4)$$

where, V = average velocity from Eq. (3)

g = acceleration due to gravity. 32.2 ft per sec^2

Velocity heads of individual filaments of flow vary considerably above and below the velocity head based on the average velocity. Since these velocities are squared in head and energy computation the average of the velocity heads will be greater than the average velocity head. The true velocity head may be expressed as

$$E_{va} = a \frac{V^2}{2g} \qquad \ldots. (5)$$

Where a is an empirical coefficient that represents the degree of turbulence. Experimental data indicate that a may vary from about 1.03 for prismatic channels. It is, however, normally taken as 1.00 for practical hydraulic work and is evaluated only for precise investigations of energy loss.

Normal Depth of Flow

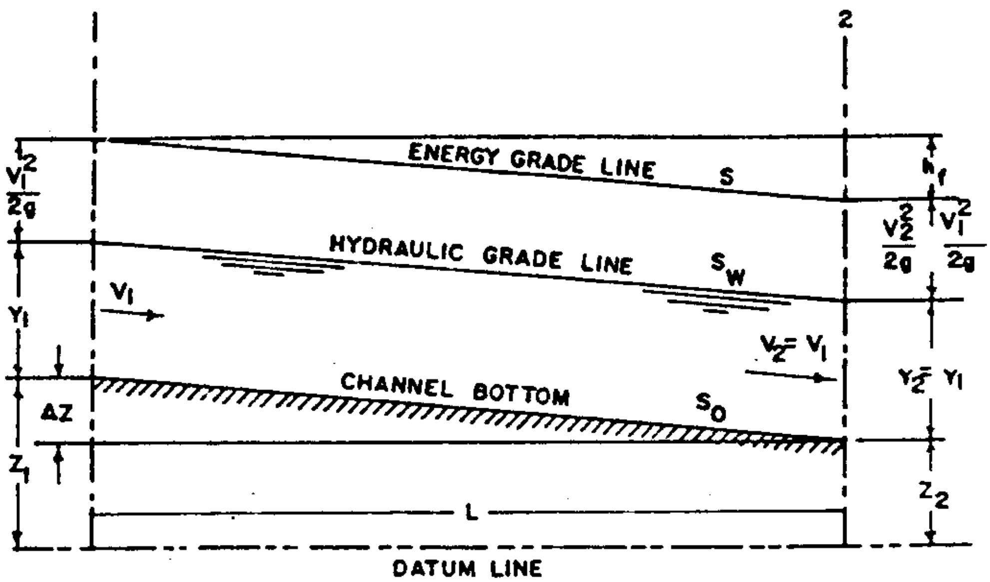

Figure 1. Flow Characteristics for Uniform Open-channel Flow

The total energy per pound of water relative to the bottom of the channel at a vertical section is called the <u>specific energy head</u> E. It is composed of the depth of flow at any point plus the velocity head at the point. It is expressed in feet as

$$E = Y + \frac{V^2}{2g} \qquad \ldots. (6)$$

A longitudinal profile of the elevation of the specific energy head is called the <u>energy grade line</u>, or the <u>total-head line</u>.

A longitudinal profile of the water surface is called the <u>hydraulic grade line</u>. The vertical distance between these profiles at any point equals the velocity head at that point.

Figure 1 shows a section of uniform open channel for which the slope of the water surface S_w and of the energy grade line S equal the slope of the channel bottom S_o.

Loss of head due to friction h_f in channel length L equals the drop in elevation of the channel Dz in the same distance.

<u>Normal Depth of Flow</u>. The depth of equilibrium flow that exists in the channel of Fig. 1 is called the normal depth Y_n. This depth is unique for specific discharge and channel conditions. It may be computed by a trial-and-error process when the channel shape, slope, roughness, and discharge are known. A form of the Manning equation has been suggested for the calculation. (V. T. Chow, "Open-channel Hydraulics," McGraw-Hill Book Company, New York.)

$$AR^{2/3} = \frac{Q\,n}{1.486\,S^{\frac{1}{2}}} \qquad \ldots.. (7)$$

where,

A = area of flow, sq ft

R = hydraulic radius, ft

Q = amount of flow or discharge, cfs

n = Manning's roughness coefficient

S = slope of energy grade line or loss of head, ft, due to friction per line ft of channel.

$AR^{2/3}$ is referred to as a section factor.

SPECIFIC ENERGY AND ALTERNATE DEPTHS

We define the specific energy E as the energy referred to the channel bed as datum, i.e.

$$E = Y + \frac{V^2}{2g} \qquad \text{....(6)}$$

and in this simple concept, introduced by B.A. Batemeteff in 1912 lies the key to even the most complex of open channel flow phenomena. Since $V = Q/A$, Eq.(6) may be written as

$$E = Y + \frac{Q^2}{2gA^2} \qquad \text{.....(8)}$$

It can be seen that, for a given channel section and discharge Q, the specific energy in a channel section is a function of the depth of flow only.

When the depth of flow is plotted against the specific energy for a given channel section and discharge, a specific energy curve is obtained. See Fig. 2. The curve has two limbs. The lower limb approaches the horizontal axis asymptotically toward the right. The upper limb approaches the 45° line as it extends upward and to the right.

The curve shows that, for a given specific energy there are two possible depths, for instance, the low stage Y_1, and the high stage Y_2. The low stage is called the alternate depth of the high stage, and vice versa. At point C, the specific energy is a minimum. This is the condition for critical state of flow. Thus, at the critical state the two alternate depths apparently become one, which is known as the critical depth of flow Y_c. When the depth of flow is greater than the critical depth, the velocity of flow is less than the critical velocity for the given discharge and the flow is called subcritical. When the depth of flow is less than the critical depth, the flow is supercritical.

Alternatively we may say that the curve represents two possible regimes of flow - slow and deep on the upper limb, fast and shallow on the lower limb - meeting at the crest of the curve C.

Other curves might be drawn for other values of Q: Since, for a given value of y, E increases with Q, curves having higher values of Q will occur inside and to the right of those having lower values of Q.

Criterion for a Critical State of Flow

Since V = Q/A, the equation for specific energy in a channel of small slope may be written

$$E = Y + \frac{Q^2}{2gA^2}$$

We know at the critical state of flow $\frac{dE}{dy} = 0$.

Differentiating E with respect to y and equating $\frac{dE}{dy} = 0$, noting Q is a constant, putting $\frac{dA}{dy} = T$, and $D = \frac{A}{T}$, we get for the critical flow condition.

$$\frac{V^2}{2g} = \frac{D}{2}$$

So it means that at the critical state of flow, the velocity head is equal to half the hydraulic depth.

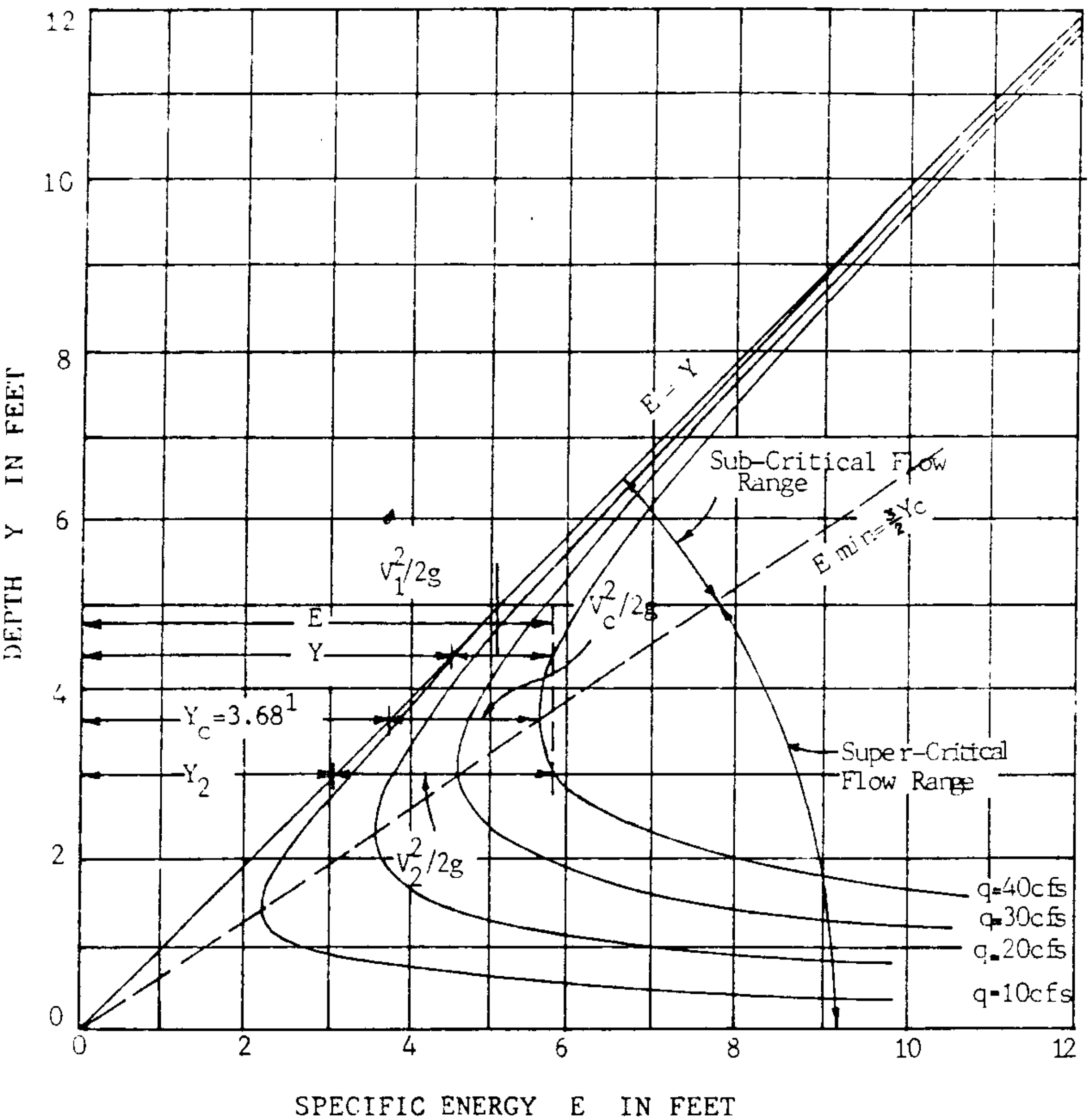

Figure 2. Specific energy and depth of flow.

The above equation may also be written as $V/(gD)^{\frac{1}{2}} = 1$, which means $F_r = 1$.

For rectangular channels, the above two equations are reduced to the following forms

$$\frac{V^2}{2g} = \frac{Y}{2} \quad \text{and} \quad \frac{V}{(gy)^{\frac{1}{2}}} = 1 \qquad \ldots.(9)$$

It can also be shown that at the critical state of flow the discharge is maximum for a given specific energy.

For rectangular channels, it can be proved to be

$$Y_c = \left[\frac{q^2}{g}\right]^{1/3} \qquad \ldots.(10)$$

and

$$Y_c = \frac{2}{3} E_c \qquad \ldots.(11)$$

Where q is the discharge per foot of channel width

= Q/width

Application of the Specific Energy Concept

Much of the complexity of open channel flow relates to the characteristics of critical, subcritical and supercritical flows.

These categories of flow affect the design of conveyance channels and hydraulic structures. This section applies the concept of specific energy to these flow categories and is taken from the National Engineering handbook of the Soil Conservation Service (1971).

Uniform flow at or near critical depth is unstable. This results from the fact that the unique relationship between energy head and depth of flow which must exist in critical flow, is readily disturbed by minor changes in energy. Examine the curve for q = 40 cfs on Figure 2. The critical depth is 3.68 ft and the corresponding energy head is 5.52. If the energy head is increased to 5.60, the depth may be 3.2 or 4.2. Those who have seen uniform flow at or near critical depth have observed the unstable, wavy surface that is caused by appreciable changes in depth resulting from minor changes in energy. In channel design these conditions must be recognized. Variations in channel roughness, cross section, slope, or minor deposits of sediment or debris may cause fluctuations in depth of flow that are significant to channel operation. In many cases it is desirable to base design computations on two or more evaluations of flow resistance in order to establish the probable range of operating conditions. Because of the unstable flow, channels carrying uniform flow at or near critical depth should not be used unless the situation allows no alternative. The critical flow principle is the basis for the design of control sections at which a definite stage-discharge relation is desired or required.

Two general characteristics of subcritical flow are important. First, at all stages in the subcritical range, except those in the immediate vicinity of critical, the velocity head is small in comparison with the depth of flow. Study of the curves of constant discharge will make this point clear (Figure 2). Second, the velocities are less than wave velocity for the depths involved and backwater curve will result from retardation of velocity. Thus, in the subcritical range we are concerned with cases in which the depth of flow is of greater importance than kinetic energy as represented by velocity head. In practice, this means that changes in channel cross section, slope, roughness, and alignment may be made without the danger of developing seriously disturbed flow conditions so long as the design assures that flow in the supercritical range will not be created for some discharges in the operational range. However, in many cases the latitude in design which may be possible as a result of dealing with subcritical flow will be offset by limited head requiring that friction losses be held to a minimum.

Designing structures to carry supercritical flow requires consideration of some of the most complex problems in hydraulics.

In supercritical flow the velocity head may range from a value approximately equal to depth of flow to many times the depth of flow. Note from the curves on Figure 2 that the velocity head increases very rapidly with decreases in depth throughout the supercritical range. Supercritical velocities exceed the velocities at which gravity waves may be propagated upstream. Any obstruction of flow will result in a standing wave, and there will be no effect upon flow upstream or the obstruction. The fact that kinetic energy predominates in supercritical flow and cannot be dissipated through the development of a water surface curve extending upstream is of primary importance in design.

Channels involving changes of direction, contraction or expansion of cross section, or the joining of two flows at a confluence at which either or both of the flows may be supercritical, require careful consideration. Changes in direction or channel contractions develop disturbances at the walls of the channel which take the form of standing waves reflected diagonally from wall to wall downstream from the disturbance points. The height of these standing waves may be several times the depth of flow immediately upstream of the origin of the disturbance. Confluences at which either flow or both flows may be supercritical also develop disturbances resulting in standing waves. In expanding channels the discharge may be incapable of following the channel walls because of the high velocities involved. This results in no uniform depth and the development of a hydraulic jump which is unstable as to both location and height.

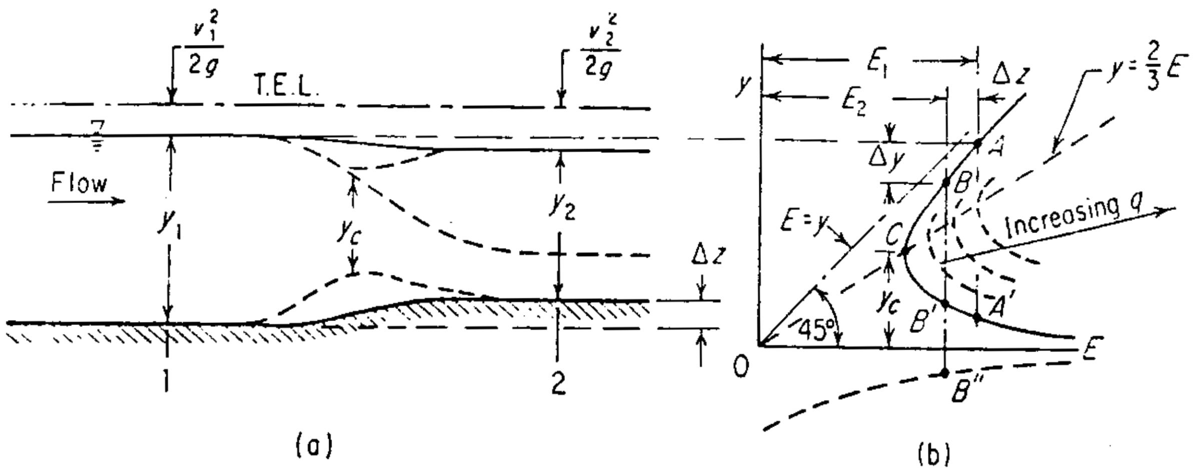

Figure 3. The Specific Energy Curve and Its Use in the Transition Problem

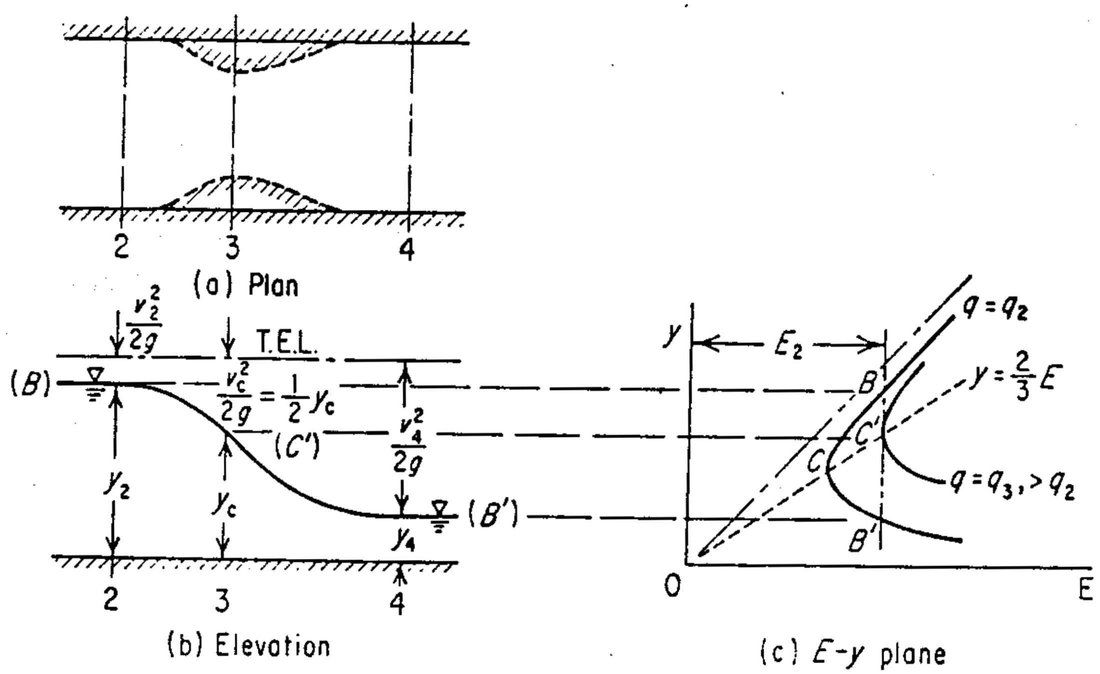

Figure 4. The Change From One State of Flow to Another Via a Contraction in Width

Specific Energy and the Transition Problem

With the aid of specific energy curve it is now possible to see clearly what is involved in the problem shown in Fig. 3. Suppose that the flow upstream of the step (section 1) in the channel bed has such a velocity and depth as to be represented by the point A on the upper (slow and deep) limb of the E-y curve. We assume also that there is no change in channel width over the step, so that value of q ($q = Q/width$) remain unaltered. Since the curve in Fig. 3 is drawn for a given constant value of q, it follows that the point representing the flow over the step (section 2) will also lie on the curve. Its position on the curve is easily located by recalling that the total energy must be the same at sections 1 and 2 and therefore that the specific energies differ by the height of the step, i.e. $E_1 - E_2 = Dz$.

Having obtained the value of E_2, we can now obtain solution describing the flow at section 2; they are represented by the points where the line $E = E_2$ cuts E-y curve. There are two physically real solutions, represented by the points B and B', the third, represented by the point B", is unreal and need not concern us further.

This two solutions would also apply to the case where the upstream flow was represented by the point A', having the same specific energy E as the point A. The question now arises as to which of the two solutions represented by B and B' is more likely to occur in reality, and this question really reduces to one of accessibility: given a certain upstream flow (represented by A) are the two possibilities for the downstream flow (represented by B and B') equally accessible? The energy equation in itself appears to have little to tell us about this problem, but the form of the E-y curve is, as we shall see, a most valuable guide.

As the flow moves over the step, the point representing the flow moves from A to either B or B': If we are concerned with the relative accessibility of B and B', it is reasonable to ask what kind of path this flow-point must follow. The answer is quite simple; since the discharge per unit width is unaltered, the flow-point must therefore remain on the E-y curve which, it will be remembered, was drawn for a certain constant value of q. The point cannot jump across the curve straight from B to B', for if it did the value of q would have to change temporarily from its original magnitude and then return to it again, i.e., the width would have to undergo a similar temporary change. It is easily seen that this change in width would take the form of a constriction followed by an expansion as shown in Fig. 4 (For the sake of clarity this figure is drawn without including the step, as if the constriction were in fact a short way downstream of the step).

If the width remains constant the only available path from B to B' is round the curve itself; but if this path were followed, the specific energy would have to drop below E_2 and return to it again. This could only happen if the bed level temporarily rose above the level of the step (just far enough to bring the point from B to C, the crest of the curve) and then dropped back to the level of the step, as shown in Fig. 3.

The conclusion is that if the width remains unchanged, and if the bed rises steadily to the level of the step without rising above it at any point, the point B' is inaccessible when the flow is represented by the point A. Similarly the point B will be inaccessible if the upstream flow is represented by the point A'. Similarly the point B will be inaccessible if the upstream flow is represented by the point A'. The jump from one side of the curve to the other can be accomplished only by inserting a local constriction either in width or in bed level at or near the step. This statement of course relates only to this particular type of problem; there may be other conditions under which a change from one regime to the other is possible.

One last remark may be made about this example. As we go from A to B the depth decreases and the velocity and velocity head must therefore increase. Since the total energy line remains at the same level, the water surface must therefore drop, as in Fig. 3 as the flow moves over the step-a slightly surprising conclusion in view of one's natural expectation that the water surface would have to rise in order to get over the obstacle presented by the step. On the other hand, movement from A' to B' is accompanied by an increase in depth, so that the water surface rise.

Example 1

Water flows at a velocity of 1.00 m/sec and a depth of 1.80 m in a rectangular channel. There is a smooth upward step of 0.20 m. in the channel bed; find the depth of water over the step and the change in the absolute level of the water surface.

Describing as section 1 and 2 the sections upstream of the step and over the step respectively, we first calculate the upstream specific energy.

$$E_1 = 1.8 + \frac{1^2}{2g} = 1.85 \text{ m}, \quad g = 9.81 \text{ m/sec}^2$$

Then the downstream specific energy is equal to

$$E_2 = E_1 - 0.20 = 1.65 \text{ m}$$

and since $q = 1.80 \text{ m}^3/\text{s}$ per meter width.

$$E_2 = Y_2 + \frac{q^2}{2gY_2^2} = Y_2 + \frac{0.165}{Y_2^2} = 1.65 \text{ m}$$

This is a cubic equation, which can nevertheless be readily solved giving the result

$$Y_2 = 1.59 \text{ or } 0.36 \text{ m}$$

and since, as is easily shown, the upstream flow is subcritical, then only the subcritical value of Y_2 (i.e. 1.59 m) is possible. The water level is now 1.59 + 0.2 = 1.79 m above the upstream channel bed, so that the water level has dropped by only 1.80 - 1.79 = 0.01 m.

Example 2

Upstream conditions are as in Example 1. The width contracts smoothly from 2.5

m to 2 m; find the depth of water within the contraction.

E_1 = 1.85 m as before, and on this occasion E_2 has the same value. But q_2 = 1.8 x 2.5/2 = 2.25, whence:

$$E_2 = Y_2 + \frac{q^2}{2gy_2^2} = Y_2 + \frac{0.258}{Y_2^2} = 1.85 \text{ m}$$

yielding the solution

$$Y_2 = 1.77 \text{ or } 0.425 \text{ m}$$

Again, only the subcritical value of Y_2 (i.e., 1.77) is possible. Also, the change in the absolute level of the water surface is again small because of the low upstream froude number.

<u>Smallest Section</u>

There is a certain degree of channel contraction beyond which no solution is possible. The amount of this contraction is readily calculated in the case of the upward step; the maximum height of step permissible is simply equal to the difference between the upstream specific energy and the minimum possible specific energy (that of critical flow), which is easily found from equations derived earlier (Figure 5).

It is important to notice that when the step is of this critical height, and is of limited length the downstream flow may be either supercritical or subcritical, depending on the downstream conditions. Referring back to the earlier discussion centered on Fig. 3 it can be seen that if the point representing flow moves

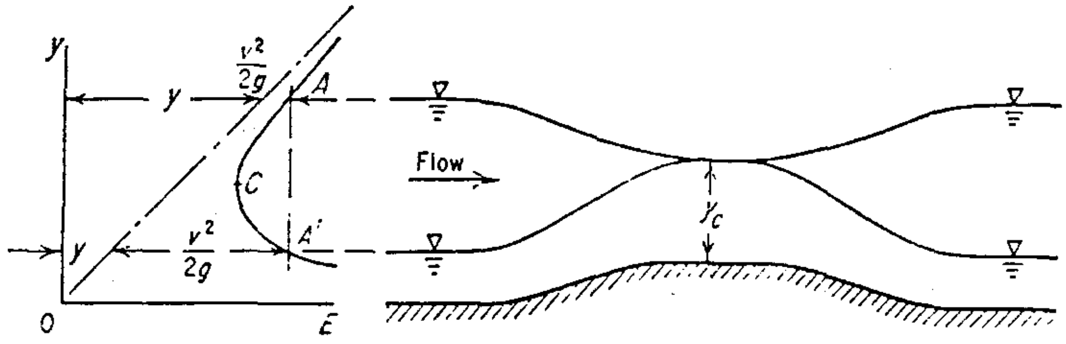

Fig. 5. The Effect of a Short Upward Step of 'Critical' Height

from A to C, it is free thereafter either to move back up the subcritical limb of the curve, or to carry on round to the supercritical. If there is a downstream control such as a sluice gate the flow will tend to become subcritical, if not supercritical. If there is any doubt about downstream conditions, the tendency will be towards supercritical flow; the convergence of flow towards the step will tend to carry on into a further convergence downstream of the step. It seems that the same principle operates when the upstream flow is supercritical. Either of the upstream flow regimes in this figure can pass to either of the downstream regimes. Similar considerations apply in the case of a constriction in width.

The problem dealt with in this section is of considerable practical interest. It often happens that a local constriction must be introduced into a channel. e.g., to reduce bridging cost when the channel is being led under a highway. In such cases it is essential to know just how much constriction can be tolerated without influencing the flow upstream of the constricted section. The problem is simply that of sustaining a certain discharge for a given specific energy; the smallest section that can do this job is the one that will operate at critical flow, which gives maximum discharge for a given specific energy.

Further, it has already been shown (for rectangular channels anyway) that if this smallest section is provided, the flow will automatically take up the critical condition.

The problem is not, of course, confined to rectangular channels; a common case is that in which flow passes from a trapezoidal channel into a circular culvert under a roadway. The principle, however, is still the same: the limiting section is that which can take the required Q at the available E under critical flow conditions.

A constriction which is severe enough to influence the upstream flow becomes a special kind of control.

<u>Example 3</u>

Water flows at a velocity of 2.5 m/s per second and a depth of 2.0 m in an open channel of rectangular cross-section. If the channel width is reduced from 3.0 m to 2.0 m and the bottom elevation is increased 0.30 m at a given section, to what extent will the surface elevation be affected by the boundary constriction?

The Froude number of the approaching flow is

$$F_1 = V_1/(gy_1)^{\frac{1}{2}} = 2.5/(9.81 \times 2)^{\frac{1}{2}} = 0.565$$

Since F_1 is less than unity, the boundary contraction should produce a drop in surface elevation if this does not require a lower specific head at the contracted section than that is physically possible.

$$E = Y + \frac{V_1^2}{2g} = 2.0 + \frac{2.5^2}{2 \times 9.81} = 2.319 \text{ m}$$

Since $Q = VA$ and $q = Q/b$, in the contracted section

$$q_2 = \frac{V_1 \quad A_1}{b_2} = \frac{2.5 \times 3 \times 2}{2} = 7.5 \ m^3/s$$

Hence, for this rate of flow

$$Y_c = (q^2/g)^{1/3}$$

$$= (68.6^2/9.81)^{1/3}$$

$$= 1.789 \ m$$

and

$$E_{min} = \frac{3}{2} Y_c = \frac{3}{2} \times 5.26 = 2.683$$

But this evidently requires a greater total head than that of the approaching flow. The contraction must therefore produce a backwater effect upstream, the entire surface rising until flow taken place at the critical depth (i.e.. minimum specific head) at the contraction. Under such circumstances $Y_2 = Y_c = 1.789$ m and

$$\frac{Q^2}{2gA_1} + Y_1 = E_{min} + DZ$$

$$\frac{15^2}{2 \times 9.81(3Y_1)^2} + Y_1 = 2.683 + 0.3$$

Solution by successive approximation yields $Y_1 = 2.82$ m, indicating that the contraction produces a 0.32 m increase in upstream depth.

DISTINCTION BETWEEN THE HYDRAULIC GRADIENT AND ENERGY GRADIENT

The hydraulic grade line, or the hydraulic gradient, in open channel flow is the water surface, and in pipe flow it connects the elevations to which the water would rise in piezometer tubes along the pipe. The energy gradient is at a distance equal to the velocity head above the hydraulic gradient. In both open channel and pipe flow the fall of the energy gradient for a given length of channel or pipe represents the loss of energy by friction. When considered together the hydraulic gradient and energy gradient reflect not only the loss of energy by friction, but also the conversions between potential and kinetic energy.

In the majority of cases the end objective of hydraulic computations relating to flow in open channels is to determine the curve of the water surface. These problems involve three general relationships between the hydraulic gradient and the energy gradient. For uniform flow the hydraulic gradient and the energy gradient are parallel and the hydraulic gradient becomes an adequate basis for the determination of friction loss, since no conversion between kinetic and potential energy is involved. In accelerated flow the hydraulic gradient is steeper than the energy gradient, and in retarded flow the energy gradient is steeper than the hydraulic gradient. An analysis of flow under these conditions cannot be made without consideration of both the energy gradient and the hydraulic gradient.

STEADY AND UNIFORM FLOW FORMULAS FOR OPEN CHANNELS

Many flow formulas have been proposed to determine the characteristics of flow. However, the Chezy and Manning relations are most commonly used.

Chezy's Formula

Chezy's relation is one of the oldest velocity-resistance to flow relations in hydraulic engineering.

$$V = C(RS)^{\frac{1}{2}}$$

Where C is a boundary roughness coefficient.

Manning's Formula

In 1889 Manning presented an equation to approximate the average velocity in open channels. This formula defines the resistance to flow. The most usual forms are:

$$V = \frac{1}{n} R^{2/3} S^{\frac{1}{2}} \qquad \text{.... (12)}$$

in metric units and

$$V = \frac{1.49}{n} R^{2/3} S^{\frac{1}{2}}$$

in English units

In the above equation n has the dimension of $TL^{-1/3}$. It is difficult to accept that n is a function of time.

In the above equation

V = Average velocity

R = Hydraulic radius = A/P

P = Wetted perimeter

S = Slope of the energy grade line

Using data from field observations of flows in rivers and large channels, both Gauckler in 1868 and Hagen in 1881 came up independently with the following relationships:

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

where M was a coefficient.

This formula was incorrectly attributed to the Irish Engineer, Robert Manning, by the Frenchman Flamant in 1891 when he published the equation, presently known as Manning's Equation in the form given in Eq. 12.

However, n not only changes with Reynolds number but is a function of:

1) the depth of flow
2) the sediment transport (bed load, suspended load, and wash load)
3) floating debris
4) the geometry of the cross section
5) the geometric pattern of scouring and silting in the channel
6) the size of bed material
7) the type and extent of vegetation on the banks
8) the water temperature
9) wind direction and magnitude

In 1959 V.T. Chow published the detailed list of suggested n values some of which are given in the chart below:

Roughness Coefficient n (after Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
C Excavated or Dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.025	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
c. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Type of Channel and Description	Minimum	Normal	Maximum
D. Natural Streams			
D-1 Minor streams (top width at flood stage 100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and section	0.040	0.048	0.055
6. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
D-2 Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050

HYDRAULICS OF FLOW THROUGH SLUICES

Introduction

Discharge through a sluice vent is similar to culvert discharge. The structure forms a unique type of construction and its entrance is a special type of contraction. Flow characteristics through the structure are very complicated because the flow is controlled by many variables, such as inlet geometry, size, roughness, headwater and tailwater levels, etc. Because of these variables the sluice may act as an orifice, a pipe, a weir, or an open channel, depending upon the prevailing conditions.

One should realize that flow is variable under these conditions and should be able to recognize which type of flow will develop under a certain set of conditions. While preparing design for a sluice one should investigate all conditions and develop a structure which will safely convey the flow under the most adverse condition.

In this section we will review the mechanics of flow through sluices for six basic conditions of flow. We will develop the formulae for determining discharge and the discharge coefficients to be used under various flow conditions.

Complied from:

Hydrologic and Hydraulic Design procedures for Drainage Structures, Design Directorate, BWDB.

Types of Flow Through Sluices

Flow through a sluice may be classified into two general types depending upon the existing headwater and tailwater levels. If the inlet and outlet are submerged the sluice vent will flow full like a pipe. If the inlet and outlet are not submerged the vent will flow partially full and will act as an open channel.

For practical purposes the two general types can be broken down into six specific types of flow.

Type 1 Inlet and Outlet Submerged

The outlet is submerged by high tailwater. The vent will flow full like a pipe with the control at the outlet. Discharge is a function of head differential between inlet and outlet water levels.

Type 2 Inlet Submerged, Outlet not Submerged, Vent Hydraulically Long

Although the tailwater level is below the vent soffit the vent will flow full. The vent is sufficiently long for friction forces to cause the depth of flow to expand, filling the vent. This type is common in long culverts but will not usually occur in sluices.

Type 3 Inlet Submerged, Outlet not Submerged, Vent Hydraulically Short

This is the same as Type 2, except the vent is short and will not flow full. It is the usual condition occurring in sluices when the headwater depth is greater than 1.5 times the vent height. The control is at the entrance and is similar to orifice discharge with the discharge coefficient varying from 0.45 to 0.75.

Type 4 Inlet not Submerged; High Tailwater

When the headwater depth is less than 1.5 times the vent height the entrance is not sealed by water and the sluice acts as a broad-crested weir. the tailwater level is higher than critical flow depth through the vent. This type is also associated with a drowned out hydraulic jump. The control is at the outlet and the discharge coefficient varies from 0.75 to 0.95.

Type 5 Inlet not Submerged; Low Tailwater

Type 5 is similar to Type 4 except the tailwater depth is less than critical flow depth through the vent. This type will usually occur when a stilling pool, or down stream drop, is part of the structure and the flow is made to pass through critical depth. A hydraulic jump will usually occur downstream. The control is at the outlet and the discharge coefficient varies from 3.47 to 2.46.

Type 6 Inlet not Submerged; Low Tailwater

This is the same as Type 5, except the vent invert slope is greater than critical. Since sluices are usually designed with a flat invert this type will not occur.

A Typical Sluice Discharge Sequence

The following example illustrates how sluice discharge may pass through the four conditions of flow associated with sluices (Types 1, 3, 4, 5).

Suppose we have a poldered basin with one sluice discharging to an adjacent river. It is near the end of the monsoon season and the basin is flooded from the catchment of monsoon rains. Up till now there has been no release of water because the river flood stage has been higher than the basin storage level. We will assume the storage level is higher than 1.5 times the vent height.

When the river stage fall to slightly below the storage level the gates are opened and discharge begins. Since the river stage is higher than the vent soffit Type 1 flow will take place and the vent flows full.

As the river stage continues to fall the storage level will also drop, but usually at a lesser rate. When the river stage drops below the soffit level, the outlet is no longer submerged. Air will enter from the outlet end at the soffit line and will push into the vent causing a separation between the soffit and the water surface in the vent. At this point one of two things may happen.

If the headwater depth is still greater than 1.5 times the vent height the entrance remains sealed by water and flow type 3 takes place. However, if the headwater depth has dropped to less than 1.5 times the vent height the air within the vent will be able to push past the entrance breaking the water seal. Flow Type 4 will then take place. In either case there will be an intermittent period of Type 3 flow for some brief or extended duration, depending on the relative inlet and outlet water levels. If there is a stilling basin as part of the sluice structure a characteristic of Type 4 flow will be a drowned out hydraulic jump.

As the river stage continues to fall the storage level may lag behind sufficiently so that enough energy is available for the flow to pass through critical depth. Flow Type 5 then takes place and a hydraulic jump will occur downstream.

We wish to emphasize at this point that the above sequence of flow types may not necessarily take place. The determining factor is the relationship between headwater and tailwater levels at any instant, and they are not directly dependent on each other. The Sluice Capacity is the controlling influence.

For Instance

If the river stage drops rapidly and the sluice capacity is small, the headwater level will remain relatively high and flow may pass from Type 1, to 3 to 5. If the river stage drops slowly and the sluice capacity is relatively large, flow may pass from Type 1 to 4 and never enter into the type 6 range.

Problems During Flow Type Transitions

In passing from one flow type to another there is a period of transition. This is always a period of turbulence with the flow shifting back and forth from one type to the other. The engineer should try, as much as he can, to help the water pass through this difficult stage.

When flow passes from Type 1 to Type 3, air enters the vent chamber intermittently with loud sucking noises. Inside the chamber some of the air becomes mixed with the flow and is carried out. A partial vacuum is formed and fresh air tries to enter along the soffit. When it does the vacuum collapses. At time this phenomenon may be severe enough to damage the structure. The engineer can reduce the severity by installing an air vent in the chamber allowing air to enter from some place other than along the soffit.

Similarly, when the flow tries to make the transition from Type 3 to 4, the water seal at the entrance intermittently forms and breaks. This is not as destructive as from Type 1 to 3, but the rate of discharge is reduced. Rounding the inlet soffit lip and providing an air vent will help the water pass through this phase.

When the flow passes from Type 4 to 5 a hydraulic jump will intermittently form and collapse downstream. This is undesirable because undulating waves are formed that travel downstream. These waves will have the tendency to erode the outfall channel banks, endanger boat traffic if it is present, and overtop canal banks if there is insufficient freeboard. A well designed stilling pool that will encourage the formation of a hydraulic jump will reduce this effect.

Discharge Formula and Coefficients

The formulae and coefficients for determining discharge under the various types of flow can be derived from basic orifice and weir formula.

$$Q = CA\ (2g\ \ h)^{\frac{1}{2}}$$

The discharge coefficient, C, varies according to the conditions and the form of the equation, and is directly related to the coefficient of entrance loss, Ke. In actual practice boundary resistance, upstream flow conditions, and other factors also influence C; however this influence is often indeterminate so we will choose to ignore them for design purposes.

In the following articles we will discuss the discharge equations and coefficients for the four types of sluice flow with examples showing their use.

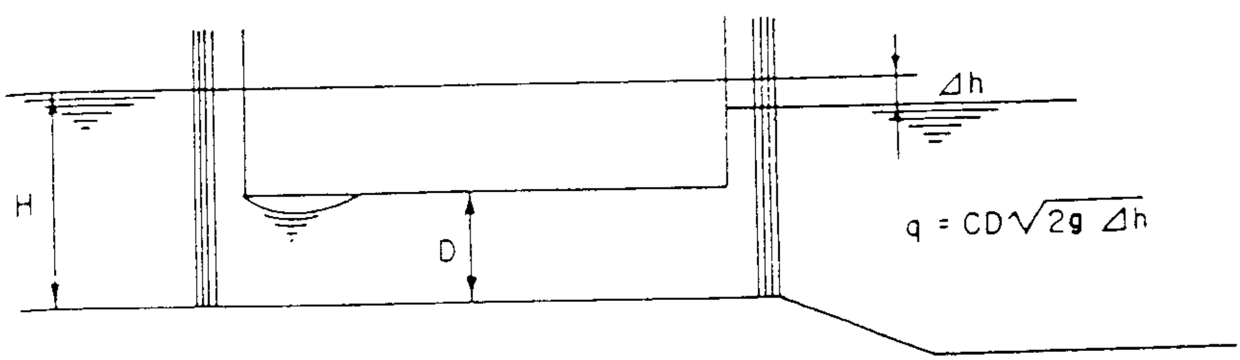

Fig. 6. Flow Condition Type 1

The outlet is submerged by high tailwater. The vent will flow full like a pipe with the control at the outlet. Discharge is a function of head differential between water levels on both sides of the structure.

Under condition 1 (Figure 6) the discharge can be solved by writing Bernoulli's equation between the headwater and tailwater.

The discharge equation

$$Q = CA\ (2g\quad h)^{\frac{1}{2}}$$

The values of C is given in King's Handbook of Hydraulics', 4th Edition, Page 3 - 43. Refer to Table 1.

Table 1. Values of C, Type 1: Culvert Discharge, Entrance and Outlet Submerged - Sluice Flowing full.

$$Q = CA\ (2g\quad h)^{\frac{1}{2}}$$

L	Hydraulic Radius - R				
Feet	.8	1.0	1.2	1.4	1.6
Rounded lip entrance					
10'	.95	.96	.96	.96	.96
20'	.92	.94	.94	.95	.95
30'	.90	.92	.93	.94	.94
40'	.88	.90	.92	.93	.93
50'	.86	.89	.90	.91	.92
Square lip entrance					
10'	.84	.83	.83	.82	.82
20'	.82	.82	.82	.82	.81
30'	.80	.81	.81	.81	.81
40'	.79	.80	.80	.80	.80
50'	.77	.78	.79	.79	.79

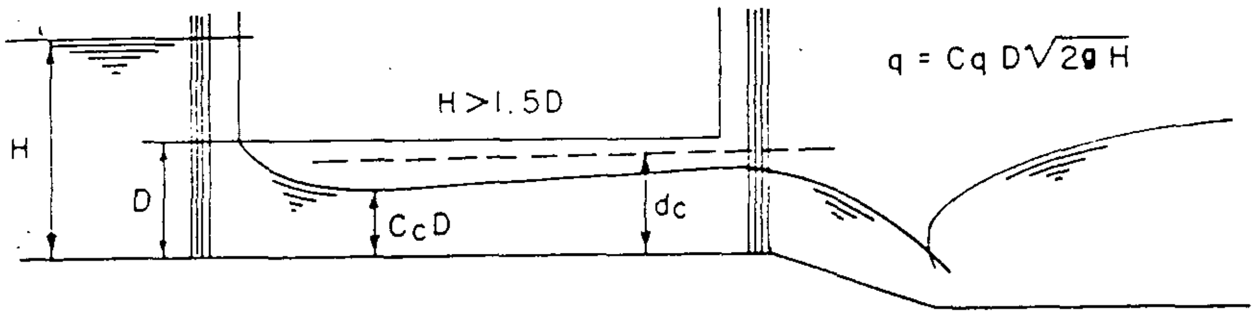

Fig. 7. Flow Condition Type 3

Condition 3 (Figure 7) occurs in sluices when the headwater depth is greater than 1.5 times the vent height. The control is at the entrance and is similar to orifice discharge.

In order for the entrance to be submerged, "H" must be greater than the critical value of 1.2 to 1.5 D. However, in this range the flow is unsteady, so for design purposes 1.5 D will be used as the limit.

A. Orifice Discharge

The flow is similar to discharge through an orifice, and with a multiple vent sluice it can be assumed that the bottom contraction is suppressed and side contractions partly suppressed.

The orifice formula for discharge per unit width is,

$$q = CD\ (2gh)^{\frac{1}{2}}$$ with "h" measured to the orifice center line.

"King's Hand Book of Hydraulics" lists several coefficients of discharge on Pg. 3 - 38, table 30. For the range of head usually found with sluices, C = 0.60 can be used.

<u>**Example:**</u>

Assuming in the above sketch that,

H = 9', D = 6', and h = 6'

$$q = 0.60 \times 6' \times (2g \times 6)^{\frac{1}{2}}$$

$$q = 70.8 \text{ cfs/ft width}$$

B. <u>Sluice Gate Discharge</u>

An alternative method has been developed by Rouse in "Fluid Mechanics for Hydraulic Engineers", Pg 314 - 315

$$q = Cq\ D\ (2gH)^{\frac{1}{2}}$$

Values of Cc may taken from Figure 8.

This above discharge equation is reliable only for values H 1.5 D.

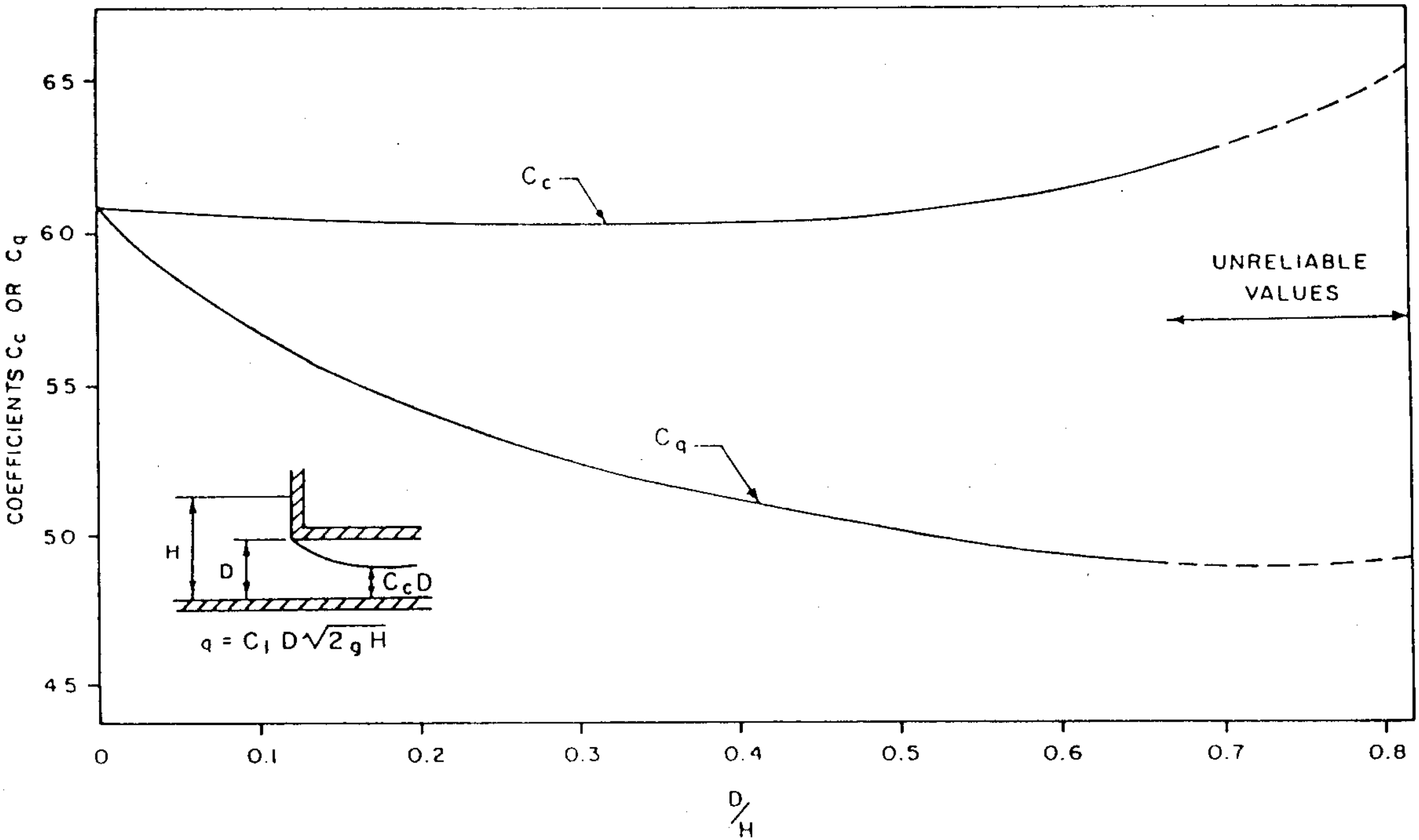

Figure 8. Flow Condition Type 3 Coefficients

Example

$H = 9'$; $D = 6'$ and $D/H = 0.67$

From the Figure 8, $C_c = 0.622$, and $C_q = 0.492$

$q = 0.492 \times 6'(2g \times 9')^{\frac{1}{2}}$

$q = 71.1$ cfs/ft. width

The depth of flow at the contraction is $0.622 \times 6' = 3.73'$

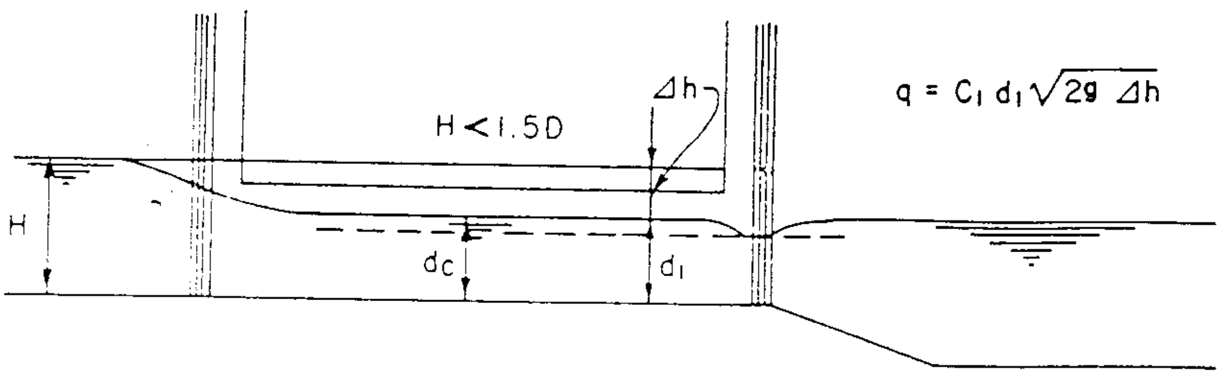

Fig. 9. Flow Condition Type 4

Under Type 4 (Figure 9) with the entrance not submerged the discharge is similar to open channel flow over a broad crested weir.

The tailwater level is higher than critical flow depth through the vent. This type is also associated with drowned out hydraulic jump.

The assumption is made that the U/S velocity head is negligible.

$$\text{or} \quad q = C_1 d \;(2g \;\; h)^{\frac{1}{2}}$$

$$h = (H-d)$$

Table 2 lists some commonly used values of Ke and C_1

<u>**Example**</u>

Usually for flow under these conditions we know the upstream depth "H" and have a tailwater rating curve for the outflow channel. Therefore both "Q" and "d" are unknown and the problem is solved by trial.

Using a sluice with 2-5' wide vents, with H = 7.2', and the tailwater rating curve (Figure 10); assume C = 0.82.

$$q = C1d\ (2g\ \ h)^{\frac{1}{2}}$$
$$q = 0.82d\ (2g\ (7.2-d))^{\frac{1}{2}}$$
$$q = 6.57d\ (7.2-d)^{\frac{1}{2}}$$

Trial 1) Assume d = 5.25' and solving the equation,
q = 48.2 cfs/ft and Q = 482 cfs

Plot this point on the rating curve.

Trial 2) Assume d = 4.75' and solving the equation,
q = 49.0 cfs/ft and Q = 490 cfs

Plot this point and draw a line to trial 1.

The intersection is at d = 5.0' and Q = 488 cfs/ft.
<u>Check Result:</u> $q = 6.57 \times 5' \ (7.2' - 5')^{\frac{1}{2}} = 48.8$ cfs/ft.

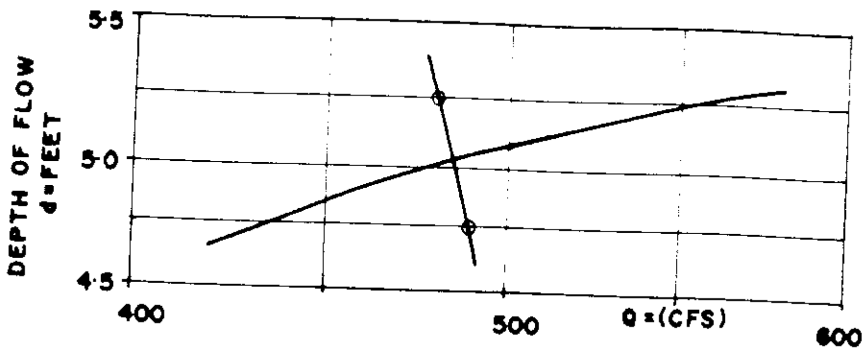

Fig. 10. Tailwater Rating Curve

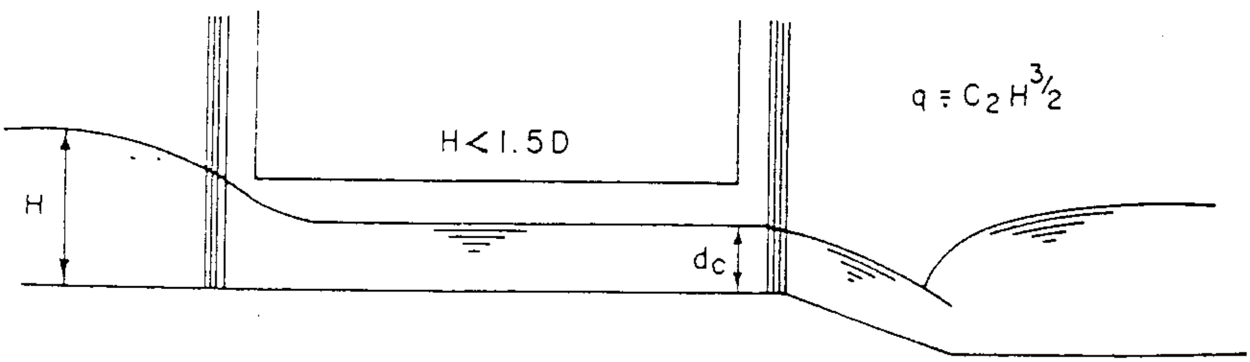

Fig. 11. Flow Condition Type 5

Type 5 (Figure 11) is similar to Type 4 except the tailwater depth is less than critical flow depth through the vent. A hydraulic jump will usually occur downstream. The control is at the outlet.

This condition will usually occur when a stilling basin with glacis is attached to the sluice,and when the flow D/S of the vents is able to spread laterally sufficiently to pass through critical depth.

$$q = C2\ H^{3/2}$$

The values of C are given in Table 2.

Table 2. Coefficients of Discharge - Sluice Flowing Partially Full

Type of entrance	Ke	C1	C2
Warped transition	0.10	0.95	2.94
Cylinder quadrant	0.15	0.93	2.86
Simplified straight line	0.20	0.91	2.80
Straight line transition	0.30	0.89	2.68
Square ended transition	0.50	0.82	2.45
Well rounded entrance	0.30	0.89	2.68
Square entrance	0.50	0.82	2.45

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CHAPTER THREE

DESIGN DATA FOR HYDRAULIC STRUCTURES

H. R. Khan

DESIGN DATA FOR HYDRAULIC STRUCTURES

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DATA CHECKLIST FOR THE DESIGN OF DRAINAGE SLUICE/ REGULATOR/WATER RETENTION STRUCTURE

While submitting proposal for the design of a Sluice, Regulator or Water Control Structure, this Data checklist shall be filled up and sent to the design office along with other requisites mentioned herein.

A. - Name of the Project/Scheme ______________________

____

B. - Name of the Structure ______________________

____

C. - Project Area ______________________

____

D. - Catchment Area above the Structure ______________________

1. PURPOSE OF THE STRUCTURE

Identify the purpose(s) that has to be served by the structure and put tick mark accordingly.

1.1 Pre-monsoon drainage
1.2 Monsoon drainage
1.3 Post-monsoon drainage
1.4 Prevention of pre-monsoon flood
1.5 Prevention of flood
1.6 Flushing of irrigation water
1.7 Retention of post-monsoon water for irrigation

2. MAPS

2.1 Project Index Map:

A project index map showing the project boundary and all project features such as embankment; improvement of drainage channel; irrigation canal; all drainage and irrigation structures; other structures like bridge, bridge culverts etc.

2.2 Basin Map:

A Basin Map in the scale of 4" - 1 mile, preferably 8" = 1 mile showing the location of the proposed structure, main & secondary drainage channels and other water bodies (ponds, beels & haor, etc.), basin boundaries of the drainage channels for the proposed structure, internal and peripheral roads, embankments with all existing regulators, sluices, bridges, culverts, breaches and other structures, spot levels contours, cultivated areas, homesteads, outfall river, location of hydrological station, etc.

2.3 Site Plan:

A plane table survey map in the scale of 1" = 50 feet covering an area of at least 1000 ft x 1000 ft showing spot levels at 25 ft interval, the proposed axis & centre-line of structure, original channel, diversion channels, alignment of embankment and closure, location of bore-holes for sub-soil exploration. In the bore holes R.L (reduced level) should be incorporated. If the boring is done in alternate site, the first boring holes should also be incorporated. The proposed centre-line of the structure and the reference line of survey should be fixed and marked by permanent pillars and those also should be shown on the site plan.

3. HYDROLOGICAL DATA

3.1 Rainfall Data:

3.1.1 Is there any rainfall station within the catchment area? If so, specify the name(s) and length of records available:-

YES NO

Station No.	Name of Station	Length of records available	Remarks

3.1.2 Specify the name and length of records of the rainfall station close to the catchment of the proposed structure.

Station No.	Name of Station	Length of records available	Remarks

3.2 Water Level Data:

3.2.1 Is there any water level station on the outfall river at or near the structure site? I so mention the name of the station(s).

YES NO

Station(s) ____

____

3.2.2 Specify the name of at least one station u/s and one station d/s of the structure site with distances:

U/s station ________________________________

U/s distance __________ miles.

D/s station ________________________________

D/s distance __________ miles.

3.2.3 Is there any water level station on the drainage channel corresponding the proposed structure? If so specify the name.

YES NO

Station(s) ____________________________________

3.2.4 Mention the highest flood level (H.F.L.) ever experienced in the basin indicating the source of the record.

H.F.L. (ever experienced) __________ SOB/PWD from gauge

level at _____________________________ Station/

From average public information.

3.3 Discharge Data:

3.3.1 Is there any record of discharge in the drainage channel?

If so, enclose the data as available:

YES NO

Enclosed discharge data for the year of ____________________

at station ______________________________ .

4. MORPHOLOGICAL DATA

4.1 (a) Are the banks of the outfall river and drainage channel at or near the structure site stable?

YES NO

(b) If not, show the movement of the bank in each year in a map.

Enclosed ________________________________ No. of map.

(c) What is the average rate of erosion in each year?

Average rate of erosion ____________________ ft./year

4.2 Cross-Section of the drainage channel for at least 1/2 mile u/s and 1/2 mile d/s of the structure site at an interval of 200 ft.

Enclosed __________________________ No(s) of Cross-Sections.

4.3 Long section of the drainage channel for at least 2 miles u/s from structure site and upto the outfall river in the d/s:

Enclosed Long Section in _________________________ No (s)

4.4 Cross-Section of the outfall river for a length of half mile u/s and half mile d/s from the confluence point of drainage channel and the outfall river.

Enclosed __________ No(s) of cross-section in __________ sheet(s).

5. MISCELLANEOUS DATA

5.1 Data Related with Embankment/Road Connecting the Structures

5.1.1 Existing or proposed road/embankment profile for at least 500 ft. on each side of the proposed structure.

Enclosed a profile in __________ No. of sheet(s)

5.1.2 Existing or proposed top elevation, top width and side slopes of embankment/road at the structure site.

(i) Top elevation ______________ SOB/PWD

(ii) Top width ______________ ft

(iii) C/S. slope ______________

(iv) R/S. slope ______________

5.1.3 Type of expected traffic loading on road/embankment :

______________________________ Loading.

5.1.4 General classification of the road :-

(a) R & H Road/District Council Road/
Thana Council Road/U.P. Road/Village Road

(b) Highway/Feeder Road/Local Road

5.2 Data Related with Drainage Aspects:

5.2.1 Are the existing section and bed slope of the drainage channel adequate for complete or desired level of drainage?

YES NO

5.2.2 If not, does the scheme include re-sectioning of the drainage channel? If so, the design cross-section and long section may be furnished as requirement indicated in para 4.2 and 4.3 : -

YES NO

Enclosed ____________________ No. of sheet(s)

5.2.3 (a) Is the complete drainage of the basin necessary?

YES NO

(b) If not, mention the drainage level required & distance of such level from the proposed structure site :

Drainage EL. requirement ____________ SOB/PWD

Distance from the structure site _____ mile(s)

5.2.4 From the field condition propose the invert level of the structure which can allow desired level of drainage from field condition.

(a) Proposed Invert Level ______________________ SOB/PWD

(b) Invert level of nearby existing Structure(s) ______________________ SOB/PWD

5.2.5 From field condition, what is the maximum level of acceptable flooding on the basin during drainage period?

Acceptable flooding level _________________________ SOB/PWD

5.2.6 Desired Post Monsoon Drainage Level:

Date ____________________ Level __________________ SOB/PWD

Date ____________________ Level __________________ SOB/PWD

Date ____________________ Level __________________ SOB/PWD

5.3 Data Related With Irrigation Aspects:

5.3.1 Specify the total cultivable and irrigable area within the project.

Cultivable Area ____________________ Acres.

Irrigable Area ____________________ Acres.

5.3.2 Principal crops with acreage in the basin with present and future cropping pattern.

Enclosed in ________________ number of sheet(s)

5.3.3 Proposed retention level of water in the u/s of the drainage channel of the structure for irrigation.

Retention level ______________________ SOB/PWD

5.3.4 For Irrigation by flushing of water, specify the period of such Irrigation.

Period from __________ To ____________ .

6. FOUNDATION DESIGN DATA

Submit the detail soil analysis report. If in the first site poor soil is found, an arrangement for boring in alternate site should be made and detail soil analysis report of the new site is to be given.

ASSESSMENT OF DISCHARGE IN UNGAGED RIVERS/KHALS.

Discharge can be computed by Manning's Formula:

$$Q = \frac{1.486}{n} AR^{2/3} S^{\frac{1}{2}}$$

Where Q - discharge in cubic feet per second

A - cross-sectional area in square feet

S - slope of energy line

n - Manning's roughness coefficient

R - hydraulic radius in feet

$$R = \frac{A}{P}$$

Where P - wetted perimeter in feet

Figure 1 illustrates the geometric elements of river cross-section.

MEASUREMENT OF A:

To compute the flow area A, first measurements have to be taken in the field, and then those have to be plotted in a graph paper.

Field Work:

1. Measure the river's flow width and divide the width into a number of equal segments. The dividing marks are to be the X-co-ordinates.

2. With the help of a levelling instrument and staff gage, adjust the line of collimation (See Fig. 2).

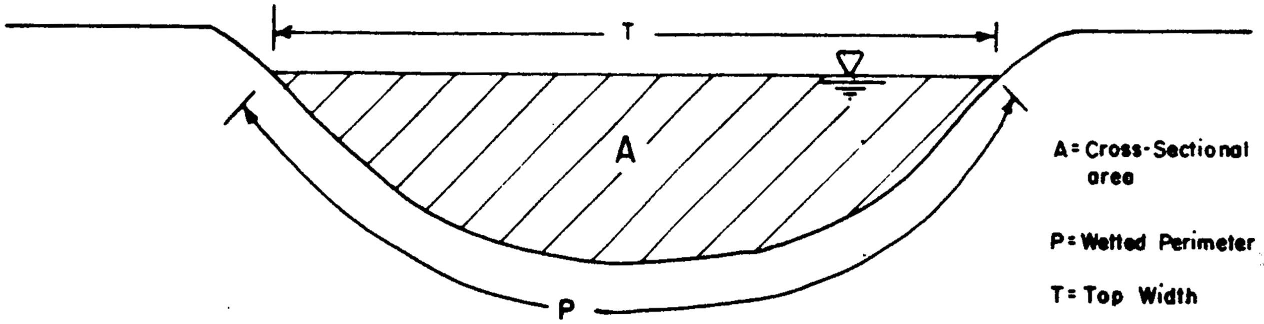


Figure 1 **Geometric Elements of River Cross Section**

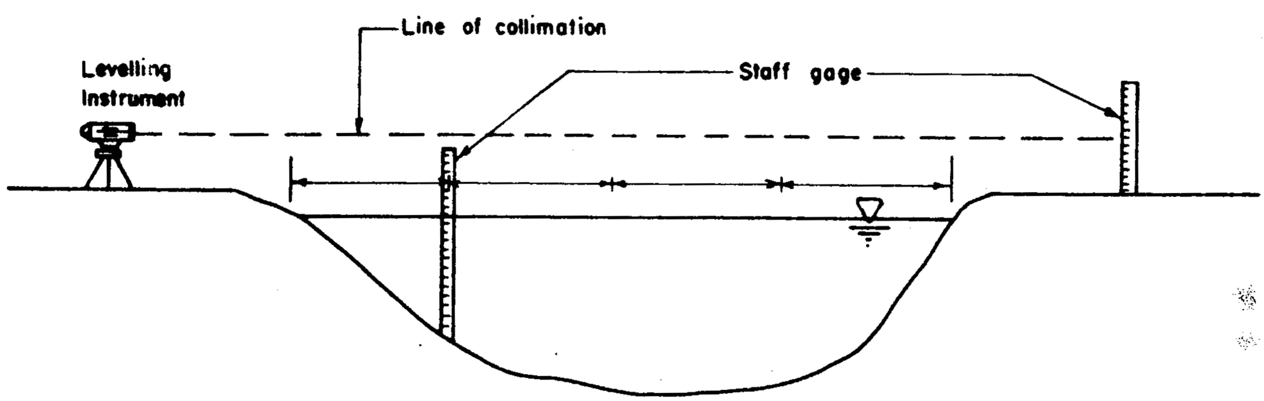


Figure 2 **Measurement of River Cross Section**

3. Locate the X - co-ordinates determined earlier on the line of collimation.

4. At each X - co-ordinate, place a staff gage on the river bed take the gage reading, which gives the Y co-ordinate of the corresponding X co-ordinate. A set of X and Y co-ordinates are recorded.

Plotting in Graph Paper

1. The X and Y co-ordinates recorded in the field area plotted in a graph paper, keeping the scales of the vertical and horizontal axis same (See Fig. 3). As shown in Fig. 3, the scale is 1" = 10' both in the X and Y coordinate.

2. Calculate the area of the smallest square in the graph paper with reference to the assigned scales. In Fig. 3, the area of the smallest square is thus 1 sq. ft.

3. Count the total number of these small squares which fall within the flow area. In Fig. 3 there are 4566 number of small squares.

4. Multiply the number with the predetermined area of a single square, which gives the total flow area A of the river. In Fig. 3, the total flow area is therefore 4566 sq. ft.

MEASUREMENT OF R:

To compute the hydraulic radius R, the methodology is as follows:

1. Take a 2 feet long thread and fix one end of it by any available means to the point on the graph paper, where the river's water surface meets the bank (either left or right).

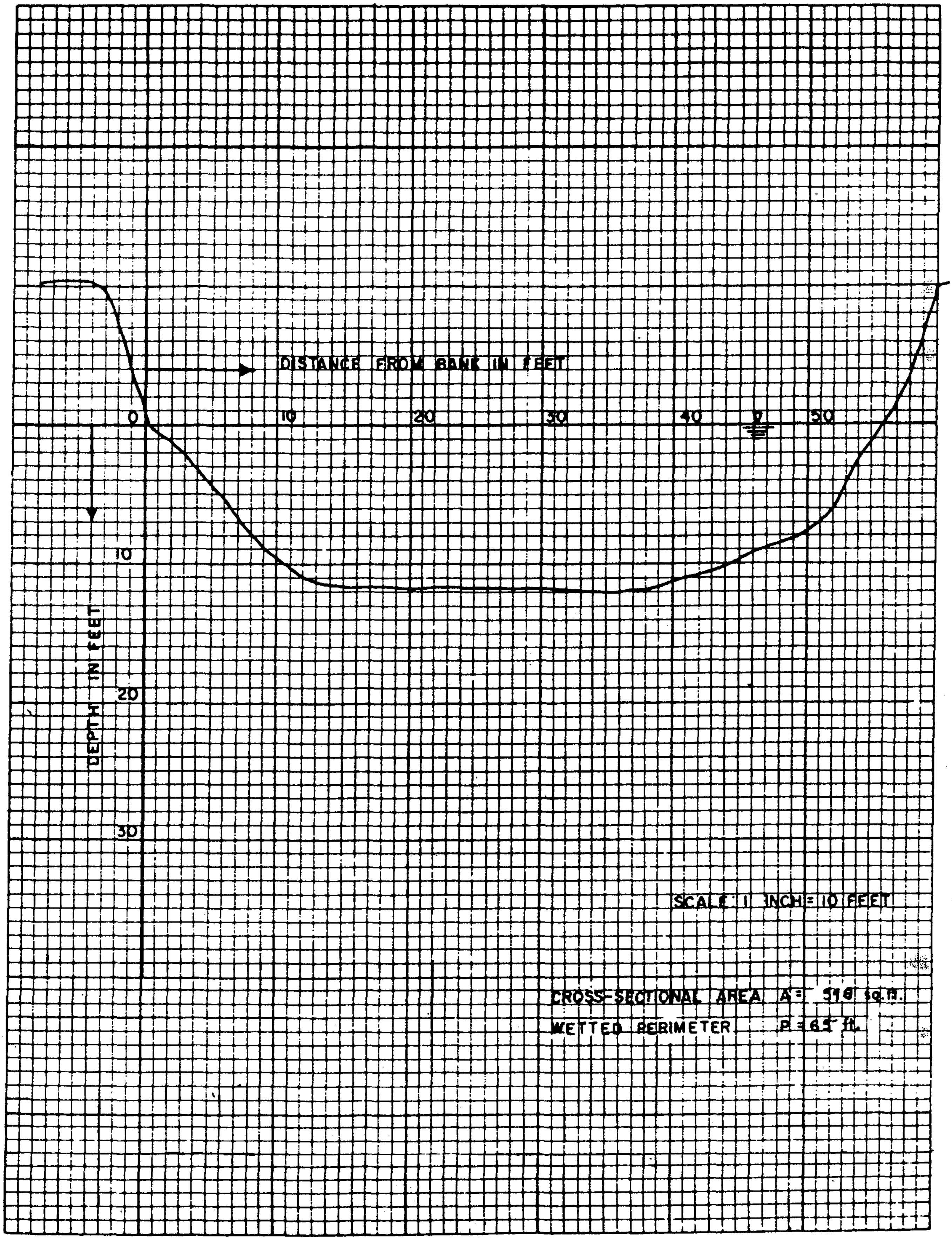


Figure 3. Graphical Measurement of Cross Sectional Area A

2. Slowly go on laying the thread exactly over the perimeter of the river's cross section.

3. Mark the point on the thread which touches the opposite bank.

4. Stretch out the thread straight, laying it on a flat surface.

5. Measure the length of the thread which covered the perimeter of the river's flow area i.e. the distance between the thread's end, which was placed on the point of origin and the marked point. This gives the wetted perimeter P.

6. Divide flow area A, calculated earlier by wetted perimeter P, to get hydraulic radius R.

MEASUREMENT OF S:

For practical purposes, considering the flow to be uniform, the energy slope line may be assumed to be parallel to the bed slope, which is equal to the fall F of the river's bed surface within a reach, divided by the length L of the reach,

So, $S = \dfrac{F}{L}$.

n - VALUES:

Take any one of the following n values which best fits the prevailing river condition:

0.025 for canals and rivers in earth, in tolerably good order, free from stones and weeds.

0.030 for canals and rivers in bad order, occasional stones and weeds.

0.035 for canals and rivers obstructed by detritus and weeds (very rough surface).

0.040 for canals and rivers obstructed by rough rubble and rough bottoms and much vegetation.

0.050 torrential rivers with beds covered with detritus and boulders.

0.060 very rough heavy grass.

If the water level rises above the bank fully flow condition, the n values increase, due to increased obstruction by vegetation on the river banks. (See Fig. 4).

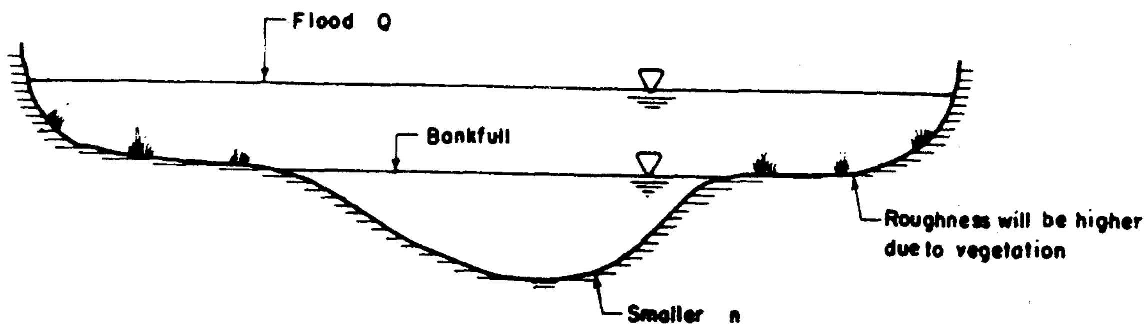


Figure 4 Manning's Roughness Coefficient Under Flood Flow Condition.

SMALL SCALE WATER CONTROL STRUCTURES III

WORKSHOP ON DESIGN OF SMALL SCALE

WATER CONTROL STRUCTURES

JUNE 3 - 7, 1990

Background Paper for Discussion Session

By H. R. Khan

Topic: Design Data Check List for Hydraulic Structures

INTRODUCTION

BWDB Design office prepared a data check list for the design of drainage sluices/ regulators and water retention structures. For approved structures, the field offices fill up the data check list and send that to design offices where designs are done on the basis of supplied data.

Some of the items in the data checklist are not clear which causes delay in design and leads to longer implementation period and lot of other problems. The purpose of the background paper is to initiate discussion on some of the items which need further clarification.

2.3[1/] Site Plan

The following points may be added.

1. In the bore logs, ground levels should be provided.
2. After boring at original site, if the site is abandoned for an alternate location nearby, the location of the earlier borings should also be shown.

1/ The numbers refer to item numbers in design data check list.

3. The bore holes should be located with reference to permanent objects like trees, buildings etc. In the absence of any permanent objects, pillars may be erected for this purpose.

3.2.2 Specify the name of at least one station u/s and one station d/s of the structure site with distances:

U/s station ______________________

U/s distance __________ miles.

D/s station ______________________

D/s distance __________ miles.

<u>Comments</u> : It should be specified whether the stations are located on the outfall rivers ?

<u>Recommendations:</u> The item may be rephrased as given below:

a) Specify the name of at least one gage station u/s and one gage station d/s of the proposed structure site, on the drainage channel, with distances.

b) Specify the name of at least one gage station u/s and one gage station d/s of the proposed structure site, on the outfall river, with distances.

4.1 (a) If the banks of the outfall river and drainage channel at or near proposed structure site are not stable show the movement of the bank in each year in a map.

Enclosed ______________________ No. of map.

(b) What is the average rate of erosion in each year?

Average rate of erosion ______________ ft./year

<u>Comments:</u> Adequate survey data may not be available to show the annual movement of the banks on a map.

Recommendations: The above items may be combined:

Show the location of bank erosions near the proposed structure site and give approximately the average annual rate of erosion.

4.2 Cross-Section of the drainage channel for at least 1/2 mile u/s and 1/2 mile d/s of the structure site at an interval of 200 ft.

Enclosed ____________________ No(s) of Cross-Sections.

Comments: In most cases the drainage channel, downstream of structure, is shorter than ½ mile. If the channel has a regular section, cross-sections at intervals of 200 feet will not be necessary. Moreover sections taken strictly at intervals of 200 feet may miss important features.

Recommendation: The item may be worded as follows.

Supply the cross section of the drainage channel for at least ½ mile u/s and up to the outfall river in the d/s. The intervals between cross sections should be such that all major details (changes in cross section) are provided but the maximum interval should not exceed 500 feet.

4.3 Long section of the drainage channel for at least 2 miles u/s from structure site and upto the outfall river in the d/s:

Enclosed Long Section in ____________________ No (s)

Comments: Longitudinal section for 2 miles u/s of the structure may not be enough.

Recommendation: This item may changed as follows.

Provide longitudinal section of the drainage channel along thalweg (deep water channel), for the total length or 5 miles whichever is shorter, u/s from structure site and upto the outfall river in the d/s.

5.2.3 (a) Is the complete drainage of the basin necessary?

YES NO

(b) If not, mention the drainage level required & distance of such level from the proposed structure site :

Drainage EL. requirement _________ SOB/PWD

Distance from the structure site ____ mile(s)

Comments: The design procedure for pre-monsoon flood routing include polder inundation analysis, using an assumed regulator ventage, for obtaining area inundated to a depth greater than 0.3m (1 foot) for 3 days due to a 10-year 5-day rainfall. The size of the regulator (number of vents) for which the above inundated area is not greater than 5% of the area that would be inundated under "without project" conditions is selected to the required size.

The information in (a) is not used in the design. The need for complete drainage depends on a number of factors i.e. topography, cropping, land use, fisheries etc. Moreover BWDB field engineer should not make that decision without consultation with beneficiaries. Input from agronomists would be useful.

Item (b) is used in fixing the invert level for drainage regulators. Similar to (a), BWDB field engineers should not make decision alone.

5.2.4 From the field condition propose the invert level of the structure which can allow desired level of drainage from field condition.

(a) Proposed Invert Level ________________ SOB/PWD

(b) Invert level of nearby existing Structure(s) ________________ SOB/PWD

Comments: The invert level should be based on the area-elevation curve, longitudinal slope of the drainage channel and the water level of the outfall river. The field office may be asked to give their opinion on the proposed invert level, but the final decision should be taken by the design office on the basis of above data and analysis.

Recommendation: Under the above circumstances items 5.2.3 and 5.2.4 may be combined as suggested below.

Considering field conditions propose an invert level for the structure which would allow desired level of drainage.

5.2.5 From field condition, what is the maximum level of acceptable flooding on the basin during drainage period?

Acceptable flooding level ________________ SOB/PWD

Comments: Is this for pre-monsoon period?
On what basis the field engineer will specify the above?
Is this information used in design?

Recent studies indicate that paddy can stand submergence of upto 0.2 m (0.7 foot) for a period of 2 days in muddy water or for a period of 4 days in clear water. If the level of acceptable flooding causes submergence, the duration of flooding should be specified.

Recommendation: BWDB should set its own criteria for maximum level of acceptable flooding which could be the tolerable flooding depth for paddy.

5.2.6 Desired Post Monsoon Drainage Level:

Date ________________ Level ______________ SOB/PWD
Date ________________ Level ______________ SOB/PWD
Date ________________ Level ______________ SOB/PWD

Comments: This would also depend on transplanting date and growth stage of paddy. Using the revised criteria of item 5.2.5 and the transplanting dates of paddy in the project area, the desired post-monsoon drainage levels may be fixed.

Post-monsoon drainage rate determine the scope of growing rabi crops. In order to grow these crops, all water in excess of field capacity must be removed early in the rabi season for timely sowing.

Recommendation: The item may be deleted from checklist and design office may decide about the desired post-monsoon drainage level on the basis of the following:

1. submergence tolerance of the type of paddy grown.
2. rate of drainage which will permit timely growing of rabi crops.

5.3.1 Specify the total cultivable and irrigable area within the project.

Cultivable Area ____________________ Acres.
Irrigable Area ____________________ Acres.

Comments: Irrigation methods should be specified.

Recommendation: This item may be changed to specify the total cultivable area and irrigable area (by methods) within the project.

Cultivable Area ....

Irrigable Area ....

Irrigation Methods	No.	Area
DTW		
STW		
LLP		
Traditional		
Others		

5.3.3 Proposed retention level of water in the u/s of the drainage channel of the structure for irrigation.

Retention level ________________________ SOB/PWD

Comments: The purpose of retention of water in the drainage channel, as normally stated in the appraisal report, is two fold: firstly to retain water in the beels for supplemental irrigation during later part of Aman and secondly to store water in the drainage channel for irrigation of HYV Boro in Rabi season.

Let us consider the Bannyar Khal (Figure 1). What could be the desired retention level? Would it be the lowest level (say 7.6 m), in that case the higher grounds may not be benefitted? Would that be an intermediate level (say 10.0 m) which may flood the lower lands.

Who should decide about this retention level.

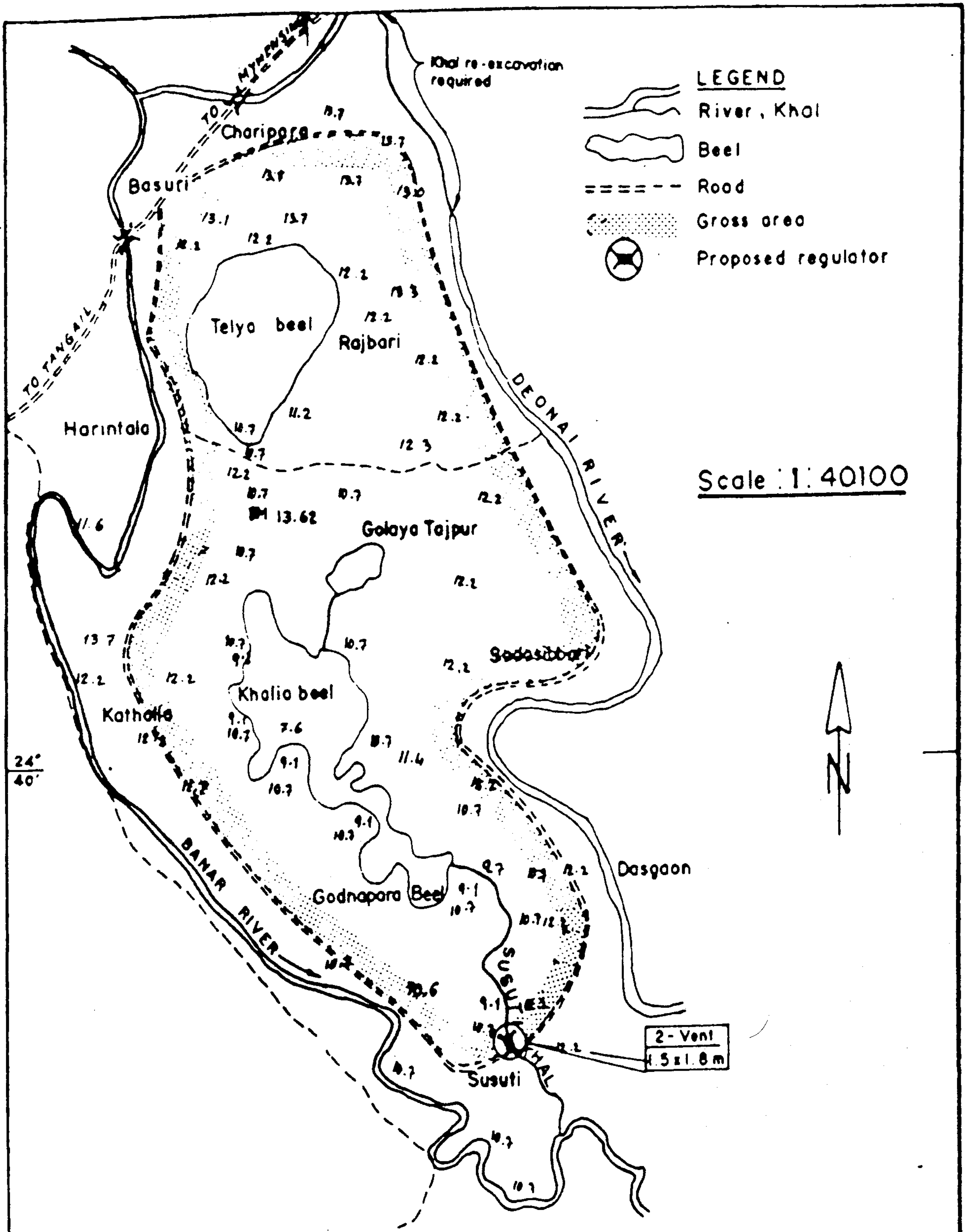
Secondly the benefit from water storage in drainage channel may not be significant. Consider a canal one mile long (5280 feet), 100 feet wide and 10 feet deep, it would hold

$$\frac{5280 \times 100 \times 10}{43560} = 121.2 \text{ Acre-feet of water.}$$

Assuming a low rate of water loss of 20% due to evaporation, seepage and deep percolation and a water requirement of only 3.25 feet (39 inches), 121.2 acre-feet of water would irrigate only 29.8 acres of Boro rice. A larger area may be irrigated if wheat or cash crops like mustard are cultivated. In many areas bed and banks of drainage channels are cultivated during dry season and this benefit will be lost if water is stored in channels.

Dead channel storage without any perennial flow will not be feasible for cultivating Boro rice.

<u>Recommendation:</u> Retention level in post-monsoon season should not exceed, under normal conditions, the bank level of drainage channel. Dead channel storage should not be permitted. These points may be added to 5.3.3.



REGULATOR FOR BANNYAR KHAL

Sub-Project No. 6

PROJECT MAP

Figure 1

ADDITIONAL POINTS

<u>Survey Datum:</u> Elevations are to be provided by using either SOB (Survey of Bangladesh) or PWD (Public Works Department) datum. The idea is that the field engineers, while submitting data, will strike out the one which does not apply. In many cases it was observed that elevations were provided without any reference to the datum. At times it was suggested that, instead of keeping scope for using either SOB or PWD datum, ask for elevation data in reference to only one datum say PWD datum.

<u>The Design Strengths of Concrete and Steel</u>: The ultimate compressive strength of concrete (fc), as used by different design circles of BWDB, varies from 2,500 to 2,800 psi.

Similarly the allowable tensile strength of steel (fs) varies between 18,000 to 20,000 psi.

It is recommended that fc and fs values may be standardized.

<u>Stilling Basin Design:</u> The flow conditions in majority of the small scale hydraulic structures in Bangladesh may be considered to be in the transition range (Froude numbers between 2.5 and 4.5) because a true hydraulic jump does not fully develop. Stilling basins that accommodate these flows are the least effective in providing satisfactory dissipation because the attendant wave action ordinarily cannot be controlled by the usual basin devices.

Presently, BWDB design circles use the following types of basin.

1. Low Froude Number Stilling Basin (USBR).
2. USBR Type IV Basin.
3 Indian Standard Stilling Basin I.

The low Froude Number basins were developed after USBR type IV basins. The type IV basin use large chute blocks. The first and third categories are very similar.

Considering their advantages and limitations, it is recommendated that BWDB may specify the conditions for which the above basins may be used.

<u>Metric Units:</u> Metric units may be used in the revised data check list.

CHAPTER FOUR

HYDROLOGIC DESIGN

A. N. M. WAHEDUL HUQ

HYDROLOGIC DESIGN

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1.0 HYDROLOGIC DESIGN

The purpose of a drainage sluice is to drain off excess water from an area and it should be designed in such a manner so that the expected inflow of water is drained out quickly and efficiently, with minimum damage to property and inconvenience to inhabitants, at a reasonable cost.

One of the steps in the hydrologic design of drainage regulators is the determination of peak flow to be discharged. Simple methods using graphs and charts are available for computation of peak flow in small basins (say less than 1000 ha). Elaborate methods have been developed for calculating design runoffs in larger basis.

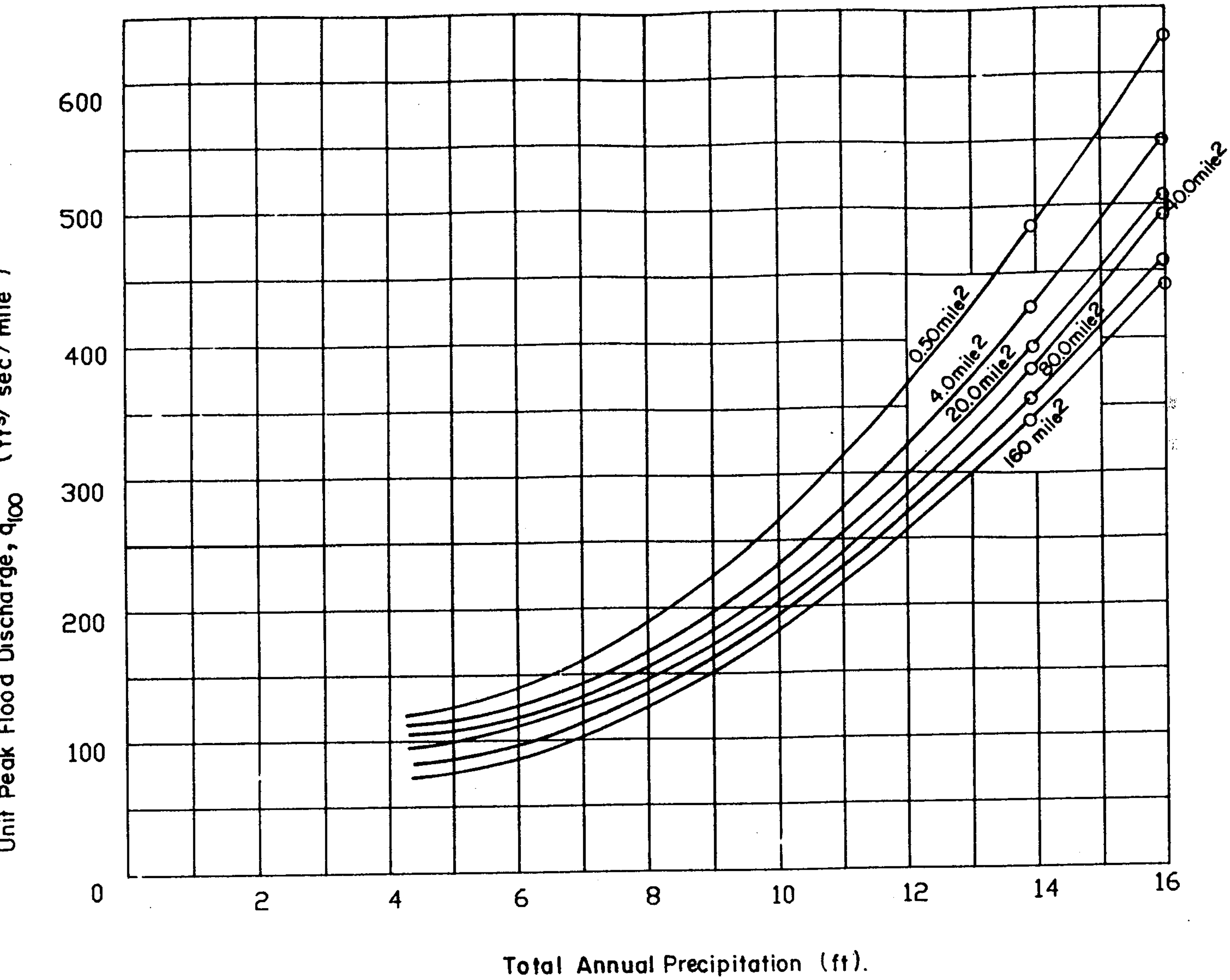
2.0 DESIGN DISCHARGE IN SMALL BASINS

Some of the simple methods for computing peak flows in small basins are discussed in this section.

2.1 IECO Method

Figures 1, 2 and 3 were developed by IECO for estimating flood flows in small basins of Bangladesh corresponding to return periods of 2 to 500 years. Computational procedure is explained below:

(i) Obtain the total annual precipitation for the project area and enter **Figure** 1 to obtain the Q_{100} unit peak flow. Multiply this value by the coefficient given in the table on **Figure** 1 to obtain the peak unit flow for the selected return period (10 years for sluices and water retention structures).



Factor for Other Flood Frequencies

$q_{500} / q_{100} = 1.14$
$q_{200} / q_{100} = 1.06$
$q_{50} / q_{100} = 0.93$
$q_{25} / q_{100} = 0.85$
$q_{15} / q_{100} = 0.79$
$q_{10} / q_{100} = 0.73$
$q_{5} / q_{100} = 0.62$
$q_{2} / q_{100} = 0.40$

Region	Rainfall
Rangpur	7.5 feet
Kurigram	7.5 "
Chittagong	9.2 "
Cox's Bazar	12.5 "
Dhaka	6.7 "
Khulna	5.8 "
Sylhet	9.2 to 16.7 ft.
Mymensingh	8.0 "
Barisal	7.5 "
Jessore	6.0 "

Figure 1. Unit Peak Discharge for 100 year Return Frequency

Source: Karnafuli Irrigation Project

International Engineering Co. Inc. (May, 1980)

(ii) Knowing the length of the main water course and the slope of the basin compute $(L^2/S)^{0.3}$ and enter **Figure** 2 to obtain the percentage adjustment to be applied to the peak flood flow rate.

(iii) Enter **Figure** 3 with the basin area and obtain the percentage error as a function of basin area.

(iv) Multiply the unit peak flow by the values obtained in (ii) and (iii) to arrive at the unit peak flow rate.

(v) Multiply the value obtained in (iv) by the basin area to obtain an estimate of the peak flood flow for the basin.

2.1.1 Example : (IECO Method)

Basin Area = 4 sq. mile

Length of water course, L = 2 mile

Slope of water course, S = 0.722 ft/mile

= 0.0001367 ft/ft

Let the basin be located in Jessore district.

From **Figure** 1, Total Annual rainfall = 72" (6 ft)

q_{100} = 130 cfs per sq mile.

q_{10} = 130 x 0.73 = 95 cfs per sq mile.

$$\left[\frac{L^2}{S}\right]^{0.3} = \left[\frac{4}{0.0001367}\right]^{0.3} = 21.87$$

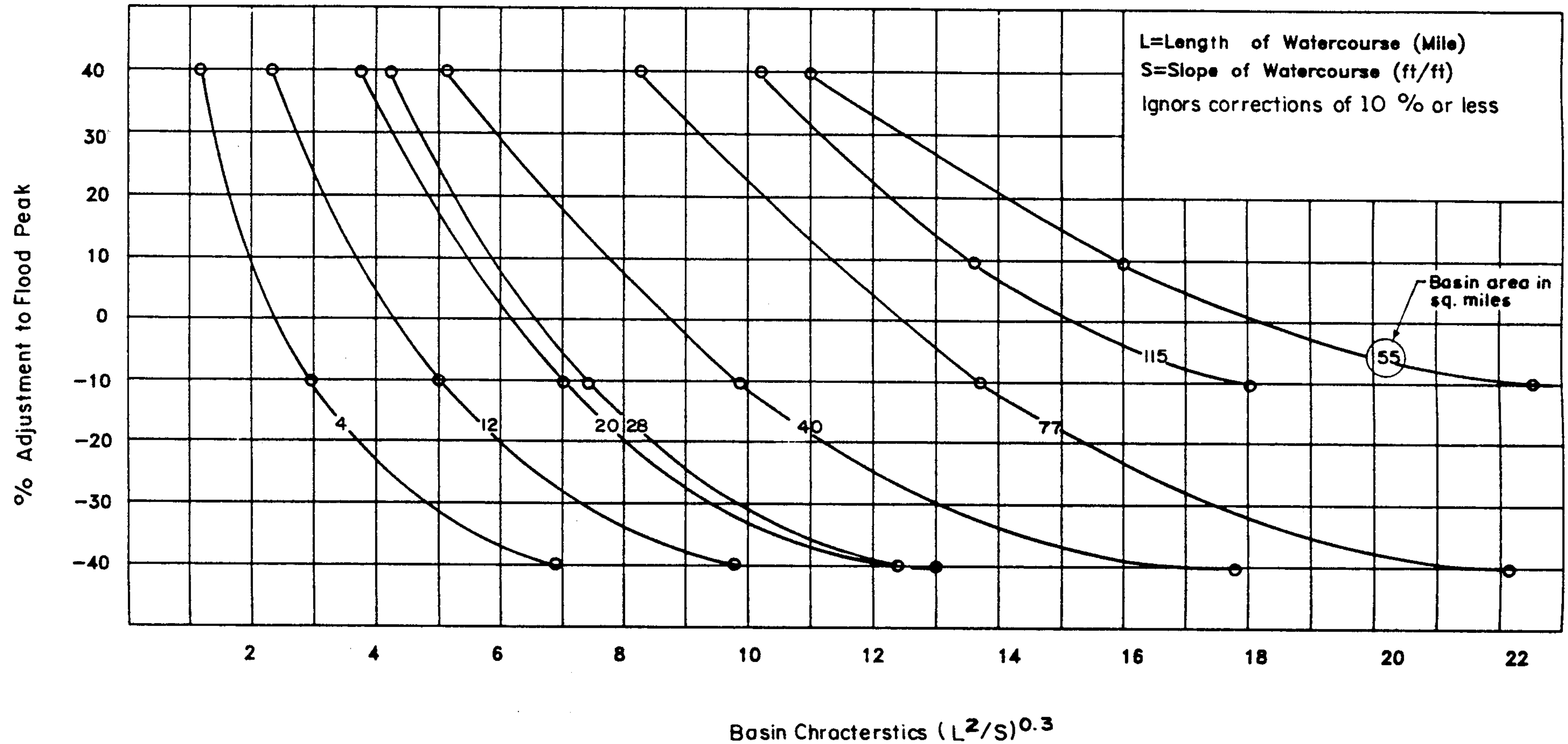


Figure 2. Estimated Peak Adjustments for Basin Slope and Shape Characterstics

Source: Karnafuli Irrigation Project

International Engineering Co. Inc.

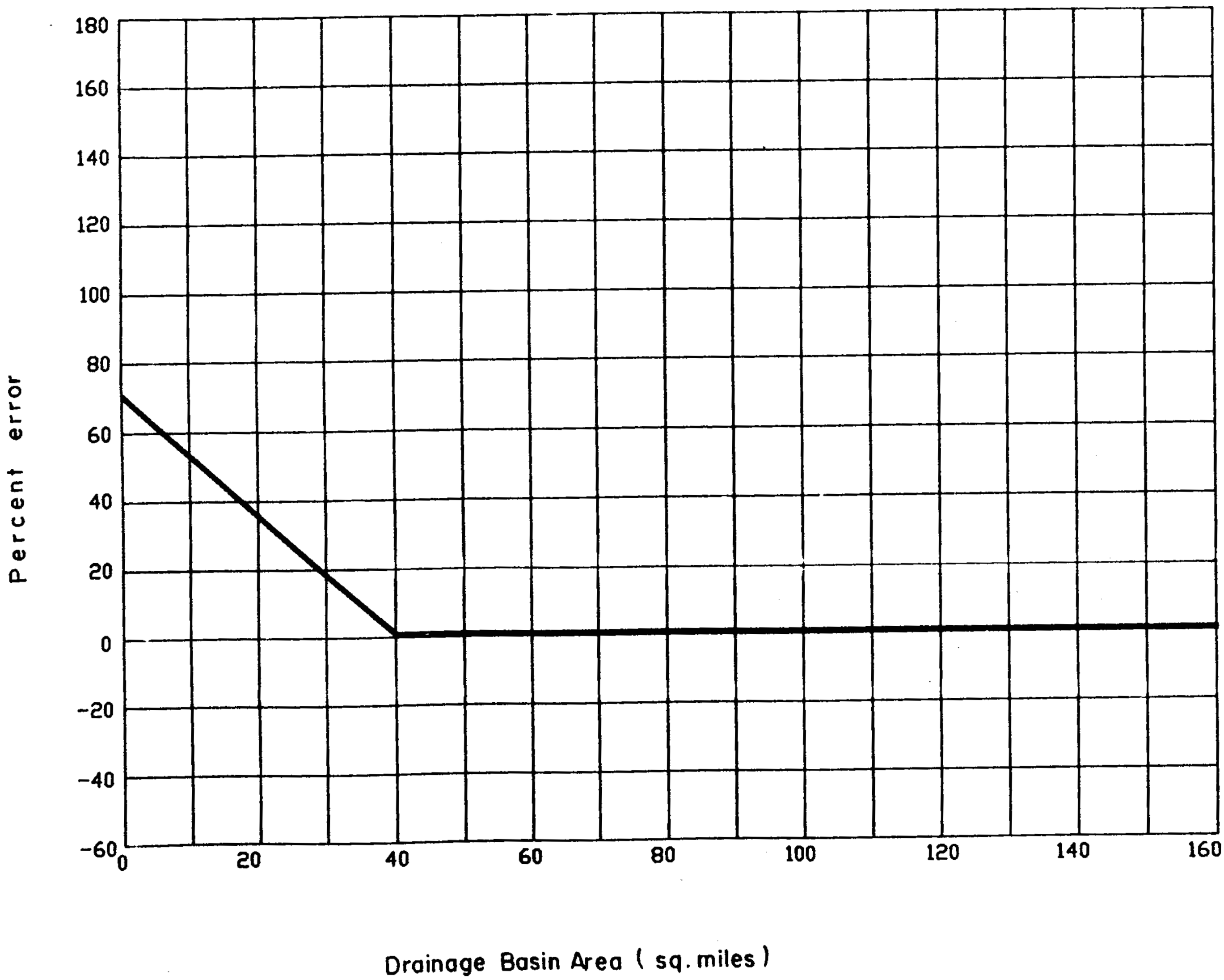


Figure 3. Error as a Function of Drainage Area

Souurce : Karnafuli Irrigation Project
International Engineering Co.Inc.

From Figure 2, the adjustment is - 40%

q_{10} = 95 - (95 x 0.4) = 57 cfs/sq mile

From Figure 3, the correction is + 66%

q_{10} (corrected) = 57 x 1.66 = 95 cfs/sq mile

For 4 sq miles

Q = 95 x 4 = 380 cfs.

= 10.77 m^3/sec.

2.2 The Rational Formula (Q = CiA)

Lack of records for rainfall intensity, runoff and stream flow have led to the development of empirical formula for estimating flood flow. One of the leading formulae for estimating peak runoff is described below:

$$Q = CiA$$

Where,

Q = Peak discharge in cfs

i = Rainfall intensity in inches/hour

$$= \frac{68.2}{t_c^{0.73}}$$

tc = Time of basin concentration in minutes.

$$= 31 \left[\frac{L^2 \ n^2}{S}\right]^{0.30}$$

C = A runoff coefficient

$$= 0.7 \left[\frac{T_P}{100}\right]^{0.18}$$

T_P = Recurrence period

A = Basin area in acres

2.2.1 Example (Q = CiA)

Basin Area, A = 4 sq mile = 2560 acres
Average channel slope, S = 0.722 ft/mile
Length of drainage channel, L = 2 miles
Channel Roughness Factor, n = 0.055

$$tc = 31\left[\frac{L^2 n^2}{S}\right]^{0.3}$$

$$= 31\left[\frac{2^2 \times 0.055^2}{0.722}\right]^{0.3}$$

$$= 9.09 \text{ hours}$$

$$= 545.5 \text{ minutes}$$

$$i = \frac{68.2}{(t_c)^{0.73}} = \frac{68.2}{(545.5)^{0.73}}$$

$$= 0.685 \text{ inch/hour.}$$

$$C = 0.7 \left[\frac{T_p}{100} \right]^{0.18}$$

$$= 0.7 \left[\frac{10}{100} \right]^{0.18}$$

$$= 0.463$$

$$Q = C\ i\ A$$

$$= 0.463 \times 0.685 \times 2560$$

$$= 812 \text{ cfs}$$

$$= 23 \text{ m}^3/\text{sec}$$

2.3 Drainage Modulus

Drainage Modulus or the drainage rate of the basin expressed in inch or mm. per day is used to determine the design discharge of the drainage channel. During monsoon the main and the secondary drainage canals, sometimes even the tertiary canals, remain flooded by the rainwater stored in the basin and the monsoon drainage is not effective due to high river stage unless drainage pumps are installed. During monsoon only the channels lying above basin storage level will be effective. These channels are mostly farm ditches or minor channels which are usually developed by the farmers and do not require much detail in design.

During pre-monsoon period i.e, in May and June, the low stage of the outfall river will permit gravity drainage and the drainage will be effective through main and secondary drains. As such the pre-monsoon storm i.e, the storm for May or June will be used to determine the drainage modulus of a basin.

2.3.1 Example : (Drainage Modulus)

Rainfall station : Narail

<u>1 in 10 year Design Rainfall (in mm)</u>

Days ---->

1	2	3	4	5	6	7	8	9	10
130	30	29	23	23	19	16	16	12	10

Rainfall losses due to evapotranspiration & deep percolation = 5 mm/day.

Rainfall Excess (in mm)

Days ---->

1	2	3	4	5	6	7	8	9	10
125	25	24	18	18	14	11	11	7	5

Cumulative (in mm)

125	150	174	192	210	224	235	246	253	258

Assumptions

1) Pre-monsoon storm occurred in the month of May when paddy height is 300 mm

2) Allowable initial paddy storage = 50 mm

3) Maximum tolerable depth of flood for paddy in the month of May is 150 mm

In **Figure** 4, the excess rainfall is plotted over allowable paddy storage and the 1 in 10 year Flood curve is drawn. Now to keep the basin water level within tolerable depth of flooding for paddy, the excess rainfall runoff over 150 mm in the basin have to be drained. To find the design discharge of a drainage channel, the drainage modulus of the basin is found by drawing a tangent to the flood curve from the 150 mm point of the rainfall excess. The slope of the tangent line represents the drainage modulus of the basin. To find the value of the drainage modulus a line parallel to the tangent line passing through the origin is drawn. The parallel line to the tangent shows that 25 mm of rain has to be drained in 1 day i.e, the drainage modulus of the basin under the influence of Narail rainfall station is 25 mm per day.

In absence of detail rainfall records the drainage modulus or the design runoff of a basin to determine the design discharge of a drainage channel can be readily obtained from the isohyetal map presented in **Figure** 5. The map has been prepared by LDL based on rainfall intensity, duration and frequency data published in the Master Plan prepared by IECO.

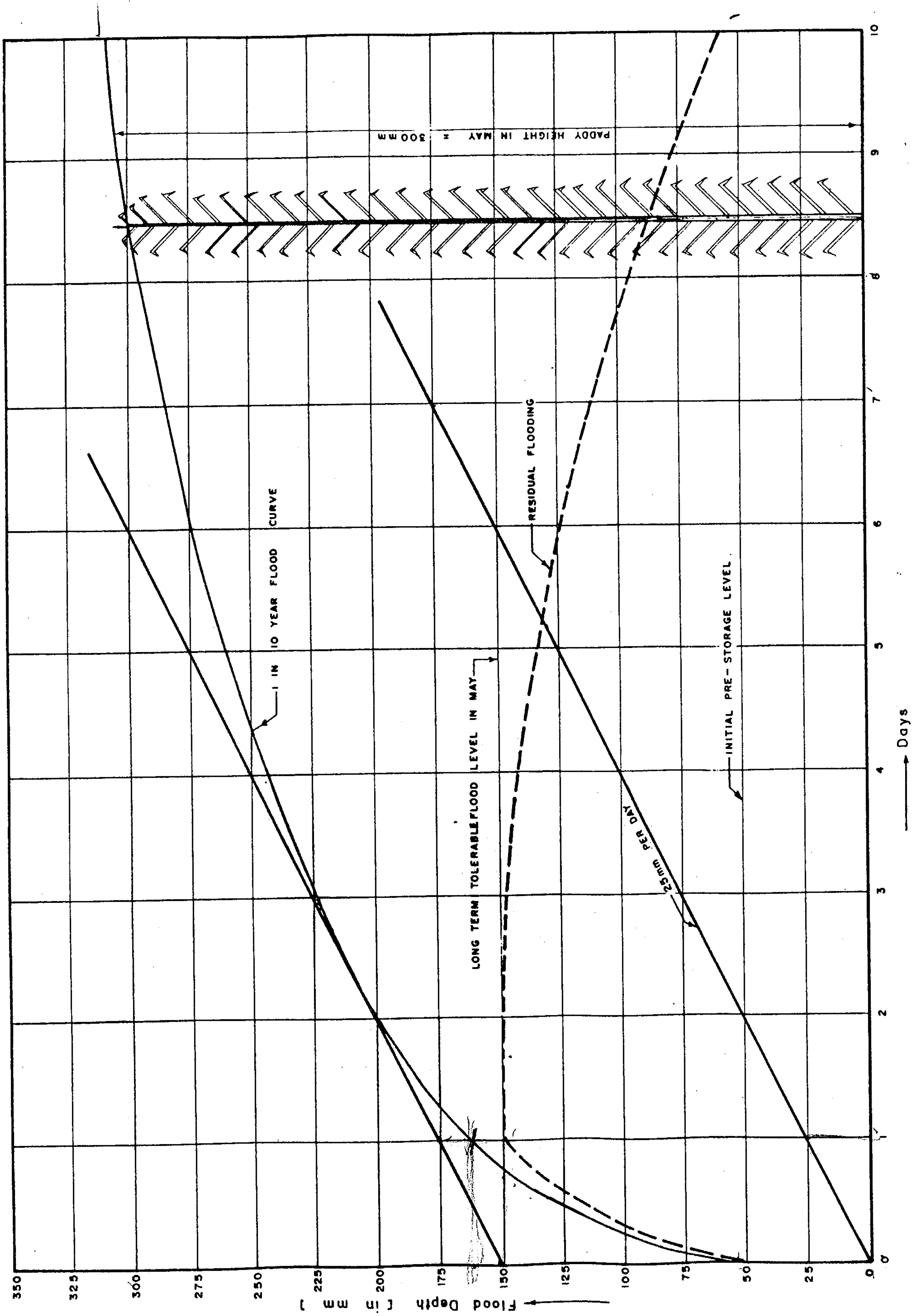
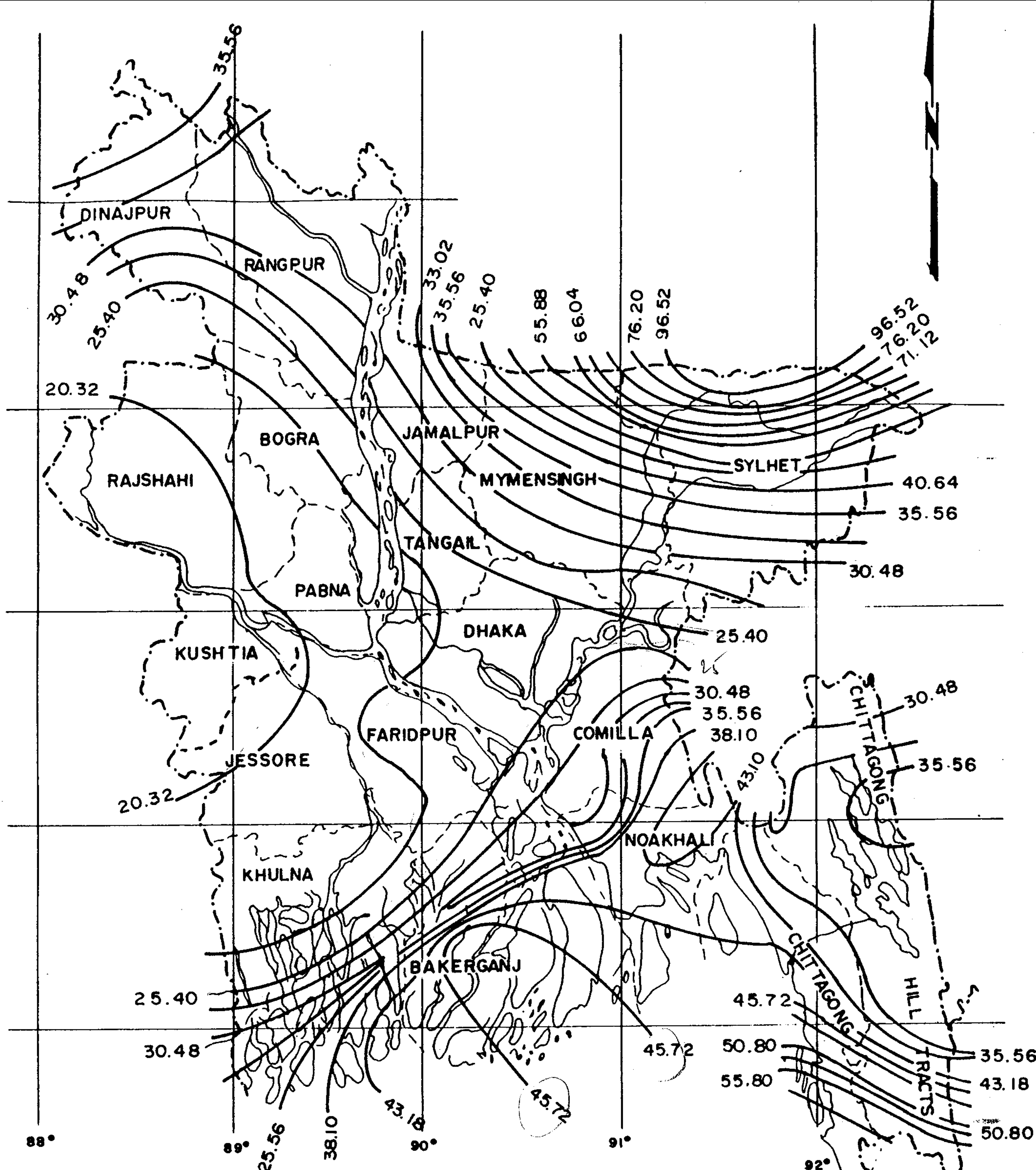


Figure 4. Drainage Modulus



NOTE:

1. Runoff rates shown are based on rainfall intensity, duration and frequency data published in the IECO water plan .....1964.

2. Losses from rainfall for evapo-transpiration, storage, seepage, etc. are assumed to be 50% of the first 50 mm of rainfall and 50% to 40 % of rainfall in excees of 50 mm for flat and hilly areas respectively. LDL 1968)

Figure 5 Design Runoff for Drainage Sytems in Bangladesh (mm/day)

3.0 HYDROLOGIC DESIGN FOR LARGER BASINS

Elaborate methods are used for computing the runoffs in larger basins.

The following steps are involved.

1. Develop design storm
2. Calculate losses and compute rainfall excess
3. Develop unit hydrograph
4. Compute the runoff hydrograph
5. Determine the basin water level by routing the runoff hydrograph through an assumed size of sluice. If the computed basin water level is not acceptable, give trials with revised sluice sections until the desired basin level is achieved. The size of the sluice corresponding to the desired water level is the design size.

3.1 The Design Storm

During monsoon the rainfall is not continuous; rather it falls in intermittent showers which may be intense and may occur several times during a day. Downpours of heavy rain are generally localized with normal rainfall intensity occurring a short distance away. Pre-monsoon rain occur during April and May, and have the characteristics of strong northwest wind with heavy thunder storms, usually of short duration.

It is uneconomical to design hydraulic structures for the maximum storm that may ever occur. The structures are designed instead with the expectation that those will be subjected to flows greater than design flow once in 10, 15 or 25 years on the average. Therefore, for flood and drainage studies it is necessary to know the frequency of intense rainfall for varying periods of time.

3.1.1 Index Rainfall

IECO developed several graphs for the computation of design storm. The first one is Figure 6 which presents the total mean rainfall that may be expected to occur during four calender month period of maximum rainfall. Two other Figures are included in the IECO study (Figures 7 and 8). Figure 7 may be used to convert the total four-month rainfall to total rainfall over a selected number of days and Figure 8 may be used to convert index frequency to a selected frequency. Since most of our studies will require a rainfall duration no longer than 5-days and frequencies of either 10-years or 25-years; the Figures 7 and 8 may be combined in the following tabular form.

Table 1. Combined Rainfall Index for Selected Frequencies

Days	Storm frequencies	
	10 Years	25 Years
1	0.128	0.153
2	0.192	0.228
3	0.230	0.272
4	0.257	0.303
5	0.276	0.326

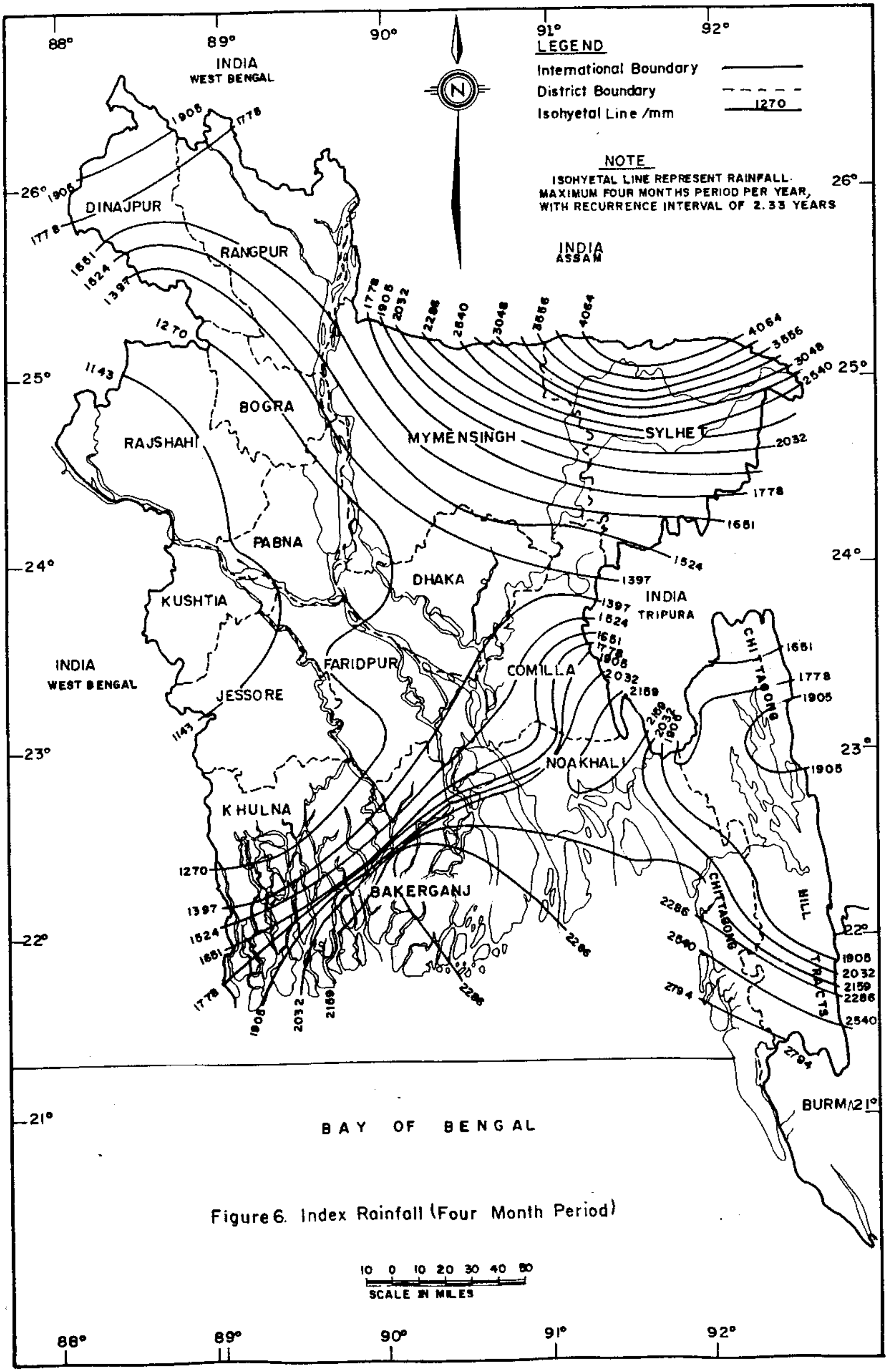


Figure 6. Index Rainfall (Four Month Period)

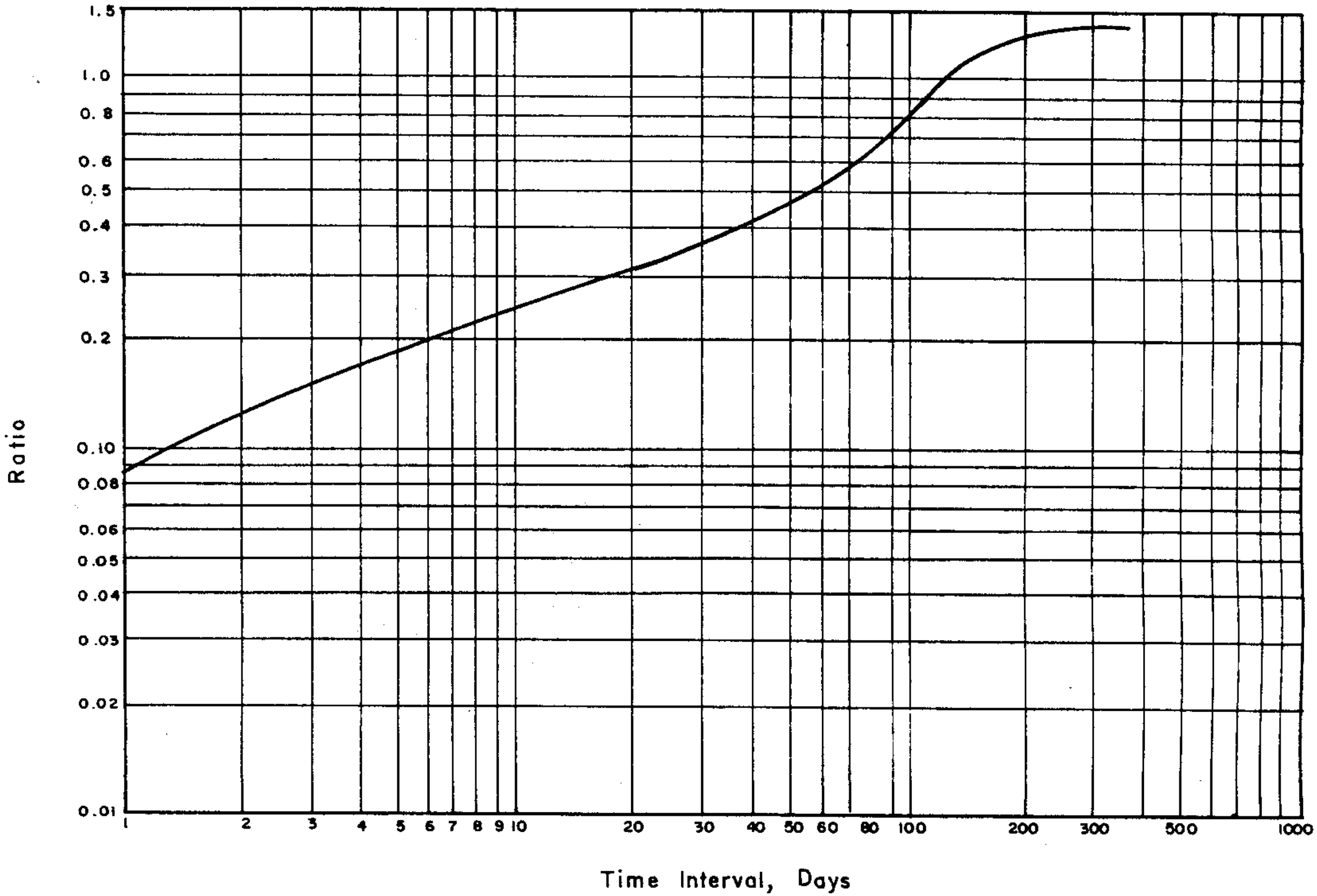


Figure 7. Ratio of Index Rainfall for a Given Time Interval to Index Rainfall for Four Months

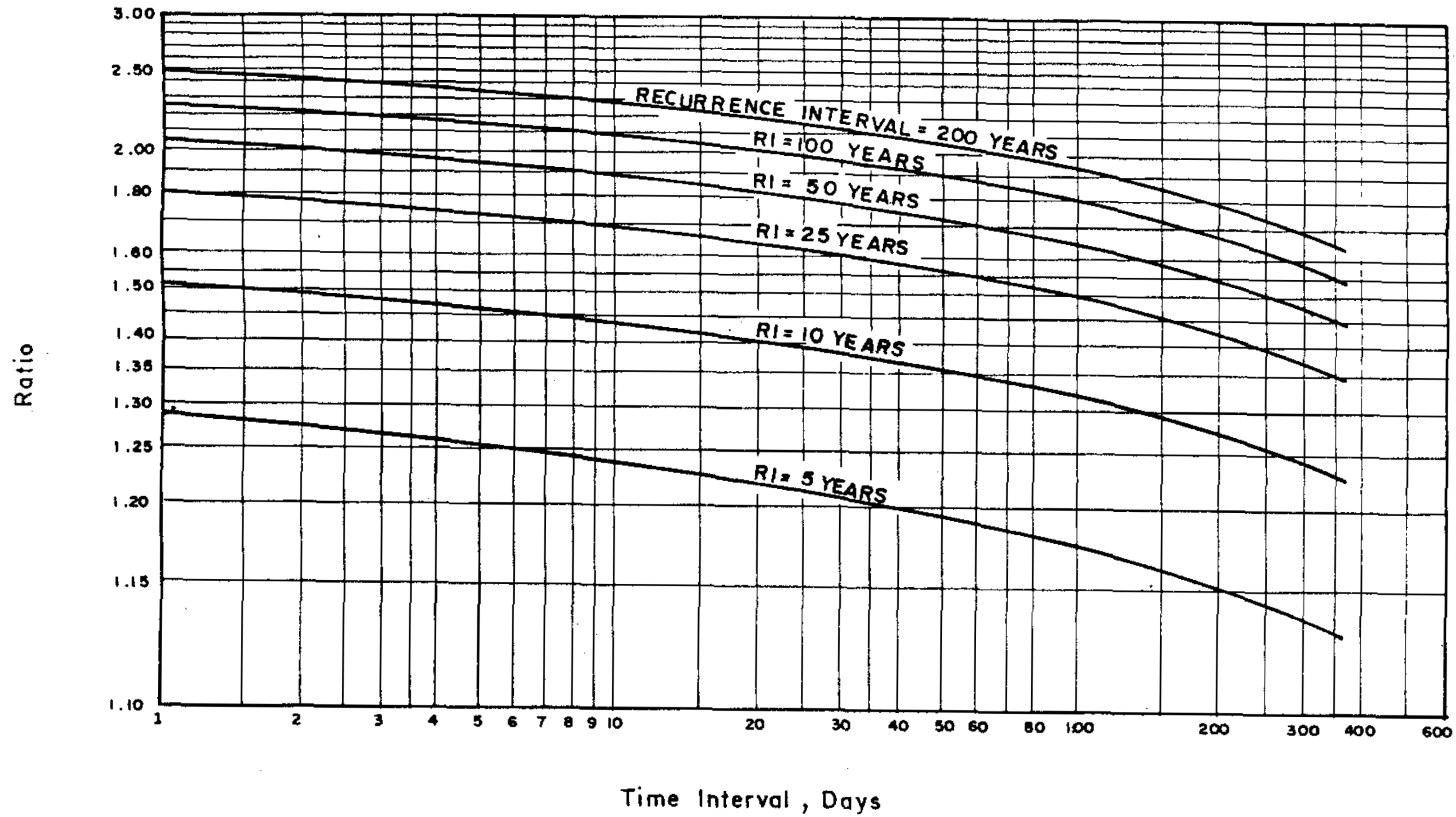


Figure 8. Ratio of Rainfall for Selected Recurrence Intervals to Index Rainfall for Indicated Time Interval

3.1.2 Point Rainfall

Point rainfall is the amount of rain falling at a specific point, usually measured at a rain gauging station. In this case point rainfall may be defined as the volume of rainfall, at the center of the storm, equal to the product of the four month rainfall index and the combined rainfall indices from Table 1.

If point rainfall were used as the basis for designing a structure, the computed runoff would be high resulting in increased project costs. Rainfall intensity decreases as the distance from the storm center increases and it becomes zero at the edge of the storm.

The rate of decrease of a storm's intensity away from its centre for Bangladesh is given in Figure 9. Table 2 shows this relationship in tabular form.

Table 2. Rainfall Variability

Average distance from Storm Centre (In miles)	5-day Storm Percentage of Point Rainfall
¼	100.0
1	88.0
2	81.7
3	77.5
4	74.2
5	72.0
6	69.8
7	67.8

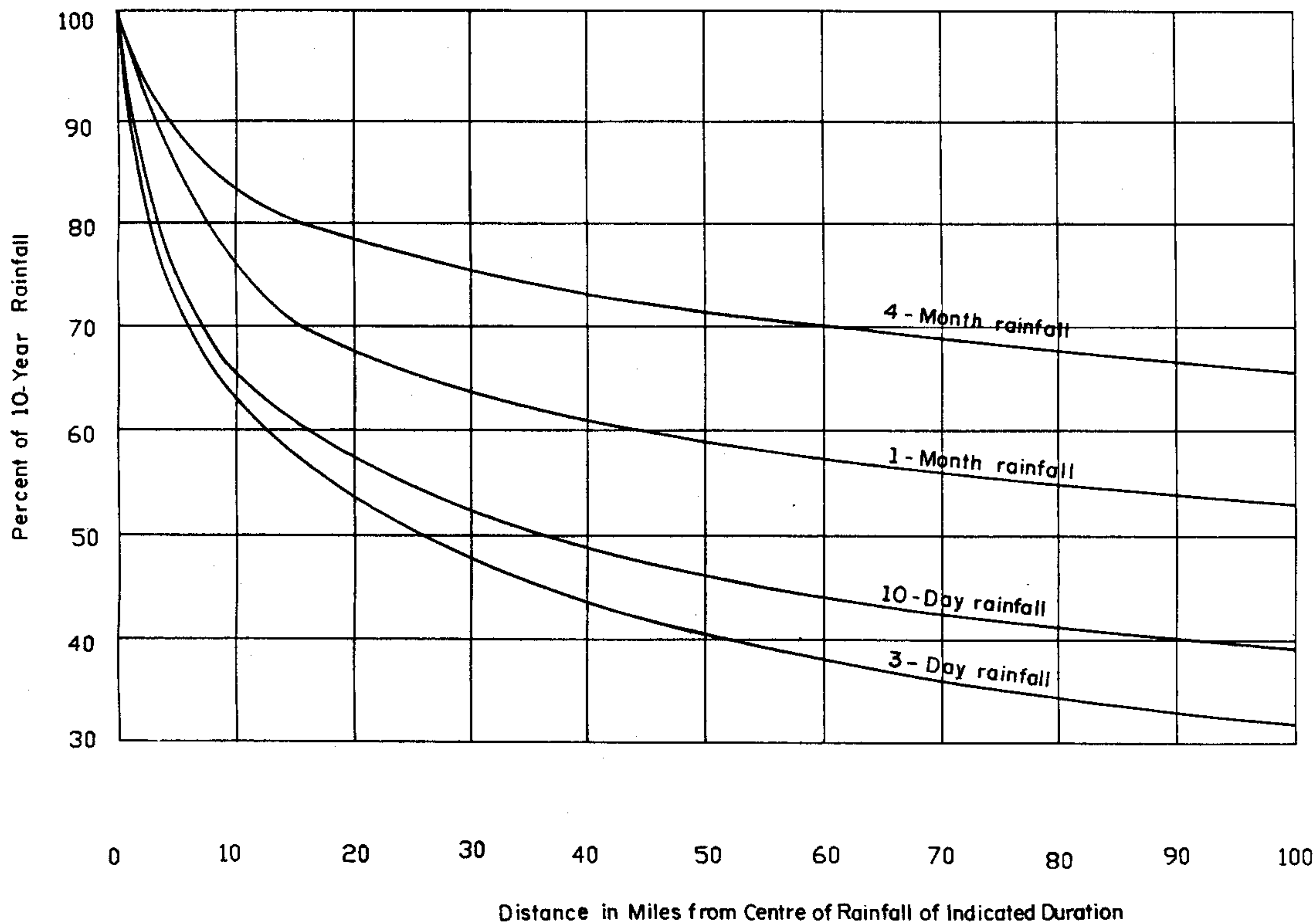


Figure 9. Rainfall Variability with Distance

3.1.3 Equivalent Uniform Depth of Rainfall

The equivalent uniform depth of rainfall is defined as the depth of water which results from spreading the total volume of basin rainfall uniformly over the total basin area.

The procedure for determining uniform equivalent depth of rainfall is:

1. Locate the geographical centre (centroid) of the basin. For locating the centroid, a card board is cut to the shape of the basin. The card board is suspended vertically from three or more points by threads. After holding the free ends of the threads together by hand, the lengths of threads are adjusted to make the board horizontal.

   The centroid is located on the card board vertically below the point of intersection of the threads for a horizontal position of the board. The centroid location may be determined by suspending a plumb-bob from the intersection point of all the threads.

2. Draw concentric circles, or isohyetals, at one mile intervals around the center starting one-half mile from the centre.

3. Planimeter the areas between the respective isohyetals within the basin area.

4 Multiply the area by the proper percentage from table 2.

5. Make a summation of the values from step 4 and divide by the total basin area.

6. The result is a percentage which when applied to point rainfall yields the equivalent uniform depth of rainfall.

3.1.4 Example

Determine the probable 5-day rainfall from a storm with 10 and 25-year frequency for the Mrigi Khal basin in Faridpur District.

A. Point Rainfall

1. From Figure 6, the 4-month rainfall index is 1270 mm.

2. Using the combined rainfall indices from Table 1, the accumulative 5-day rainfall is calculated in Table 3.

Table 3. Accumulative Point Rainfall

Days (1)	10 years storm frequency (2)	Accumulative point rainfall (mm)	
		From 4 month rainfall index (3) = (2) x 1270 mm	From actual data
1	0.128	162.56	132.13
2	0.192	243.84	168.83
3	0.230	292.10	194.76
4	0.257	326.39	220.39
5	0.276	350.52	244.69

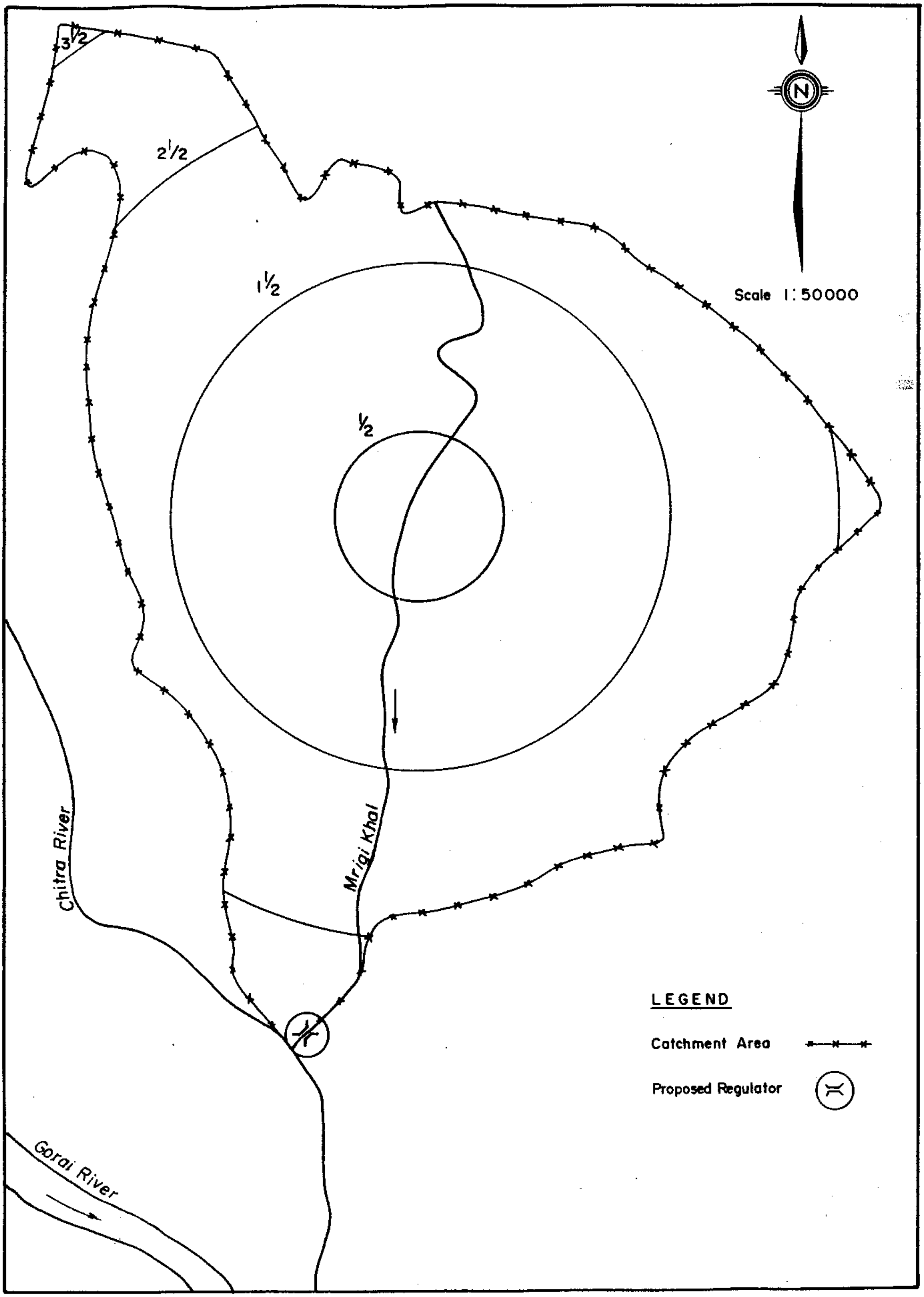


Figure 10. Isohyetals for Calculating Equivalent Uniform Depth

B. Equivalent Uniform Depth Of Rainfall

1. Figure 10 is a map of the basin area with its geographical center located and isohyetals drawn at one mile intervals starting one-half mile from the center.

2. Areas between isohyetals have been planimetered and the percentages from Table 2 were applied (Table 4).

Table 4. Percentage of Point Rainfall

Average distance from storm centre	Area in sq. miles	Area times percentage
½ mile	0.79	0.79
1 mile	6.21	5.47
2 mile	7.83	6.40
3 mile	1.78	1.38
4 mile	0.12	0.09
	16.73 (43.33 sq. km)	14.13 (36.60 sq. km)

3. Compute the percentage of point rainfall that forms the equivalent uniform depth of rainfall:

$$\frac{36.6}{43.33} \times 100 = 85\%$$

4. Convert accumulative point rainfall from Table 3 to daily uniform depth equivalents by multiplying the values by 0.85 (Table 5).

Table 5. Equivalent Uniform Depth Of Rainfall

Days (1)	Equivalent uniform depth		
	Accumulative		Daily increments (in mm) (4)
	point rainfall (in mm) (2)	uniform depth (3) = (2) x 0.85	
1	162.56	138.18	138.18
2	243.84	207.26	69.09
3	292.10	248.29	41.02
4	326.39	277.43	29.14
5	350.52	297.94	20.51

3.1.5 Daily Rainfall Sequence of Equivalent Uniform Depth

The daily increments of rainfall shown in Table 5 can occur in any order. An arbitrary sequence of 3, 2, 1, 4, 5 is considered in order to satisfy the losses that will take place early in the storm. A graphical arrangement of this sequence is presented in **Figure 11.**

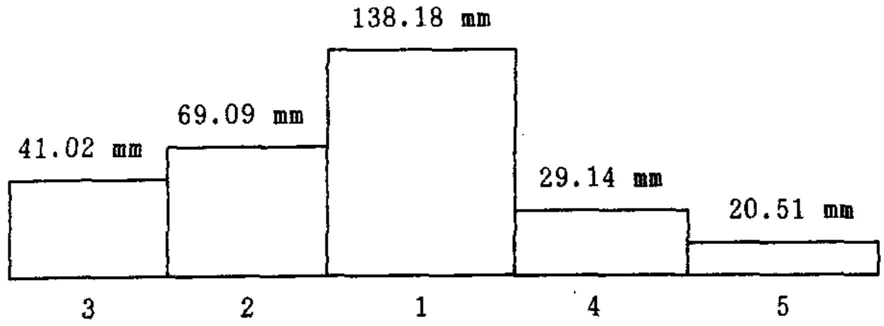


Fig. 11 : Daily rainfall sequence of uniform equivalent depth

3.1.6 Rainfall Time Distribution

For further calculation we will wish to know the quantity of rain falling during certain hourly intervals. As an aid for breaking down rainfall time distribution into intervals less than 24-hours, Table-6 has been prepared from the Master Plan rainfall data prepared by IECO. A reasonable 24-hour rainfall time distribution can be made for any basin by using the particular rainfall data closest in distance to the basin being studied and adjusting it to fit the calculated 24-hour basin rainfall. The smallest time interval to be used will depend on the length of the main drainage course within the basin. The following intervals are recommended:

1. **Less than 3.22 km (2 miles) : 1 hour**

2. **3.22 km (2 miles) to 9.65 km (6 miles) : 3 hours**

3. over **9.65 km (6 miles) : 6 hours**

Table 6. 24 Hour Point Rainfall Time Distribution

4-Month Index(mm)		Storm Duration in Hours Accumulative rainfall in mm						
		1	2	3	6	12	18	24
Barisal	1651.00	59.18	70.36	78.99	97.03	136.14	173.99	212.09
Jessore	1168.40	60.20	76.20	84.07	96.52	112.52	130.81	149.86
Cox's Bazar	2921.00	59.18	73.15	84.07	111.76	168.91	266.70	358.14
Chittagong	2184.40	61.72	77.98	90.93	115.82	169.42	228.60	279.40
Dhaka	1397.00	78.74	101.60	139.70	133.35	154.94	167.64	179.32
Bogra	1473.20	86.36	107.44	119.38	138.94	160.02	175.26	187.96
Sylhet	3048.00	72.39	105.41	127.00	156.46	208.28	304.80	388.62

The length of natural drainage channel = 14.48 km > 9.65 km. Therefore the 24-hour time distribution is to be made 6 hourly. The rainfall data located closest to project area is Jessore district, so an adjustment must be made. The adjustment will be made on the basis of the ratio of the respective 4 month rainfall index, that is, 1270/1168.4. Table 7 shows the adjustment procedure.

Table 7. Mrigi Basin 24 HR Rainfall Time Distribution

	Hours			
	6	12	18	24
1. Jessore maximum accumulative point rainfall	96.52	112.52	130.81	149.86
2. Project area maxm. accumulative point rainfall (1 x 1270/1168.4)	104.91	122.30	142.18	162.89
3. Project area max^m incremental point rainfall	104.91	17.39	19.88	20.71
4. Project area uniform equivalent depth(3x0.85)	89.16 (1)	14.73 (2)	16.76 (3)	17.53 = 138.18 (4)

As it has been pointed out for rainfall time distribution with 24 hours increments, the increments within a 24 hour period also can not be predicted. Therefore a sequence of 3,4,1,2 is arranged arbitrarily. Thus during the maximum 24 hour rainfall period the 6 hour incremental rainfall is 16.76 mm, 17.53 mm, 89.16 mm and 14.73 mm which is the sequence for the 3rd day. The sequence for the other days are established in the following way.

1) Moving to the 2nd day use the same increments for 1st, 2nd and 4th periods as were used for the 3rd day.

2) For the 3rd period of the 2nd day we will assign the remainder of the total 2nd day rainfall or 20.07 mm.

3) The 1st day is similarly handled with the remainder of 0.00 mm in the 3rd period.

4) The 4th and 5th days are similarly handled and the design storm is complete (Table 8)

Table 8. Mrigi Basin Design Storm (in mm)

Day	6 Hour Increments
1	16.76 17.53 0.00 6.73
2	16.76 17.53 20.07 14.73
3	16.76 17.53 89.16 14.73
4	16.76 0.00 0.00 12.38
5	16.76 0.00 0.00 3.75

4.0 RAINFALL EXCESS

4.1 Soil Moisture Initial Loss

The U.S. soil conservation service has recommended an initial abstraction of Ia = 0.2S to be made to cover initial interception, infiltration, surface storage losses for this group. 'S' is the difference between storm rainfall and direct runoff. Utilizing the SCS rainfall runoff curves for land use classification and soil groups similar to those in Bangladesh 'S' has been found to equal 2.5 and Ia = 12.7 mm (0.5").

i) For paddy land no initial loss will be used.

ii) For non-paddy land an initial soil moisture loss of 12.7 mm (0.5") will be considered.

4.2 Soil Moisture Subsequent Loss

During the storm period infiltration continues to take place at a slower rate than the initial soil moisture repletion. Infiltration studies have shown that all but impervious clay soils have a maximum constant infiltration rate after saturation that may range from 1.27 mm (0.05")/hour to greater than 25.4 mm (1")/hr.

We will assume an average infiltration rate of 25.4 mm (1")/day or 1.058 mm (0.04")/hour or 6.35 mm (0.25")/6 hours for relatively impervious soils .

4.3 Depression Storage

4.3.1 Paddy Land

Paddy land is bounded by small earthen bund to retain water. This may be as high as 152.4 mm (6)" or more. We will assume that the first 101.6 mm (4") of rain falling upon paddy land serves to fill up the paddy basin. Rainfall in excess of 101.6 mm becomes runoff.

4.3.2 Non-paddy Land

We will assume that the constant rate of retention is approximately 0.8382 mm (0.033")/hour or 5.08 mm (0.2") per 6 hour during periods of rainfall until a maximum of 25.4 mm (1") is stored.

Whenever possible the percentages of paddy and non-paddy land should be determined by field survey. This is particularly important for small basins where the proportion of paddy land may be very high.

In the absence of a survey Table 9 may be used for determining the agricultural land classification.

Table 9. Agricultural Land Classification

District	Percentage of Total Land Area	
	Available for cultivation	Cultivated as paddy land
Dhaka	70	33
Mymensingh	75	40
Faridpur	84	50
Bakerganj	69	57
Chittagong	42	39
Noakhali	66	41
Comilla	79	61
Sylhet	56	35
Rajshahi	74	45
Dinajpur	69	46
Rangpur	66	42
Bogra	84	56
Pabna	81	37
Kushtia	75	26
Jessore	73	35
Khulna	36	25
Average	67	42

5.0 RAINFALL EXCESS - WEIGHTED BASIN AVERAGE

5.1 Paddy Land

Separate the rainfall losses from the rainfall time distribution determined in Table 8 by assuming that the entire basin area consists of paddy land. Computations are shown in Table 10.

5.2 Non-paddy Land

Separate the rainfall losses from the rainfall time distribution determined in Table 8 by assuming that the entire basin area consists of non-paddy land. Computations are shown in Table 11.

5.3 Weighted Basin Average

Compute the weighted basin runoff distribution by multiplying the net runoff determined in Table 10 and Table 11 by the respective land classification percentage. Computations are shown in Table 12.

Table 10. Rainfall Excess - Paddy Land

Days	Hours	Rainfall mm	Losses in mm			Available Paddy Storage in mm	Net Run off in mm
			Soil Moisture		Depression Storage		
			Initial	Subsequent			
1	0-6	16.76	0.00	-6.35	-101.60	-91.19	0.00
	6-12	17.53		-6.35		-80.01	0.00
	12-18	0.00		-6.35		-86.36	0.00
	18-24	6.73		-6.35		-85.98	0.00
2	0-6	16.76		-6.35		-75.57	0.00
	6-12	17.53		-6.35		-64.39	0.00
	12-18	20.07		-6.35		-50.67	0.00
	18-24	14.73		-6.35		-42.29	0.00
3	0-6	16.76		-6.35		-31.88	0.00
	6-12	17.53		-6.35		-20.70	0.00
	12-18	89.16		-6.35		0.00	62.11
	18-24	14.73		-6.35		0.00	8.38
4	0-6	16.76		-6.35		0.00	10.41
	6-12	0.00		-6.35		-6.35	0.00
	12-18	0.00		-6.35		-12.70	0.00
	18-24	12.38		-6.35		-6.67	0.00
5	0-6	16.76		-6.35		0.00	3.74
	6-12	0.00		-6.35		-6.35	0.00
	12-18	0.00		-6.35		-12.70	0.00
	18-24	3.75		-6.35		-15.30	0.00

Table 11. Rainfall Excess - Non Paddy Land

Days	Hours	Rainfall mm	Losses in mm			Available Non-Paddy Storage in mm	Net Run off in mm
			Soil Moisture		Depression Storage		
			Initial	Subsequent			
1	0-6	16.76	-12.70	0.00	-4.06	0.00	0.00
	6-12	17.53		-6.35	-5.08	0.00	6.10
	12-18	0.00		-6.35	0.00	-6.35	0.00
	18-24	6.73		-6.35	0.00	-5.97	0.00
2	0-6	16.76		-6.35	-4.44	0.00	0.00
	6-12	17.53		-6.35	-5.08	0.00	6.10
	12-18	20.07		-6.35	-5.08	0.00	8.64
	18-24	14.73		-6.35	-1.66	0.00	6.72
3	0-6	16.76		-6.35	0.00	0.00	10.41
	6-12	17.53		-6.35	0.00	0.00	11.18
	12-18	89.16		-6.35	0.00	0.00	82.81
	18-24	14.73		-6.35	0.00	0.00	8.38
4	0-6	16.76		-6.35	0.00	0.00	10.41
	6-12	0.00		-6.35	0.00	-6.35	0.00
	12-18	0.00		-6.35	0.00	-12.70	0.00
	18-24	12.38		-6.35	0.00	-6.67	0.00
5	0-6	16.76		-6.35	0.00	0.00	3.74
	6-12	0.00		-6.35	0.00	-6.35	0.00
	12-18	0.00		-6.35	0.00	-12.70	0.00
	18-24	3.75		-6.35	0.00	-15.30	0.00

Table 12. Rainfall Excess - Weighted Basin Average

Days	Hours	Paddy land (50%)		Non paddy land (50%)		Basin weighted runoff (mm)
		Net runoff (mm)	Weighted runoff (mm)	Net runoff (mm)	Weighted runoff (mm)	
1	0-6	0.00	0.00	0.00	0.00	0.00
	6-12	0.00	0.00	6.10	3.05	3.05
	12-18	0.00	0.00	0.00	0.00	0.00
	18-24	0.00	0.00	0.00	0.00	0.00
2	0-6	0.00	0.00	0.00	0.00	0.00
	6-12	0.00	0.00	6.10	3.05	3.05
	12-18	0.00	0.00	8.64	4.32	4.32
	18-24	0.00	0.00	6.72	3.36	3.36
3	0-6	0.00	0.00	10.41	5.21	5.21
	6-12	0.00	0.00	11.18	5.59	5.59
	12-18	62.11	31.05	82.81	41.41	72.46
	18-24	8.38	4.19	8.38	4.19	8.38
4	0-6	10.41	5.21	10.41	5.21	10.41
	6-12	0.00	0.00	0.00	0.00	0.00
	12-18	0.00	0.00	0.00	0.00	0.00
	18-24	0.00	0.00	0.00	0.00	0.00
5	0-6	3.74	1.87	3.74	1.87	3.74
	6-12	0.00	0.00	0.00	0.00	0.00
	12-18	0.00	0.00	0.00	0.00	0.00
	18-24	0.00	0.00	0.00	0.00	0.00
				Total Runoff	=	119.57 mm

6.0 *RUNOFF HYDROGRAPH*

The runoff hydrograph represents *the time-discharge relationship of* the total design storm runoff at the basin outlet. The following three methods have been presented in *this section for the development of runoff* hydrograph.

1. Unit Hydrograph Method
2. Richard's Runoff Hydrograph
3. Soil Conservation Service (SCS) Method

6.1 Unit Hydrograph

A unit hydrograph is a discharge hydrograph resulting from one unit (1 cm or 1 mm or 1 inch) of rainfall excess generated uniformly over the basin at a uniform rate during a specified period of time.

The runoff hydrograph is developed on the basis of the following unit hydrograph principles.

1. Hydrographs of the same basin with the same unit rainfall duration, Tr, have vertical ordinates, qx, directly proportional to the depth of rainfall excess.
2. Hydrographs of the same basin with the same unit rainfall duration have the same time base, Tb, regardless of intensity of rainfall.
3. Several of the above hydrographs can be combined to form one composite hydrograph by the simple addition of their vertical ordinates.

6.1.1 The Unit Hydrograph Shape

Most of the basins we will encounter will be ungauged with scanty hydrological data available. Therefore, rather than using the more complicated and classical curvilinear hydrograph shape we will use the simpler triangular approximation. The shape of the hydrograph and its nomenclature is as follows:

q_p = Peak rate of runoff in m^3/sec

T_p = Time of rise from beginning of rainfall excess to peak rate in hours

T_r = Time of recession from peak rate to end of hydrograph in hours

T_b = Time base of hydrograph in hours

t_r = Duration of rainfall excess in hours

t_p = Lag time in hours from centre of rainfall excess to peak

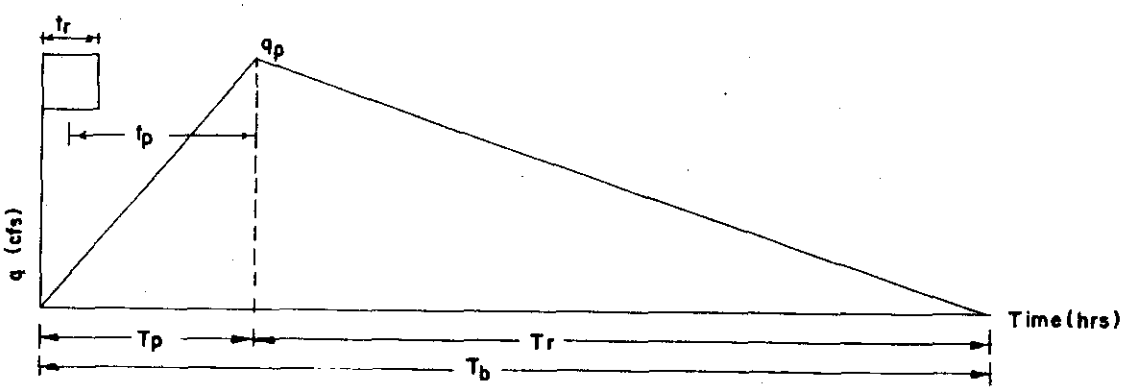


Fig. 12. Simplified Hydrograph Shape.

6.1.2 Developing the Unit Hydrograph

The following data describing the basin characteristics have been obtained from topographic maps.

1) Basin area, A = 4333.15 ha
2) Length of the principle drainage channel: L = 14.49 km
3) Distance to basin centroid: $\bar{L}$ = 5.8 km
4) Average overland slope of basin measured between contours = 0.2367 m/km
5) Straight line length of basin = 8.37km
6) Average channel slope : $S = \dfrac{0.2367 \times 8.37}{14.49} = 0.1367$ m/km
7) Channel roughness factor: n = 0.055

6.1.3 The Elements of the unit Hydrograph

1. Time of concentration $T_c = 14.15 \left[\dfrac{L^2 \; n^2}{S}\right]^{0.3}$

$$= 14.15 \left[\frac{14.49^2 \times 0.055^2}{0.1367}\right]^{0.3} = 22.43 \text{ hrs}$$

2. Lag time to peak : $t_p = 0.6\, T_c = 0.6 \times 22.43 = 13.5$ hrs

3. Time of rise : $T_p = t_p + \frac{1}{2}\, t_r = 13.5 + \frac{1}{2} \times 6 = 17$ hrs

4. Coefficient C_t :

$$C_t = \frac{1.331 \times t_p}{(L X \bar{L})^{0.3}} = \frac{1.331 \times 13.5}{(14.49 \times 5.8)^{0.3}} = 4.76$$

5. Coefficient Cp : 0.43, obtained from **Figure** 13

6. Unit-graph Base Length:

$$T_b = \frac{1.2\ T_c}{C_p} = \frac{1.2 \times 22.43}{0.43} = 63 \text{ hours}$$

7. Peak discharge:

$$q_p = \frac{0.14A}{T_b} = \frac{0.14 \times 4333.15}{63}$$

$$= 9.63 \text{ m}^3\text{/sec/25.4 mm} = 0.3791 \text{ m}^3\text{/sec/mm}$$

The completed hydrograph is shown in **Figure** 14

7.0 THE RUNOFF HYDROGRAPH: (UNIT HYDROGRAPH METHOD)

The runoff hydrograph represents the time discharge relationship of the total design storm runoff at the basin outlet. The computations are shown in Table 13. **Figure** 15 represents the shape of the runoff hydrograph.

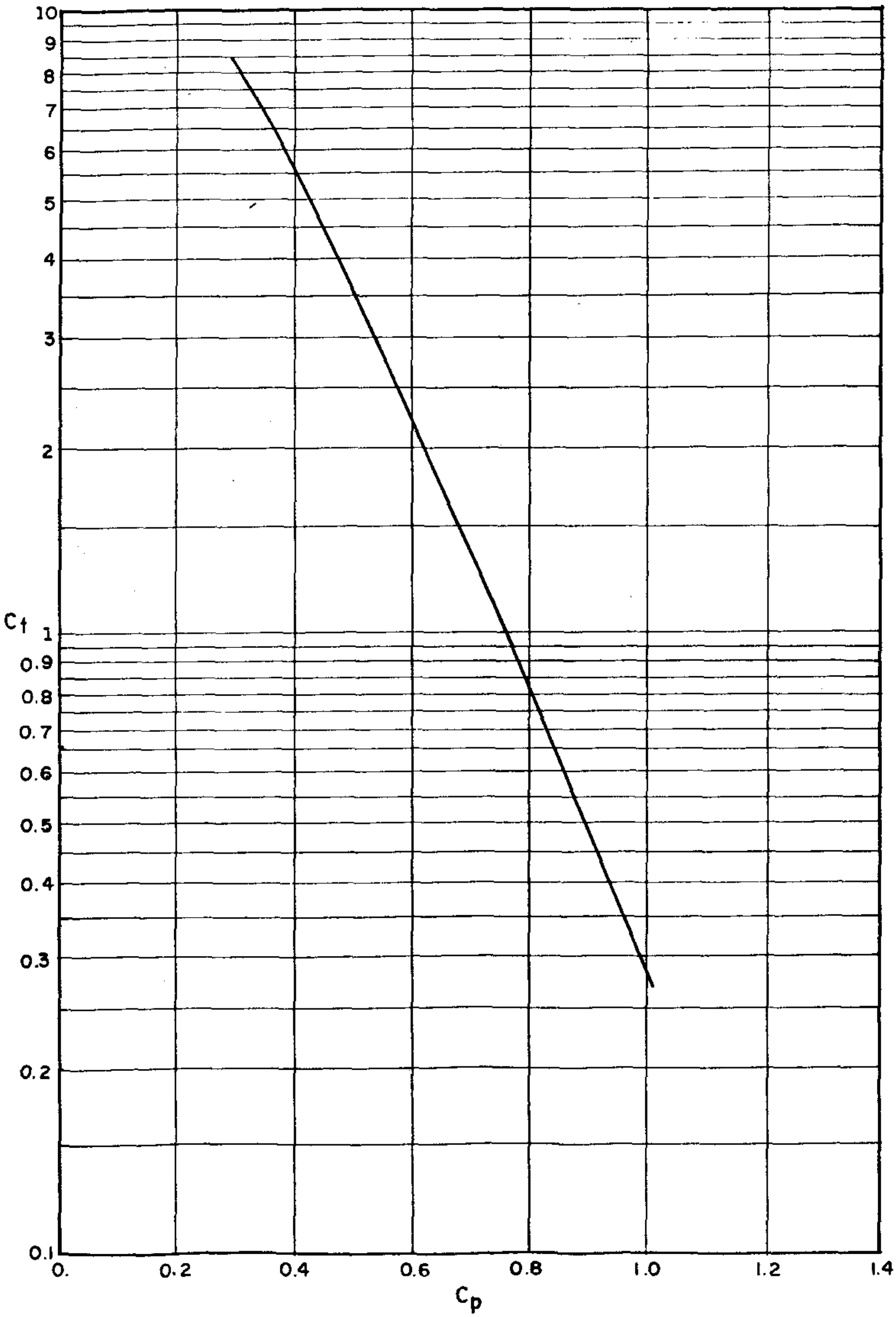


Figure 13. Snyder Co-efficients

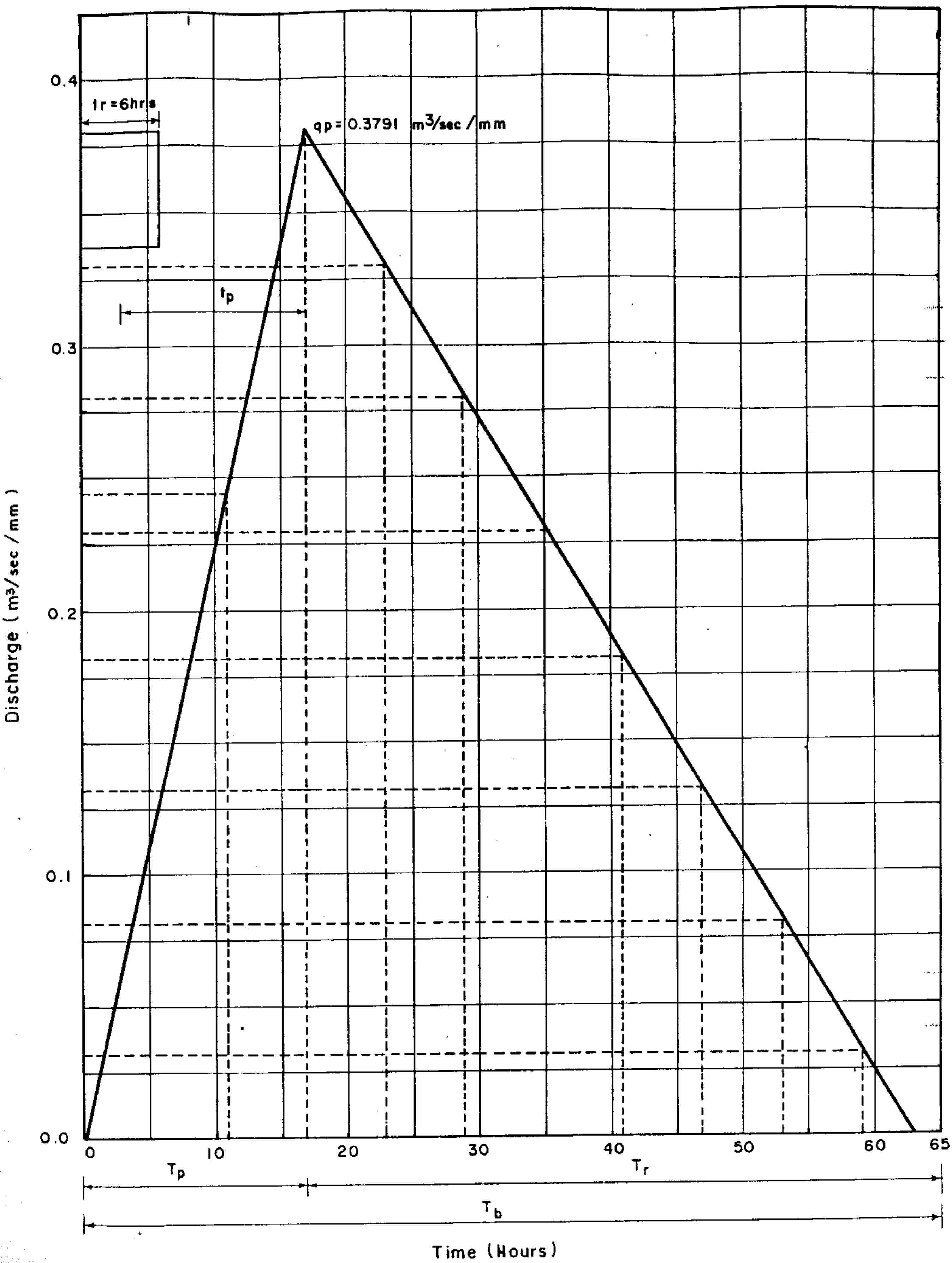


Figure 14. Unit Hydrograph

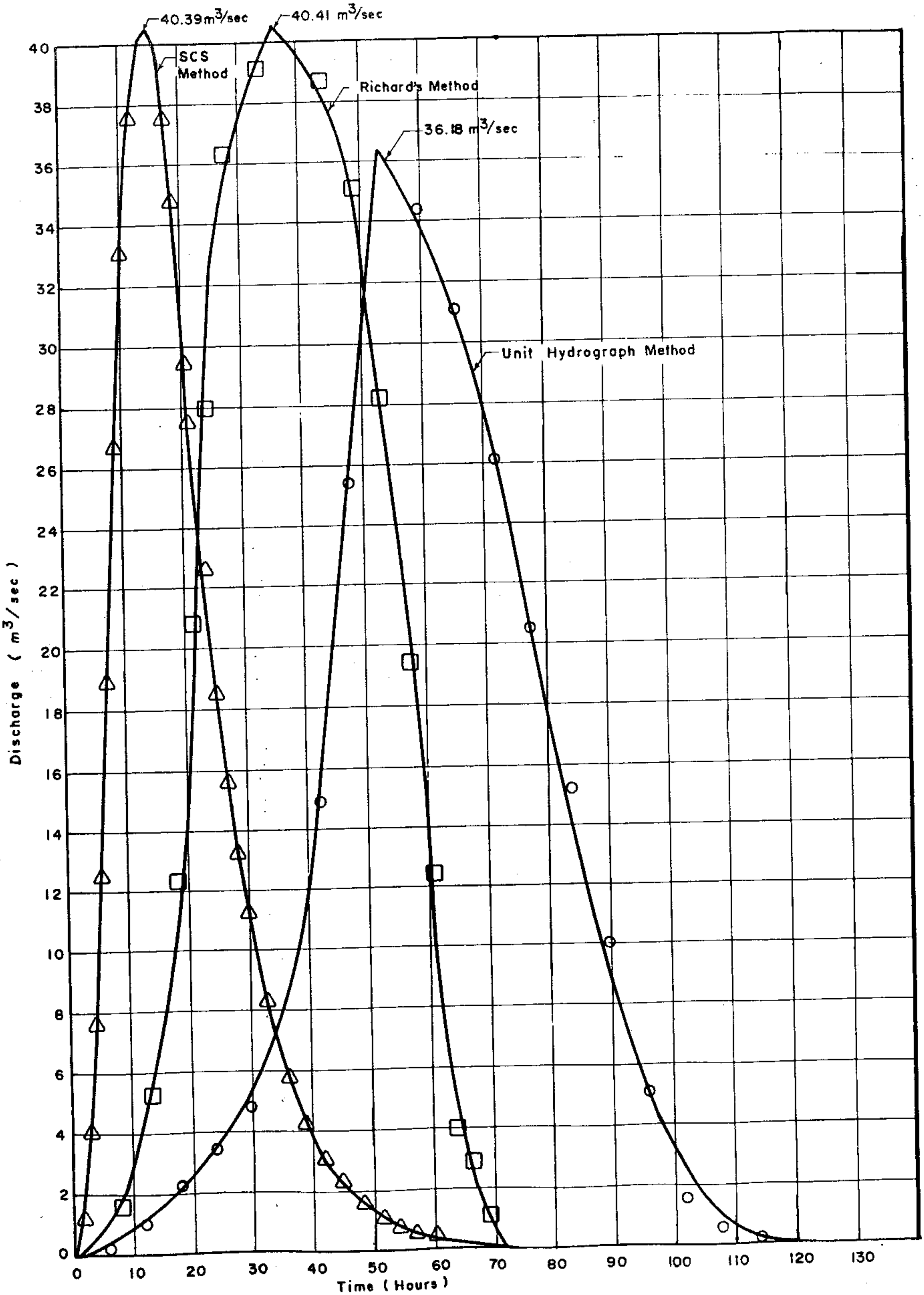


Figure 15. Runoff Hydrograph

tT **Table 13. Runoff Hydrograph Computations (Unit Hydrograph Method)**

Unit Hydrograph Ordinates m^3/Sec/mm	Hours	Days	Rainfall Excess-6 hours duration (in mm)										Runoff hydrograph m^3/Sec
			3.05	3.05	4.32	3.36	5.21	5.59	72.46	8.38	10.41	3.74	
0	6		0										0
0.11	12	1	0.34	0									0.34
0.245	18		0.75	0.34	0								1.09
0.3791	24		1.16	0.75	0.48	0							2.39
0.33	6		1.01	1.16	1.06	0.37	0						3.60
0.28	12	2	0.85	1.01	1.64	0.82	0.57	0					4.89
0.23	18		0.70	0.85	1.43	1.27	1.28	0.62	0				6.15
0.1825	24		0.56	0.70	1.21	1.11	1.98	1.37	7.97	0			14.90
0.1325	6		0.40	0.56	0.99	0.94	1.72	2.12	17.75	0.92	0		25.40
0.0825	12		0.25	0.40	0.79	0.77	1.46	1.84	27.47	2.05	1.15	0	36.18
0.0325	18	3	0.10	0.25	0.57	0.61	1.20	1.57	23.91	3.18	2.55	0.41	34.35
0	24		0	0.10	0.36	0.45	0.95	1.29	20.29	2.77	3.95	0.92	31.08
	6			0	0.14	0.28	0.69	1.02	16.67	2.35	3.44	1.42	26.01
	12	4			0	0.11	0.43	0.74	13.22	1.93	2.91	1.23	20.57
	18					0	0.17	0.46	9.60	1.53	2.39	1.05	15.20
	24						0	0.18	5.98	1.11	1.90	0.86	10.03
	6							0	2.35	0.69	1.38	0.68	5.10
	12	5							0	0.27	0.86	0.50	1.63
	18									0	0.34	0.31	0.65
	24										0	0.12	0.12
	6	6										0	0
													239.68

<u>A check on the computation:</u>

1. Total volume of water under the hydrograph : V = 0.36 x It,

I = from last col., t = 6 hour time interval

$$V = 0.36 \times 239.68 \times 6 = 517.71 \text{ ha.m}$$

2. Total rainfall excess, $i = 1000 \dfrac{V}{A} = \dfrac{1000 \times 517.71}{4333.15} = 119.48$ mm

$$\approx 119.57 \text{ mm}$$

Which is equal to the summation of weighted rainfall excess in Table 12.

8.0 THE RUNOFF HYDROGRAPH (RICHARD'S METHOD)

8.1 Richard's Equations

(In the computation of peak Q by Richard's and SCS methods, some graphs, charts and formulas are used, which could not be converted from FPS system to MKS system. Hence computations of peak Q by these two methods are done by FPS system and rest of the computations are made in MKS system).

The assumptions made by Richard are that the storm covered the catchment and that its duration was equal to the period of concentration of the flood. These assumptions implied that the whole catchment contributed to produce the maximum intensity of flood.

On this basis, the following principal equations were deduced by Richard:

i) $$i = \frac{R.f(a)}{t + 1} \text{ or } R = \frac{i\ (t+1)}{f(a)}$$

ii) $$\frac{t^3}{t + 1} = \frac{N.C.L^2}{K.S.R.f(a)}$$

iii) $$Q = k\ i\ a$$

Where, t denotes the period of concentration in hrs.

Q " maximum intensity of flood in cusecs

N " storm shape factor with range of value from 0.72 to 1.72 (usually 1.1)

i " the average intensity of rainfall over the catchment in inches per hour

a " the area of the catchment in acres

L " the distance in miles from the point of concentration to the furthest point on the catchment

S " the slope of the catchment

k " the runoff coefficient

f(a) " a function of the area for the adjustment of the average intensity of the rainfall.

C " a co-efficient

R " a co-efficient of rainfall

The intermediate points on the curve of rising flood are given by:

$$Q_i = Qm. \frac{a_j}{a} \quad \text{and } t_i = t\left[\left[\frac{r_j}{L}\right]^2\right]^{1/3}$$

$j = 1,2,3 \ldots\ldots n$

Where, r is the radius from the point of concentration along L for the area, a, bounded by the catchment boundary.

The falling flood curve is the reverse of the rising flood curve.

8.2 Design Computations & Procedure

a) Use 24-hour point rainfall with required frequency and determine i.

b) Use topo-map and determine a, L & S

c) Determine f(a) from **Figure** 16 for the area of the catchment.

d) Compute R using Equation (i)

e) Select K for the catchment using Table 14 and compute K.R.

f) Determine C from **Figure** 17 using the computed value of K.R.

g) Find t by using Equation (ii)

h) Use t in nearest days and take point rainfall for this duration.

i) Repeat necessary steps to obtain t at the end of the operation as same in the beginning of the operation

j) Compute i for this determined t

k) Use equation (iii) and compute Q.

l) Draw arcs with 'O' as centre from the point of concentration along L with equal intervals say 1,2,3,4,5.....miles

m) Compute area of the catchment intercepted by each arc

n) Compute intermediate points on the hydrograph by using equations for intermediate points.

Table 14. Values of Runoff Co-efficient, K

Type of catchment	Large	Small & Steep
Rocky and impermeable	0.8	1.0
Slightly permeable, bare	0.6	0.8
Slightly permeable, partly cultivated or covered with vegetation	0.4	0.6
Cultivated absorbent soil	0.3	0.4
Sandy absorbent soil	0.2	0.3
Heavy forest	0.1	0.2

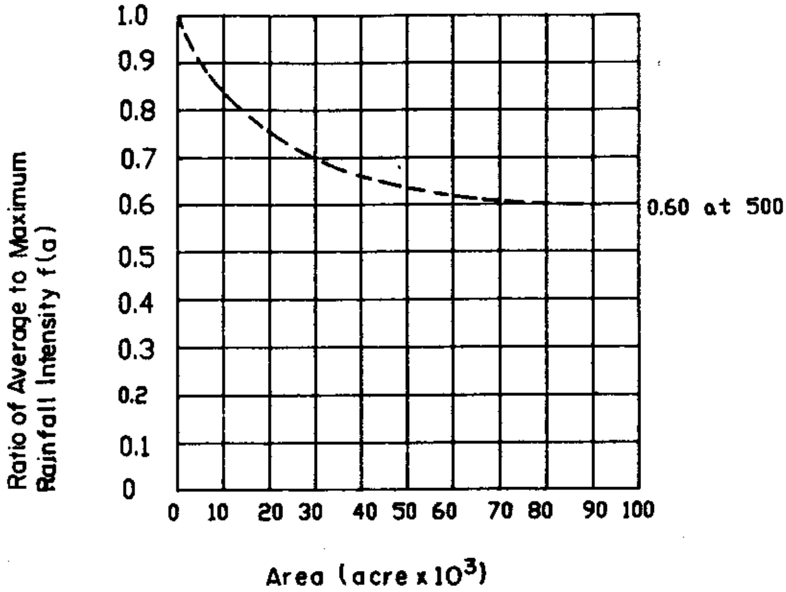


Figure 16. Ratio of Rainfall Intensity f(a)

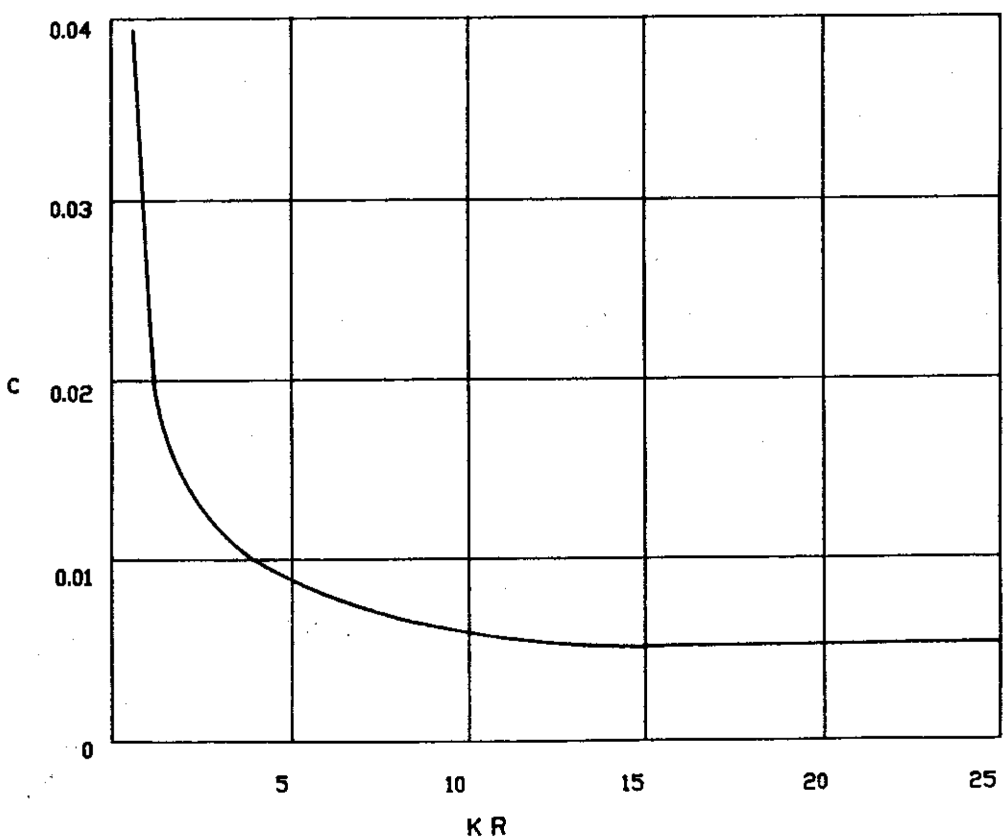


Figure 17. Co-efficient, C

8.3 Example

$$a = 4333.15 \text{ ha} = 10707.2 \text{ acres}$$

$$L = 14.49 \text{ km} = 9 \text{ miles}$$

$$S = 0.1367 \text{ m/km} = 0.0001367$$

24 hour rainfall = 162.56 mm = 6.4 inch (from Table 5)

$$i = \frac{6.4}{24} = 0.267 \text{ in/hour}$$

f(a) = 0.825 from **Figure 16** for a = 10707.2 acres

$$R = \frac{i\,(t + 1)}{f(a)} = \frac{0.267 \times 25}{0.825} = 8.09 \text{ inches}$$

$$K = 0.6 \text{ selected from Table 14}$$

$$KR = 0.6 \times 8.09 = 4.854$$

C = 0.009 from **Figure** 17 for KR = 4.854

$$\frac{t^3}{t+1} = \frac{N.C.L^2}{K.S.R.f(a)} = \frac{1.1 \times 0.009 \times 9^2}{0.6 \times 0.0001367 \times 8.09 \times 0.825} = 1465$$

By trial, t = 39 hours ≈ 2 days

2 - day rainfall = 243.84 mm = 9.6"

$$i = \frac{9.6}{48} = 0.2 \text{ inch/hour}$$

$$R = \frac{0.2 \times 49}{0.825} = 11.88$$

$KR = 0.6 \times 11.88 = 7.128$

$C = 0.0075$

$$\frac{t^3}{t+1} = \frac{1.1 \times 0.0075 \times 9^2}{0.6 \times 0.0001367 \times 11.88 \times 0.825} = 831.28$$

$t = 30$ hours $\approx 1\frac{1}{2}$ day

$1\frac{1}{2}$ day rainfall = 203 mm = 8" [From **Figure** 6, 7, & 8]

$$i = \frac{8}{36} = 0.222 \text{ inch/hour}$$

$$R = \frac{0.222 \times 37}{0.825} = 9.956$$

$KR = 0.6 \times 9.956 = 5.97$

$C = 0.0085$

$$\frac{t^3}{t+1} = \frac{1.1 \times 0.0085 \times 9^2}{0.6 \times 0.0001367 \times 9.956 \times 0.825} = 1124$$

$t = 34$ hours ≈ 36 hours

$i = 0.222$ inch/hour

$Q = Kia = 0.6 \times 0.222 \times 10707.2 = 1426$ cfs $= 40.41$ m^3/Sec

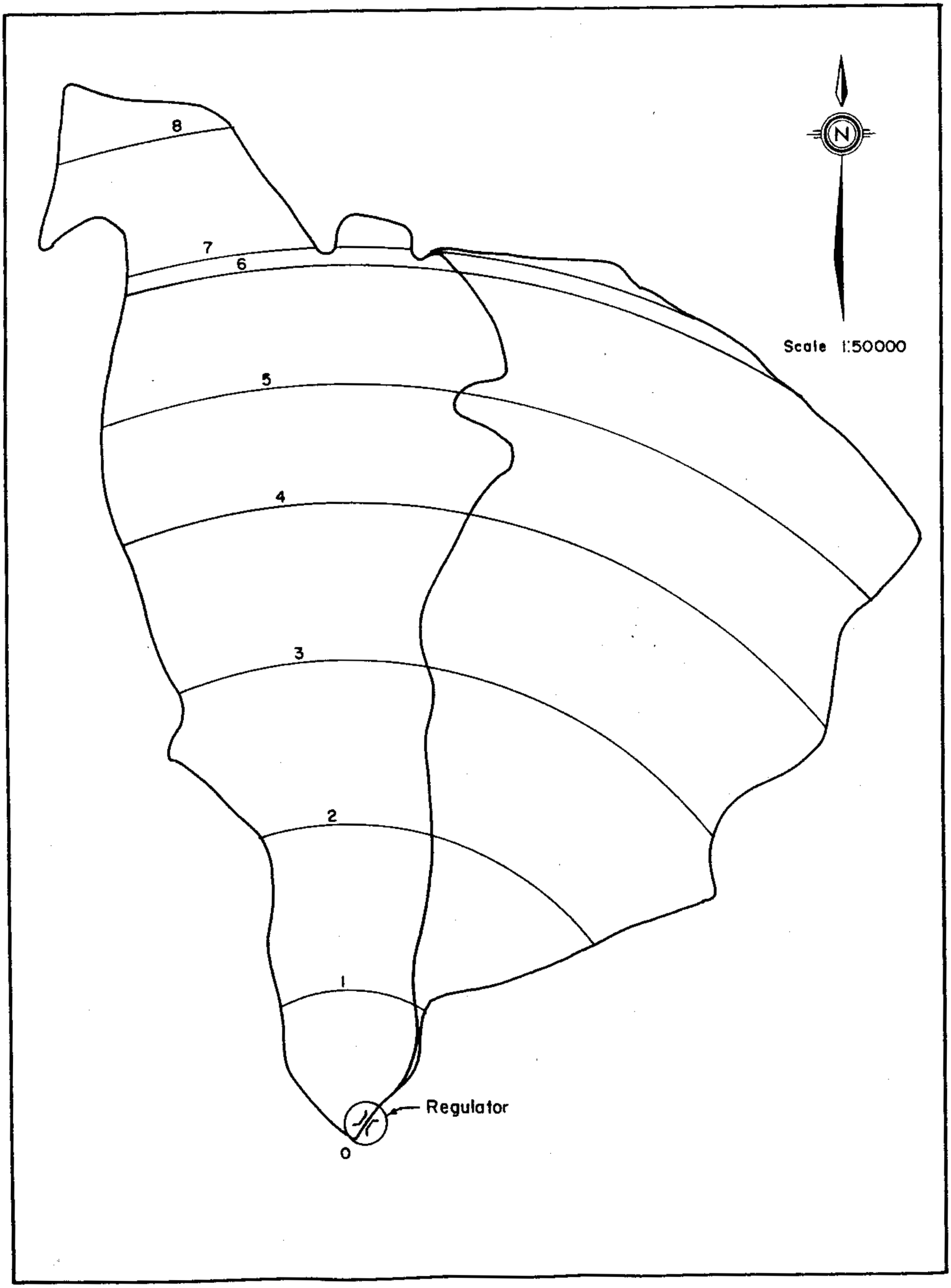


Figure 18. Richard's Basin Characteristics

To compute intermediate points arcs are drawn with 0 as centre from the point of concentration along L with radii 1, 2, 3, 4, 5, 6, 7 & 8 (mile) (**Figure** 18) and the area of the catchment intercepted by each arc is computed. The intermediate points on the curve of raising flood are given by :

$$Q_j = Q_m . \frac{a_j}{a} \quad \text{and } t_j = t \left[\left[\frac{r_j}{L} \right]^2 \right]^{1/3} \quad j = 1, 2, 3 \ldots\ldots n$$

The falling flood curve is the reverse of the rising flood curve. Computations are shown in Table 15 and the runoff hydrograph is shown in **Figure** 15.

Table 15. Runoff Hydrograph Computations (Richards Method)

Dist. from outlet (miles)	$\frac{a_i}{a}$	$\left[\left[\frac{r_i}{L}\right]^2\right]^{1/3}$	Rising Flood		Falling Flood	
			Q_R (in m^3/Sec)	t_R (in hour)	Q_F (in m^3/Sec)	t_F (in hour)
(1)	(2)	(3)	(4) (40.41xcol.2)	(5) (36xcol.3)	(6) (40.41-col.4)	(7) (36+col.5)
0	0	0	0	0	40.41	36.00
1	0.043	0.2311	1.74	8.32	38.67	44.32
2	0.132	0.3669	5.33	13.21	35.08	49.21
3	0.302	0.4807	12.20	17.31	28.21	53.31
4	0.517	0.5824	20.89	20.97	19.52	56.97
5	0.693	0.6758	28.00	24.33	12.41	60.33
6	0.899	0.7631	36.33	27.47	4.08	63.47
7	0.925	0.8457	37.38	30.45	3.03	66.45
8	0.973	0.9245	39.32	33.28	1.09	69.28
9	1.000	1.0000	40.41	36.00	0	72.00

9.0 THE RUNOFF HYDROGRAPH (SCS METHOD)

9.1 Design Computation and Procedure

a) Estimate curve number (CN) according to hydrologic soil group & soil-cover complexes using Table 17 and Table 18.

b) Estimate $Tc = 14.15 \left[\dfrac{L^2 n^2}{S} \right]^{0.3}$ (in hours)

Where, L = Length of the principle drainage channel in km

n = Channel Roughness Factor

S = Average channel slope in m/km

c) Find duration of rainfall, D = 0.133 Tc (in hours)

d) Find time to rise, $Tp = \dfrac{D}{2} + 0.6\ Tc$ (in hours)

e) Find direct runoff, Q (excess rainfall) using the following equation;

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \quad \text{(in inches)}$$

Where, P = Potential maximum rainfall

$$S = \frac{1000}{CN} - 10$$

g) Find peak discharge

$$q_p = \frac{484\ AQ}{T_p} \quad \text{in cfs}$$

Where, A = Basin area in sq. mile

Take T_p & Q from steps (d) & (e)

h) Estimate total actual hydrograph from SCS dimensionless unit hydrograph (q/q_p Vs t/T_p), values are presented in Table 19.

Where, q = Discharge at time t

q_p = peak discharge

t = A selected time

T_p = Time from beginning of rise to the peak.

9.2 Example (SCS Method)

Location of Basin = Faridpur Dist.

Type of soil = clay

Land use = Cultivated Land

From Table 17, Hydrologic soil group = D

From Table 18, Curve number = 91

$$Tc = 14.15 \left[\frac{L^2 n^2}{S} \right]^{0.3} \quad \text{(in hours)}$$

= 22.43 hrs. [From section 6.1.3]

$$D = 0.133\ Tc = 0.133 \times 22.43 = 2.98 \text{ hrs}$$

Say 3 hrs

$$Tp = \frac{D}{2} + 0.6\ T_c = \frac{3}{2} + 0.6 \times 22.43 = 15 \text{ hrs}$$

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

From Table 6, P for 3 hrs = 84.07 for Jessore Dist.

For Faridpur Dist.

$$P = 84.07 \times \frac{1270}{1168.4} = 91.38 \text{ mm} = 3.6"$$

$$S = \frac{1000}{CN} - 10$$

$$= \frac{1000}{91} - 10 = 0.99$$

$$Q = \frac{(3.6 - 0.2 \times 0.99)^2}{3.6 + 0.8 \times 0.99} = 2.64"$$

$$q_p = \frac{484 \ AQ}{Tp}$$

A = 4333.15 ha = 16.73 sq. mile

$$q_p = \frac{484 \times 16.73 \times 2.64}{15}$$

= 1425 cfs

= 40.39 m^3/Sec

Table 16. Runoff Hydrograph Computation (SCS Method)

Time Ratio t/T_p	Time (in hours)	Discharge Ratio q/q_p	Discharge q (in m^3/Sec)
0	0	0.000	0.00
0.1	1.5	0.030	1.21
0.2	3.0	0.100	4.04
0.3	4.5	0.190	7.67
0.4	6.0	0.310	12.52
0.5	7.5	0.470	18.98
0.6	9.0	0.660	26.66
0.7	10.5	0.820	33.12
0.8	12.0	0.930	37.56
0.9	13.5	0.990	39.99
1.0	15.0	1.000	40.39
1.1	16.5	0.990	39.99
1.2	18.0	0.930	37.56
1.3	19.5	0.860	34.74
1.4	21.0	0.780	31.50
1.5	22.5	0.680	27.47
1.6	24.0	0.560	22.62
1.7	25.5	0.460	18.58
1.8	27.0	0.390	15.75
1.9	28.5	0.330	13.33
2.0	30.0	0.280	11.31
2.2	33.0	0.207	8.36
2.4	36.0	0.147	5.94
2.6	39.0	0.107	4.32
2.8	42.0	0.077	3.11
3.0	45.0	0.055	2.22
3.2	48.0	0.040	1.62
3.4	51.0	0.029	1.17
3.6	54.0	0.021	0.85
3.8	57.0	0.015	0.61
4.0	60.0	0.011	0.44
4.5	67.5	0.005	0.20
5.0	75.0	0.000	0.00

The runoff hydrograph is plotted with the values of t & q from Table 16 and presented in **Figure** 15.

Table 17. Hydrologic Soil Groups

A. (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

D. (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Table 18. Runoff Curve Numbers For Selected Agricultural, Suburban, And Urban Land Use (Antecedent Moisture Condition II, And I = 0.2S)

(From SCS Technical Release No. 55, P.2-5)

Land Use Description		Hydrological Soil Group A	B	C	D
Cultivable land/	: Without conservation treatment	72	81	88	91
	with conservation treatment	62	71	78	81
Pasture or range land:	poor condition	68	79	86	89
	Good condition	39	61	74	80
Meadow	: Good condition	30	58	71	78
Wood of forest land	: thin scant, poor cover, no mulch	45	66	77	83
	good cover 2/	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc					
good condition :	grass cover on 75% or more of the area	39	61	74	80
fair condition :	grass cover on 50% or 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential 3/					
<u>Average lot size</u>	<u>Average % impervious</u> 4/				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveway, etc. 5/		98	98	98	98
Streets and roads:	paved with curbs and storm sewers 5/	98	98	98	98
	gravel	76	85	89	91
	dirt	72	82	87	89

1/ For a more detailed description of agriculture land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.
2/ Good cover is protected from grazing and litter and brush cover soil.
3/ Curve numbers are computed assuming that the runoff from the house & driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.
4/ The remaining pervious areas (Lawns) are considered to be in good pasture condition for these curve numbers.
5/ In some warmer climate of the country a curve number of 95 may be used.

Table 19. Ratios For Dimensionless Unit Hydrograph

Time Ratios (t/Tp)	Discharge Ratios (q/qp)
0.0	.000
0.1	.030
0.2	.100
0.3	.190
0.4	.310
0.5	.470
0.6	.660
0.7	.820
0.8	.930
0.9	.990
1.0	1.000
1.1	.990
1.2	.930
1.3	.860
1.4	.780
1.5	.680
1.6	.560
1.7	.460
1.8	.390
1.9	.330
2.0	.280
2.2	.207
2.4	.147
2.6	.107
2.8	.077
3.0	.055
3.2	.040
3.4	.029
3.6	.021
3.8	.015
4.0	.011
4.5	.005
5.0	.000

10.0 FLOOD ROUTING

The flood wave moving through a short reach of channel of regular section will undergo little change in its configuration. However, if the flow is obstructed in any way the flood wave configuration may be modified appreciably. The determination of this modification is called flood routing. Flood routing in our case will consist of predicting an outflow hydrograph below a drainage structure from a known inflow hydrograph above the structure.

Flood routing serves several purposes.

a) The maximum discharge under the outflow hydrograph is used as a basis for designing the size of the structure.

b) Because the inflow discharge during the flood peak is greater than the outflow discharge, a portion of runoff becomes storage. Flood routing will determine the accumulated storage volume. The storage water surface elevation can be found out by using the basin area- capacity curve. This information is used to compute crop damages due to flooding.

c) The structure is designed for percolation and uplift using the head differential across the structure measured from the storage level to the tail water level as determined by the outflow hydrograph.

Generally a routing will be made for a pre-monsoon storm, when the river stage is low, as criteria for designing the structural elements.
Before the flood routing process can begin, the designer must estimate a preliminary size of structure required. In a non-tidal area roughly for each 1200 ha to 1500 ha of the catchment area a single vent of size 1.52 m x 1.83 m may be assumed. The routing procedure will determine whether the assumed structure is adequate after computing the extent of flood damage. Additional routing may be required through an enlarged or reduced structure depending upon the results of flood damage computed from the

first routing. The range of tolerable submergence of area may be as high as 10% or even more of the gross project area depending on the topography of the basin. The designer will apply his judgement after studying the basin topography and importance of land, that how great an area can be flooded and to what depth. In general the tolerable range of submergence of area for not more than 30 cm for a period of 72 hours may be considered upto an incremental area of 5% of the total project area in addition to the existing low lands that can not be drained by gravity.

A postmonsoon routing, when the discharge is controlled by the tail water level due to high river stage, will also be required to check the size of the structure selected by pre-monsoon routing. The criteria for checking the adequacy of the size is to observe that the difference of water level between R/s and C/s cannot be visually identified. This difference of water level is considered to be 15 cm to 23 cm which is not easily detectable in the field to a maximum of 30 cm for three consecutive days. The farmer will then be satisfied with the design provision and will not insist for extra ventage. The tendency for cutting off the embankment for quicker drainage of the area will also be stopped.

The flood routing procedure have been illustrated with the following example on Mrigi Khal regulator under Mrigi drainage and water conservation scheme, Faridpur. Computations of pre-monsoon flood routing are done with basin runoff hydrographs generated by Unit Hydrograph Method, Richard's Method and SCS Method. The computed results are discussed in Art 13.0

11.0 PRE-MONSOON FLOOD ROUTING

Flood routing is basically a system of book keeping involving inflow, outflow and storage. During a definite time interval (assuming there are no losses other than those already extracted.)

$$I_v - D_v = S$$

Where,

I_v = Total volume of inflow during the period.

D_v = Total volume of outflow " "

S = Total volume going to storage " "

The eqn. written for a definite time period and expressing inflow and outflow in m^3/sec is

$$\frac{S}{t} = I - D$$

Where,

t = time of period in sec

I = Average inflow during the period in m^3/sec

D = Average outflow " " "

S = Total volume going to storage during the period measured in m^3

This equation forms the basis for a hydrological procedure in routing in which 't' is known the routing period.

11.1 Flood Routing Equation

$$(I - D)\ t = S$$

Let $S_2 - S_1 = S$, Storage differential during period

$$\frac{I_1 + I_2}{2} = \text{I average inflow during the period}$$

$$\frac{D_1 + D_2}{2} = \text{D average outflow during the period}$$

Substituting the terms

$$\left(\frac{I_1 + I_2}{2} - \frac{D_1 + D_2}{2}\right) t = S_2 - S_1$$

$$I_1 + I_2 - D_1 - D_2 = \left(\frac{2S_2}{t}\right) - \left(\frac{2S_1}{t}\right)$$

$$(I_1 + I_2) + \left(\frac{2S_1}{t} - D_1\right) = \left(\frac{2S_2}{t} + D_2\right)$$

To solve the equation 4 curves are required

1. The inflow hydrograph - already prepared
2. Area - capacity curve
3. The stage discharge relation for condition 3 & 5.
4. The 2S/t + D curve.

11.2 Area-Capacity Curves

To draw these curves a contour map of the project area is collected and the area between each contour level is planimetered. The area curve is plotted as ground elevation Vs total accumulated area at any elevation. The capacity curve is the accumulation of volume below a particular elevation. Capacity between two contour intervals is calculated as the average of the area at the two levels multiplied by the contour interval. Calculations for Mrigi Khal Basin are shown in Table 20. The area capacity curves are presented in **Figure** 19.

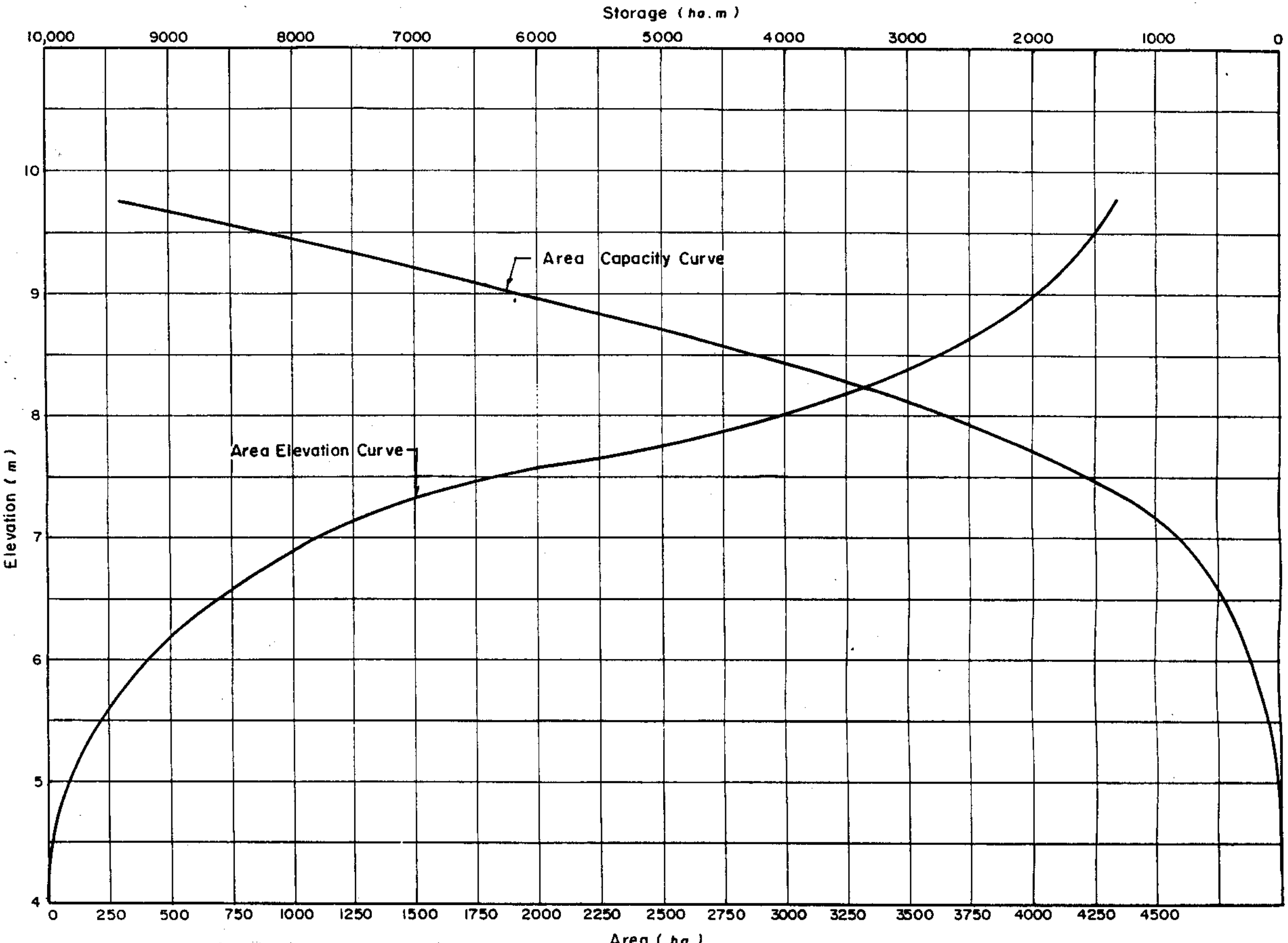


Figure 19. Area Elevation and Storage Elevation Curve of Mrigi Basin

Table 20. Area Capacity Computations

Ground Elevation (m)	Area in (ha)	ΣArea in (ha)	Volume in (ha m)	ΣVolume in (ha m)
4.270	0	0	0	0
4.575	20.46	20.46	3.12	3.12
4.880	40.92	61.38	12.48	15.60
5.185	61.38	122.76	28.08	43.68
5.490	81.85	204.61	49.93	93.61
5.795	102.31	306.92	78.01	171.62
6.100	102.31	409.23	109.21	280.83
6.405	54.39	463.62	133.11	413.94
6.710	206.56	670.18	172.90	586.84
7.015	377.82	1048.00	262.02	848.86
7.320	478.84	1526.84	392.66	1241.52
7.625	552.65	2079.49	549.97	1791.49
7.930	584.38	2663.87	723.36	2514.85
8.235	523.84	3187.71	892.37	3407.22
8.540	446.78	3634.49	1040.39	4447.61
8.845	217.56	3852.05	1141.70	5589.31
9.150	228.25	4080.30	1209.68	6798.99
9.455	195.55	4275.85	1274.31	8073.30
9.760	57.3	4333.15	1312.87	9386.17

11.3 Stage Discharge Curve

For pre-monsoon routing condition i.e. when discharge is a function of storage level only the flow condition changes between condition 5 to condition 3 when the head water depth is 1.5 times the vent height.

11.4 **Characteristics of Condition 5**

Inlet not submerged and low tail water level: When the head water depth is less than 1.5 times the vent height the entrance is not sealed by water and the regulator acts as a broad crested weir. The tail-water depth is less than critical flow depth through the vent. This type will usually occur when a stilling pool or downstream drop is a part of the structure and the flow is made to pass through critical depth. A hydraulic jump will usually occur down stream. The control is at the outlet.

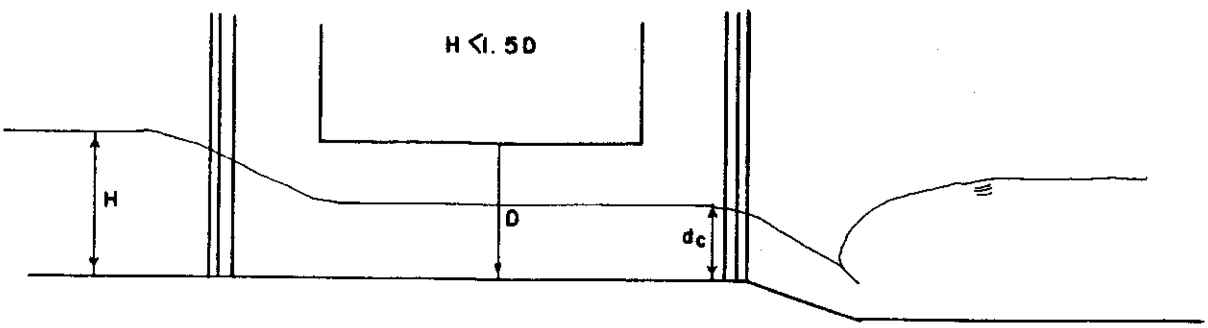


Fig. 20. Condition 5

11.5 **Characteristics of condition 3**

Inlet submerged, outlet not submerged and vent hydraulically short: The vent is short and will not flow full. It is the usual condition occurring in regulators when the head water depth is greater than 1.5 times the vent height. The control is at entrance and is similar to orifice discharge.

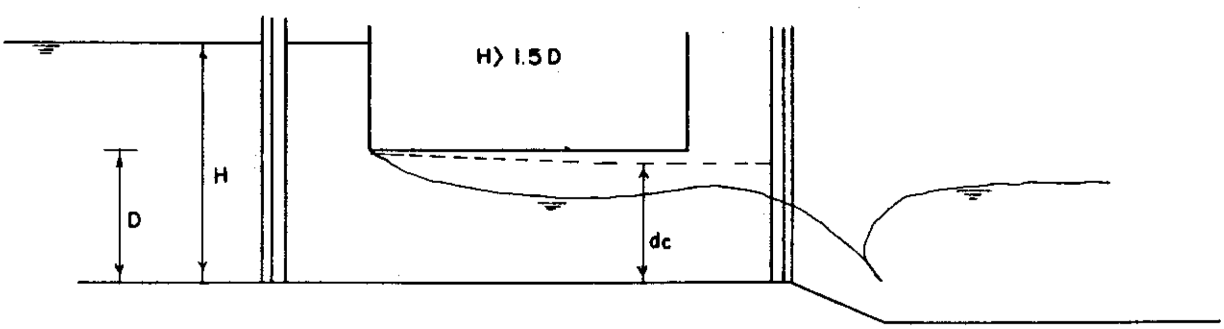


Fig. 21. Condition 3

Invert level of the structure is fixed at RL + 4.27 m (PWD). The discharge under condition 5 & 3 are computed for different elevation and entered in Table 21. **Figure** 22 shows the stage discharge curve.

11.6 Discharge Formula for Condition 5

$q = C_2 H^{3/2}$ per meter width

$C_2 = 1.49$

For 3 vents (1.52 m x 1.83 m)

$Q = 4.56 \times 1.49 H^{3/2}$

$= 6.79 H^{3/2}$

11.7 Discharge Formula for Condition 3

$q = CqD (2gH)^{1/2}$ per metre width

$= Cq \times 1.83 (2 \times 9.81 \times H)^{1/2}$

$= 8.11 CqH^{1/2}$

For 3 vents (1.52 m x 1.83 m)

$Q = 3 \times 1.52 \times 8.11 CqH^{1/2}$

$= 36.98 CqH^{1/2}$

C_q is obtained from **Figure** 23 for the corresponding value of D/H

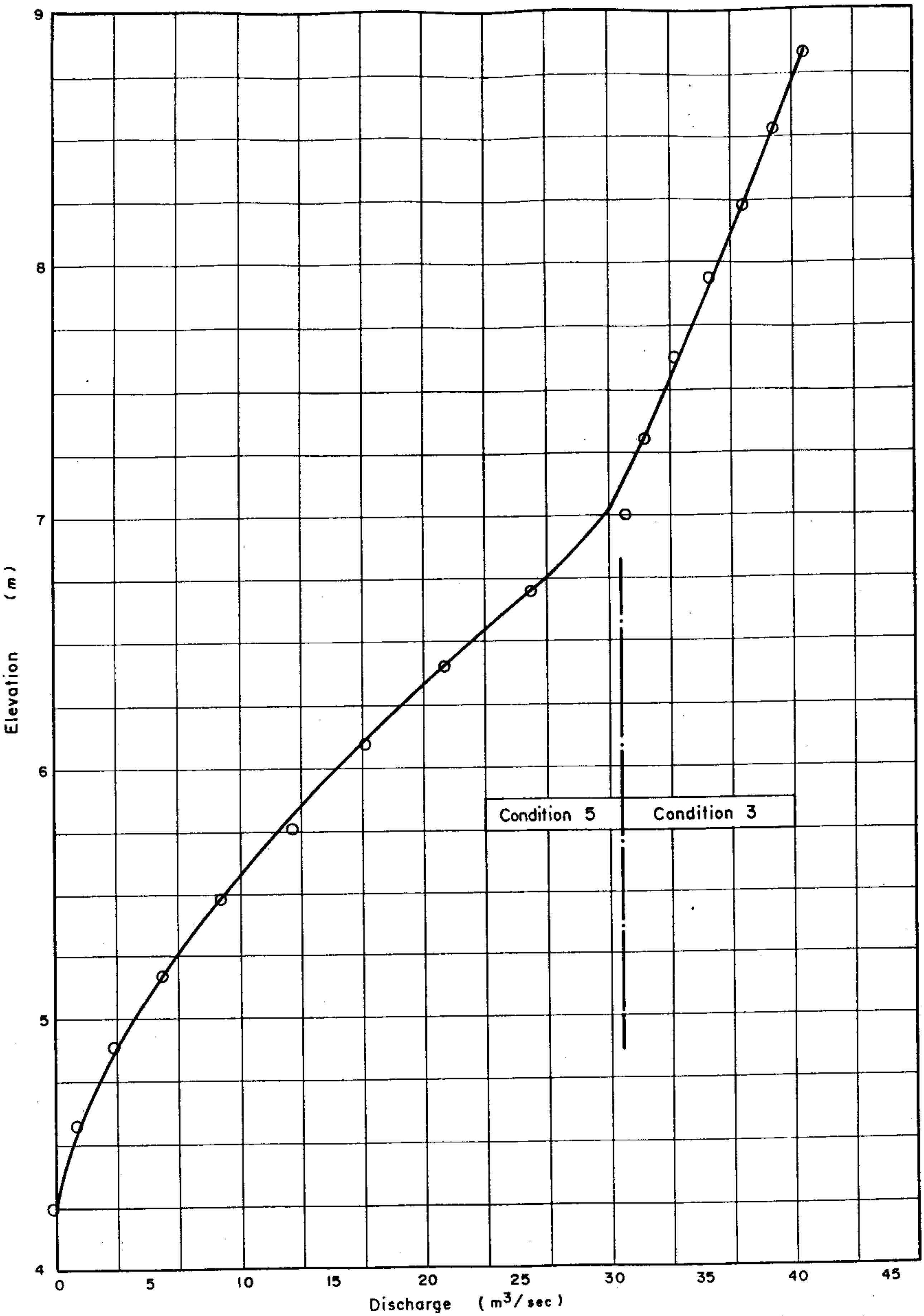


Figure 22. Stage Discharge Curve (3vent-1.52 m x 1.83 m)

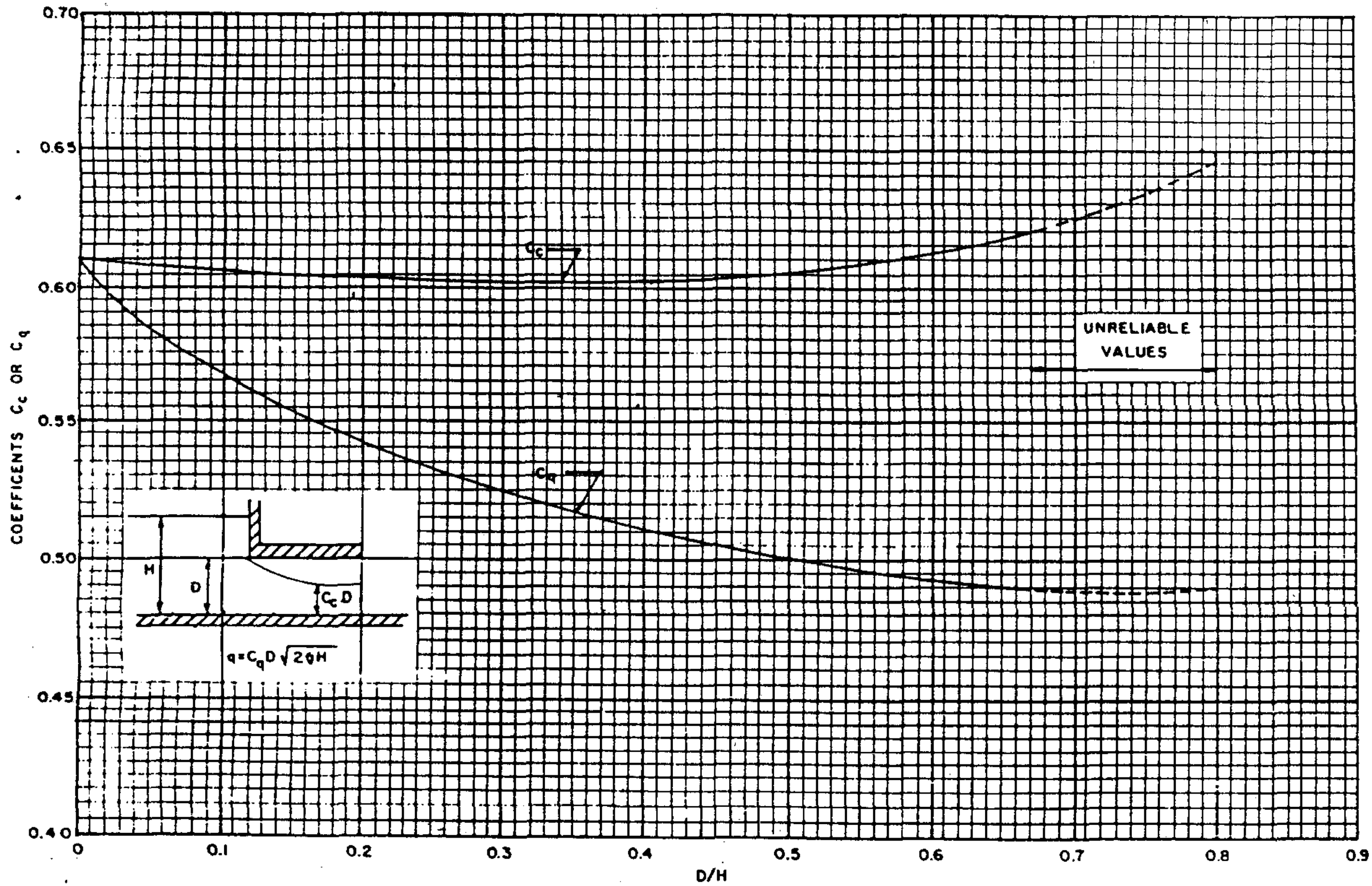


Figure 23. Discharge Co-efficients for Flow Condition 3

Table 21. Stage Discharge chart

(3 Vents 1.52m x 1.83m)

R.L (m PWD)	H in m	Flow Condition	D/H	Cq	Q in m3/Sec
4.270	0	-			0
4.575	0.305	5	-	-	1.14
4.880	0.610	5	-	-	3.23
5.185	0.915	5	-	-	5.94
5.490	1.220	5	-	-	9.15
5.795	1.525	5	-	-	12.79
6.100	1.830	5	-	-	16.81
6.405	2.135	5	-	-	21.18
6.710	2.440	5	-	-	25.88
7.015	2.745	5	-	-	30.88
7.320	3.050	3	0.6	0.493	31.84
7.625	3.355	3	0.55	0.496	33.60
7.930	3.660	3	0.50	0.500	35.37
8.235	3.965	3	0.46	0.504	37.11
8.540	4.270	3	0.43	0.507	38.74
8.845	4.575	3	0.40	0.511	40.42
9.150	4.880	3	0.38	0.514	41.99

11.8 2S/T + D Curve

The computations for this curve are performed in Table 22. Column (2) is the storage capacity in ha m taken from Table 20. Column 3 is storage capacity in cubic-metre (1 ha m = 10,000 cu. m) divided by time. We have selected a time interval of 21,600 seconds (6 hours). Discharge in column (4) is taken from the stage discharge chart (Table 21). Column (4) Vs column (5) is plotted on **Figure 24.**

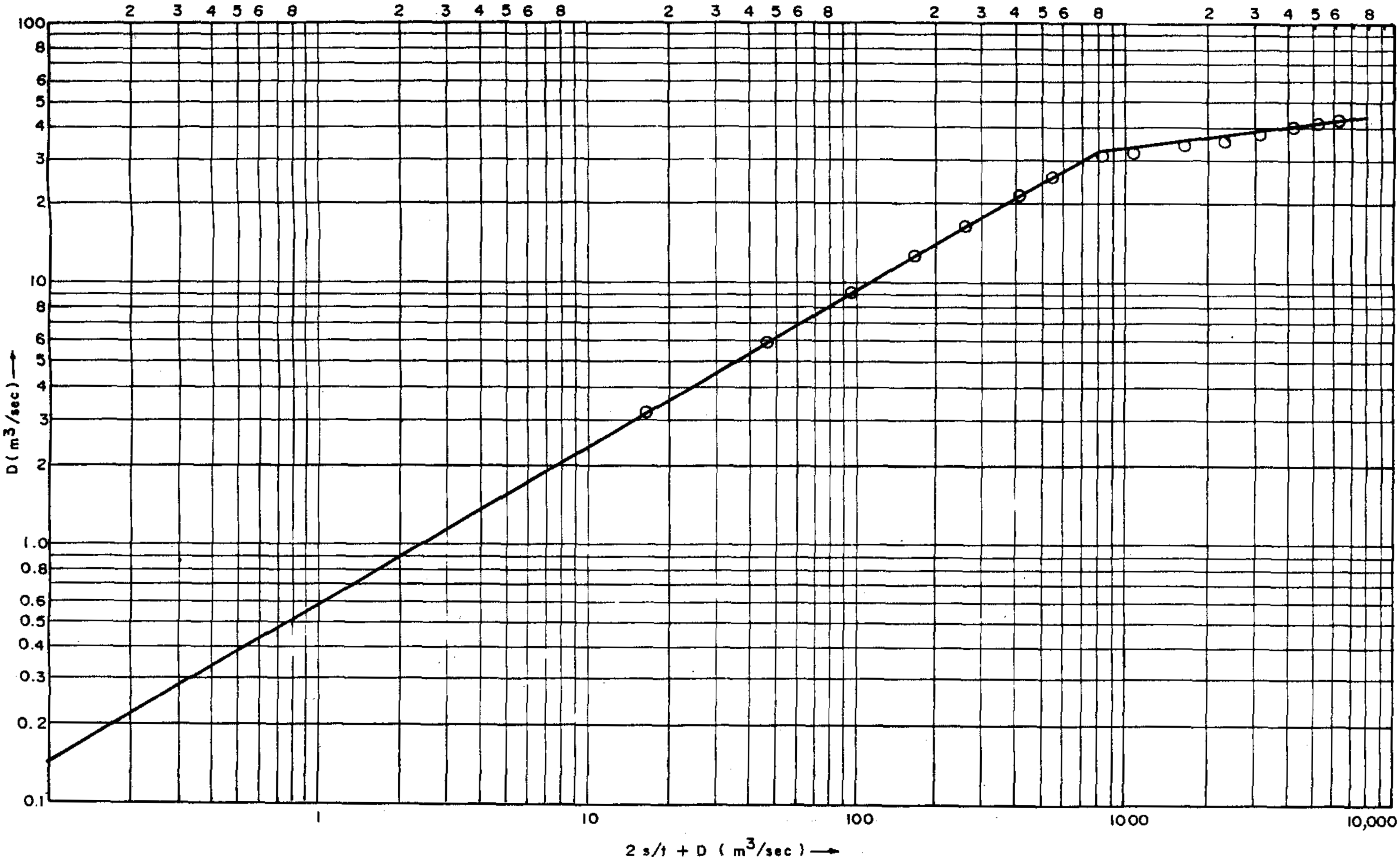


Figure 24. D vs 2S/t + D (From Table 22)

Table 22. Computations for 2s/t + D Curve

Elevation in m(PWD)	Storage Volume(s) in ha. m	2s/t in m^3/Sec	D in m^3/Sec	2s/t + D in m^3/Sec
(1)	(2)	(3)	(4)	(5)
4.270	0	0	0	0
4.575	3.12	2.89	1.14	4.03
4.880	15.60	14.44	3.23	17.67
5.185	43.68	40.44	5.94	46.38
5.490	93.61	86.68	9.15	95.83
5.795	171.62	158.91	12.79	171.70
6.100	280.83	260.03	16.81	276.84
6.405	413.94	383.28	21.18	404.46
6.710	586.84	543.37	25.88	569.25
7.015	848.86	785.98	30.88	816.86
7.320	1241.52	1149.56	31.84	1181.40
7.625	1791.49	1658.79	33.60	1692.39
7.930	2514.85	2328.56	35.37	2363.93
8.235	3407.22	3154.83	37.11	3191.94
8.540	4447.61	4118.16	38.74	4156.90
8.845	5589.31	5175.29	40.42	5215.71
9.150	6798.99	6295.36	41.99	6337.35

11.9 Pre-monsoon Flood Routing Example (3 Vents 1.52m x 1.83m)

The design flood, represented by the runoff hydrograph in **Figure** 15 will now be routed through a 3-vent regulator, each vent is 1.52m wide and 1.83m high. The computations are done in Table 23, 24 and 25. The procedure is as follows:

1) The first value in column (6) and (8) are known or assumed.
2) The first value in column (5) is the value of 2s/t + D taken from **Figure** 24 corresponding to the known value of D, in this case zero.
3) In subsequent lines column (5) is column (4) plus column (7) from the line above.

4) Column (7) is column (5) minus twice column (6) in each case throughout the computations.

5) After the first line, value of D in column (6) are taken from **Figure** 24 corresponding to the value of 2s/t + D on the same line.

6) Column (8) is the basin water surface elevation taken from **Figure** 22 for the corresponding value of D in column (6).

After completing the computations the outflow hydrograph i.e. discharge Vs time is plotted and presented in **Figure** 25. To assist in determining the extent of crop damage, a basin water surface elevation Vs time is also plotted and presented in **Figure** 26. This completes the flood routing procedure.

Comparison of inflow and outflow hydrographs for unit hydrograph method, Richard's method and SCS method are presented in **Figures** 27, 28 and 29 respectively.

Pre-monsoon flood routing computations by Unit Hydrograph method, Richard's method and SCS method are presented in Tables 23, 24 and 25 respectively while the results of the computations are discussed in Article 12.0

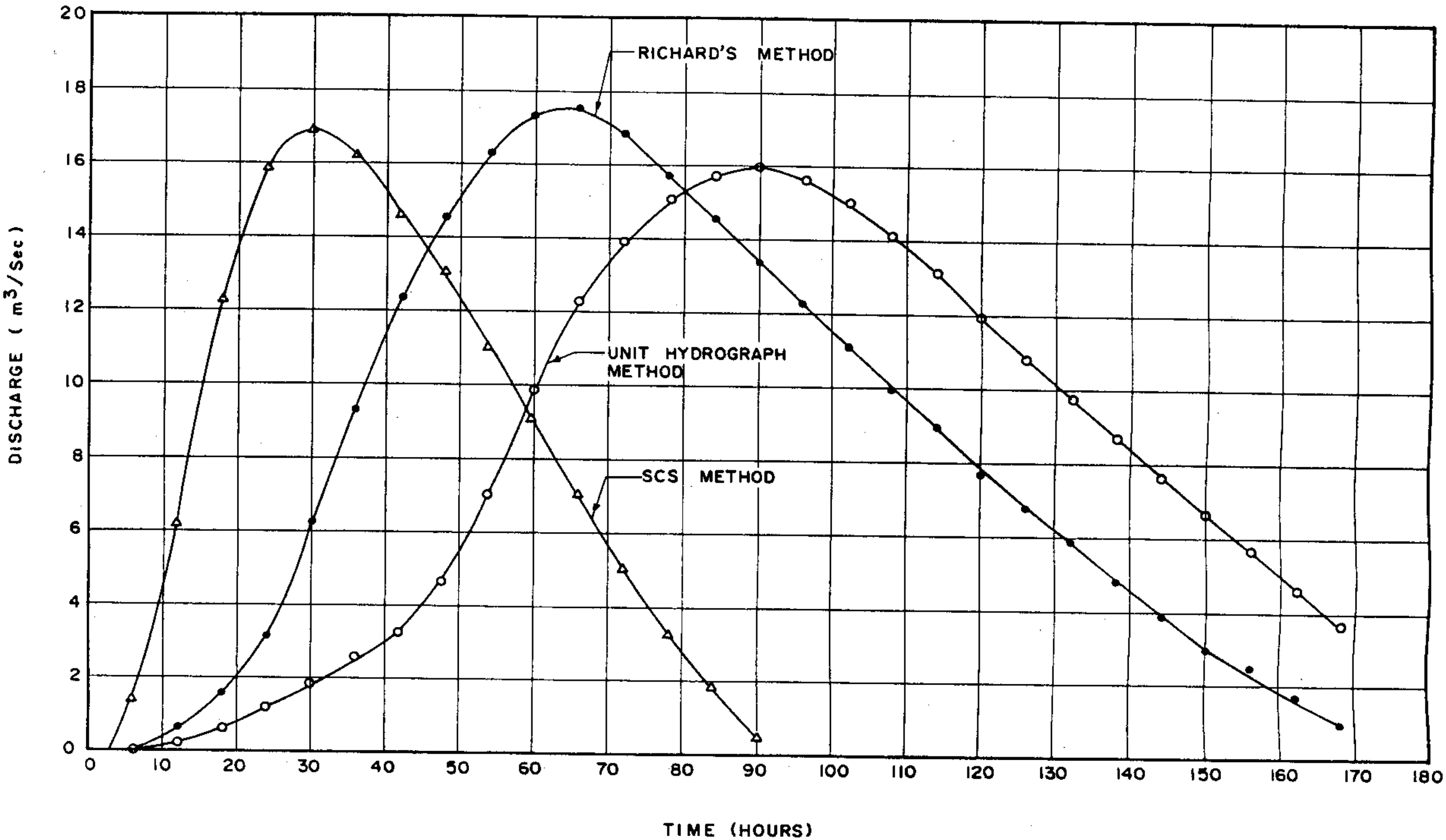


Figure 25. Outflow Hydrograph (3 vents-1.52m x 1.83m)

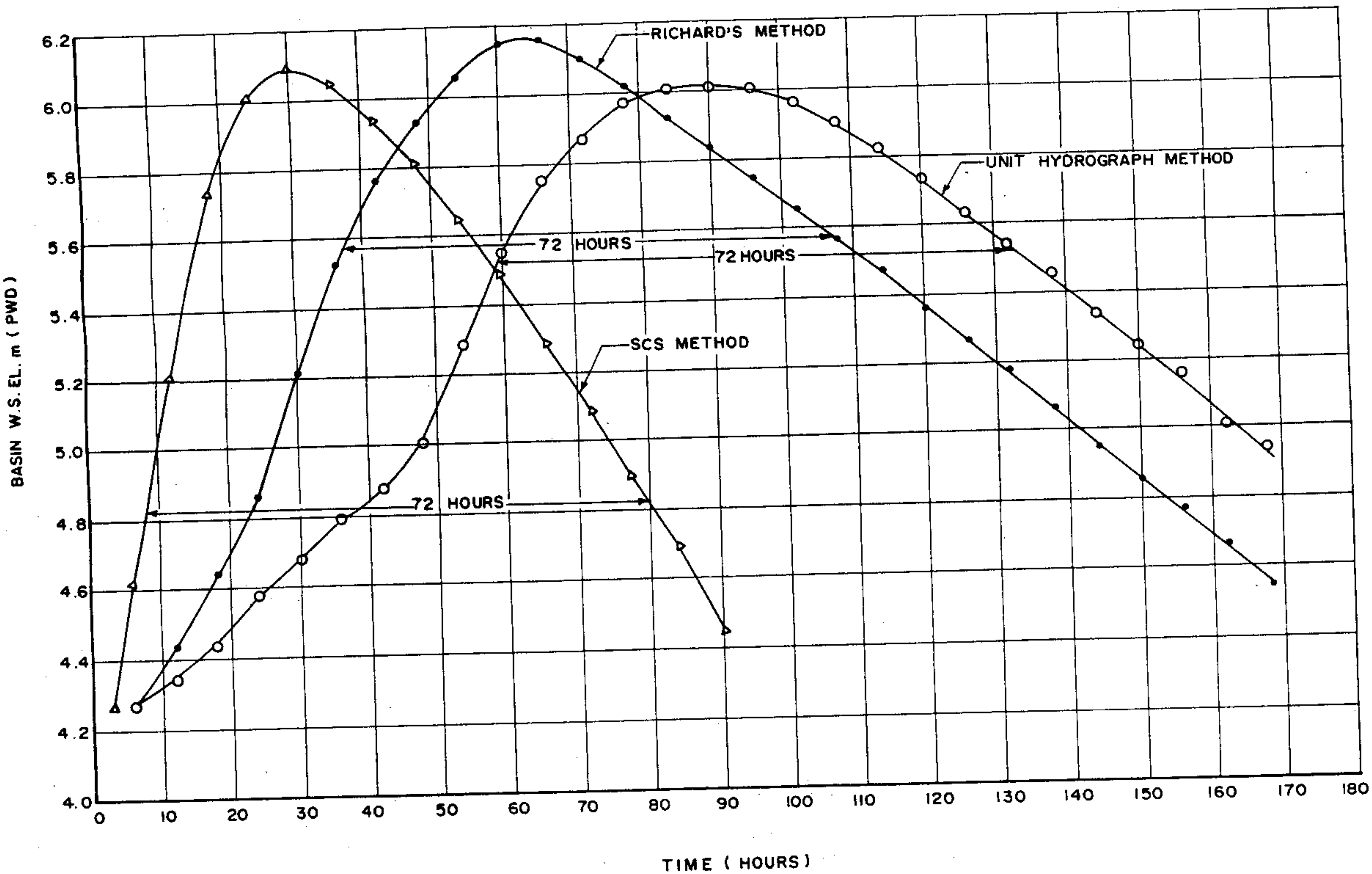


Figure 26. Stage Time Relation (3 vents - 1.52 m x 1.83 m)

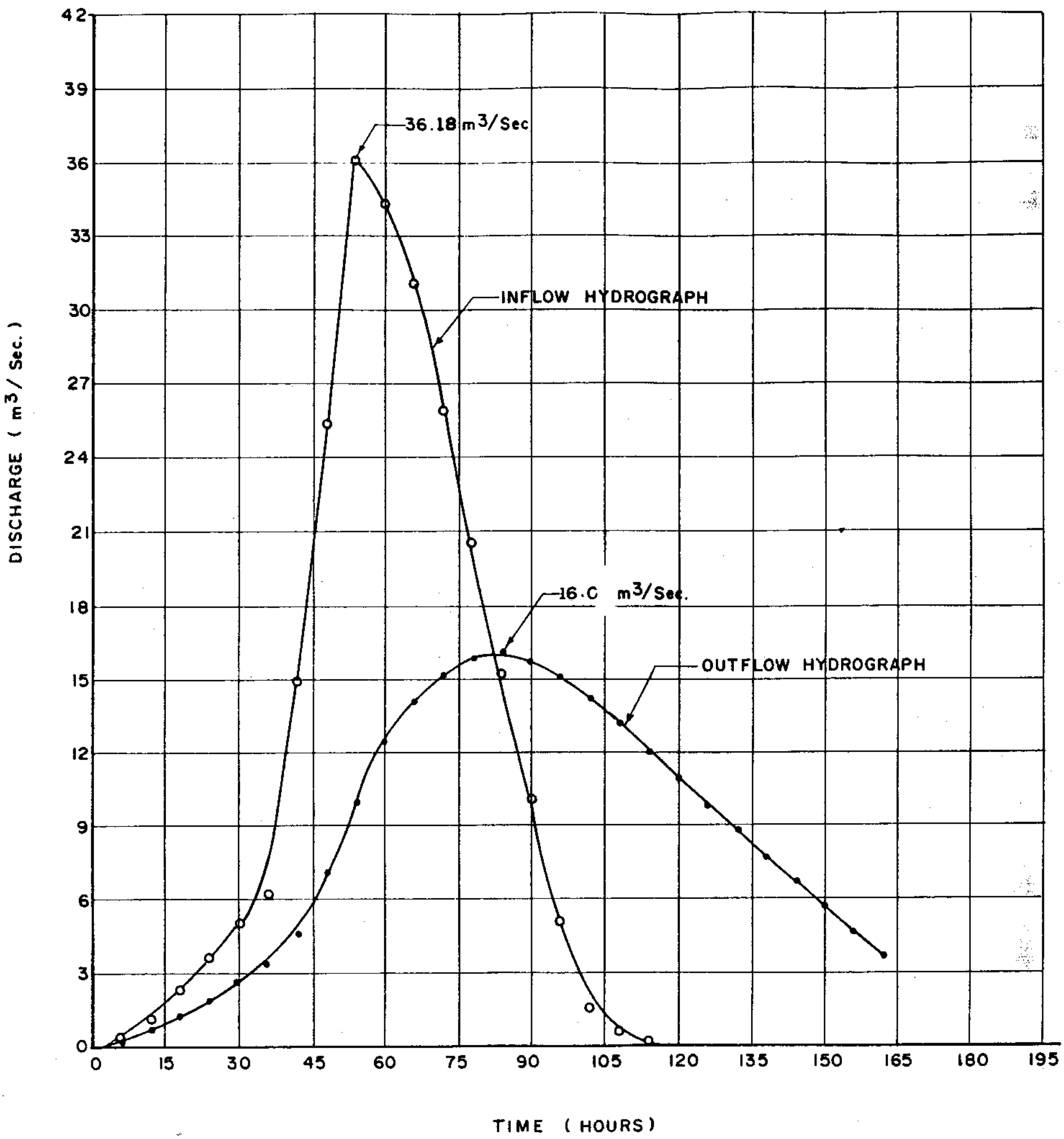


Figure 27. Inflow and Outflow Hydrograph (Unit Hydrograph Method)

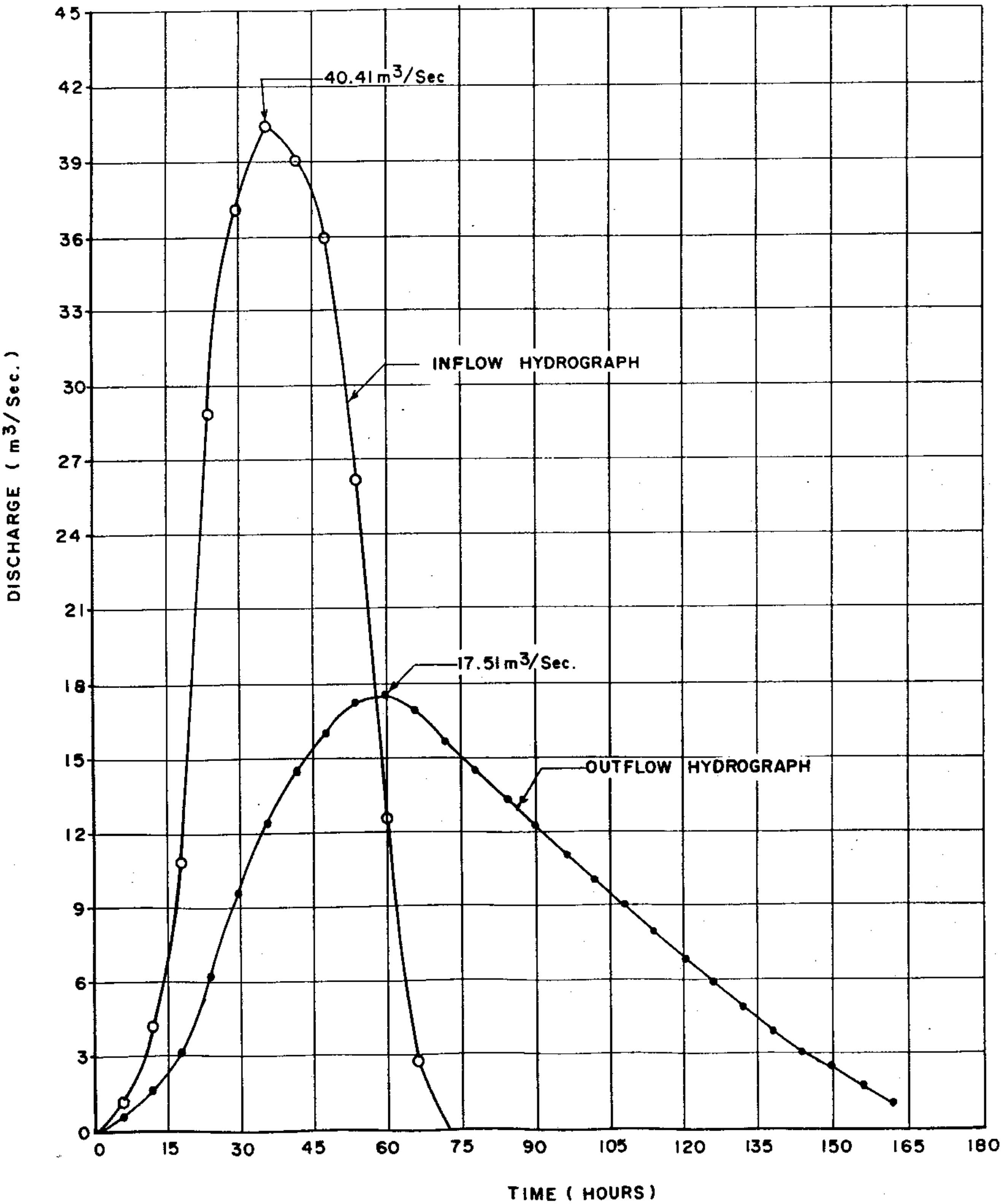


Figure 28. Inflow and Outflow Hydrograph (Richards Method)

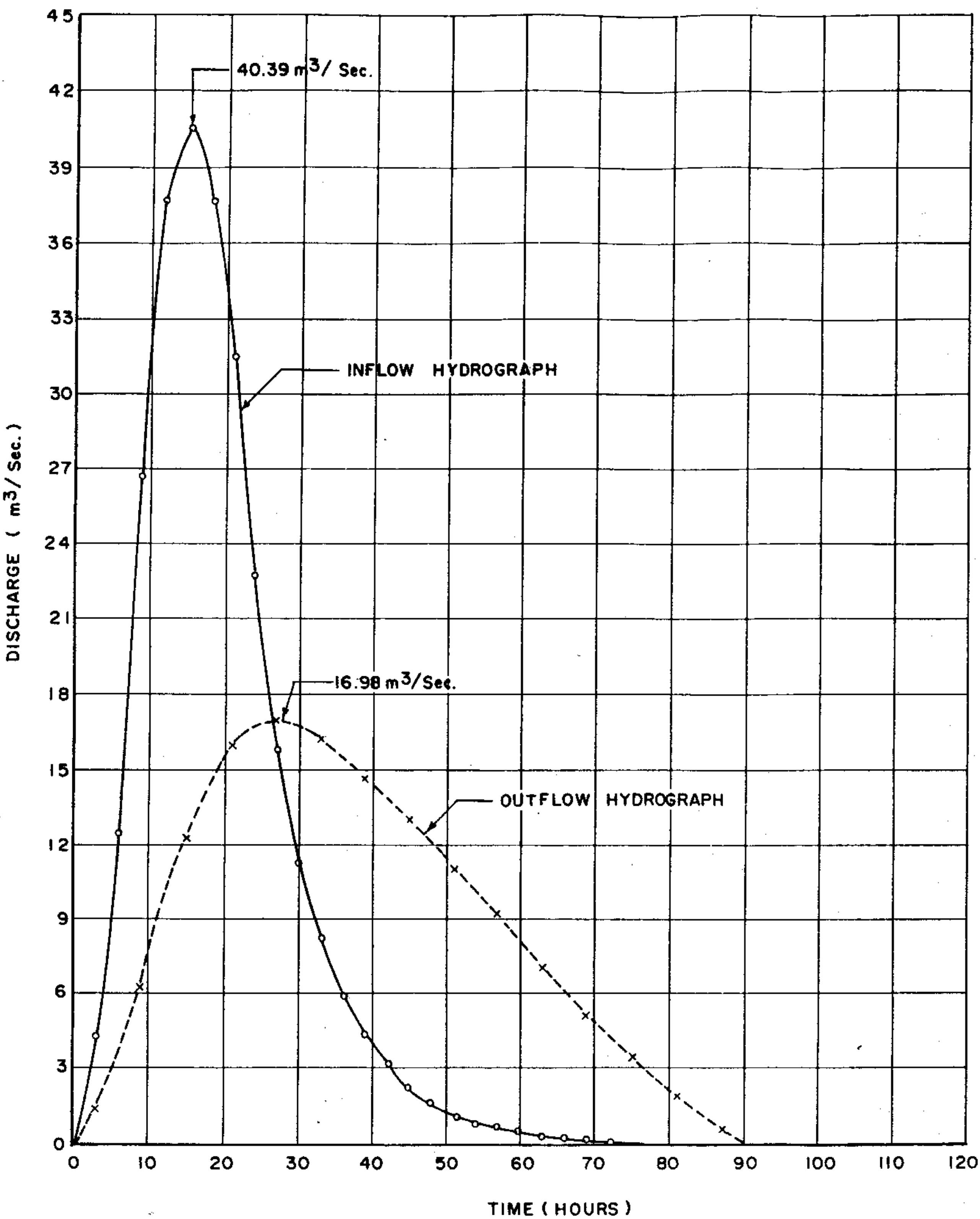


Figure 2.9. Inflow and Out flow Hydrograph (SCS method)

Table 23. Pre-Monsoon Flood Routing Computation By Unit Hydrograph Method (3 Vents 1.52 m X 1.83 m)

Day	Hour	I (m^3/sec)	I1+I2 (m^3/sec)	$\frac{2s}{t}$ + D (4) + (7)	D (m^3/sec)	$\frac{2s}{t}$ - D (5)-2x(6)	Basin Water Surface Elevation(m.PWD)
1	2	3	4	5	6	7	8
	6	0	0	0	0	0	4.27
1	12	0.34	0.34	0.34	0.28	-0.22	4.34
	18	1.09	1.43	1.21	0.62	-0.03	4.44
	24	2.39	3.48	3.45	1.20	1.05	4.58
	6	3.60	5.99	7.04	1.85	3.34	4.68
2	12	4.89	8.49	11.83	2.60	6.63	4.79
	18	6.15	11.04	17.67	3.23	11.21	4.88
	24	14.90	21.05	32.26	4.61	23.04	5.04
	6	25.40	40.30	63.34	7.04	49.26	5.29
3	12	36.18	61.58	110.84	9.87	91.10	5.55
	18	34.35	70.53	161.63	12.31	137.01	5.75
	24	31.08	65.43	202.44	13.97	174.50	5.87
	6	26.01	57.09	231.59	15.08	201.43	5.97
4	12	20.57	46.58	208.01	15.71	216.59	6.02
	18	15.20	35.77	252.36	16.00	220.36	6.04
	24	10.03	25.23	245.59	15.64	214.31	6.01
	6	5.10	15.13	229.44	15.00	199.44	5.96
5	12	1.63	6.73	206.17	14.11	177.95	5.90
	18	0.65	2.28	180.23	13.12	153.99	5.82
	24	0.12	0.77	154.76	11.98	130.80	5.73
	6	0	0.12	130.92	10.84	109.24	5.63
6	12	0	0	109.24	9.80	89.64	5.54
	18	0	0	89.64	8.75	72.14	5.45
	24	0	0	72.14	7.62	56.90	5.34
	6	0	0	56.90	6.63	43.64	5.25
7	12	0	0	43.64	5.69	32.26	5.16
	18	0	0	32.26	4.61	23.04	5.04
	24	0	0	23.04	3.74	15.56	4.94

Table 24. Pre-Monsoon Flood Routing Computation By Richard's Method

(3 vents 1.52 m x 1.83 m)

Day	Hour	I (m^3/sec)	I1+I2 (m^3/sec)	$\frac{2s}{t}$ + D (4) + (7)	D (m^3/sec)	$\frac{2s}{t}$ - D (5)-2x(6)	Basin Water Surface Elevation(m.PWD)
1	2	3	4	5	6	7	8
	6	0	0	0	0	0	4.27
1	12	1.20	1.20	1.20	0.61	-0.02	4.43
	18	4.20	5.40	5.38	1.60	2.18	4.64
	24	10.80	15.00	17.18	3.10	10.98	4.86
	6	28.90	39.70	50.68	6.22	38.24	5.21
2	12	37.20	66.10	104.34	9.56	85.22	5.52
	18	40.41	77.61	162.83	12.36	138.11	5.76
	24	39.10	79.51	217.62	14.55	188.52	5.93
	6	36.00	75.10	263.62	16.31	231.00	6.06
3	12	26.20	62.20	293.20	17.37	258.46	6.14
	18	12.60	38.80	297.26	17.51	262.24	6.15
	24	2.80	15.40	277.64	16.84	243.93	6.10
	6	0	2.80	246.76	15.66	215.44	6.01
4	12	0	0	215.44	14.46	186.52	5.92
	18	0	0	186.52	13.36	159.80	5.84
	24	0	0	159.80	12.22	135.36	5.75
	6	0	0	135.36	11.05	113.26	5.65
5	12	0	0	113.26	9.99	93.28	5.56
	18	0	0	93.28	8.98	75.32	5.47
	24	0	0	75.32	7.82	59.68	5.36
	6	0	0	59.68	6.80	46.08	5.27
6	12	0	0	46.08	5.91	34.26	5.18
	18	0	0	34.26	4.80	24.66	5.06
	24	0	0	24.66	3.89	16.88	4.95
	6	0	0	16.88	3.00	10.88	4.85
7	12	0	0	10.88	2.50	5.88	4.77
	18	0	0	5.88	1.70	2.48	4.66
	24	0	0	2.48	1.00	0.48	4.54

Table 25. Pre-Monsoon Flood Routing Computation By SCS Method

(3 vents 1.52 m x 1.83 m)

Day	Hour	I (m^3/sec)	I1+I2 (m^3/sec)	$\frac{2s}{t}$ + D (4) + (7)	D (m^3/sec)	$\frac{2s}{t}$ - D (5)-2x(6)	Basin Water Surface Elevation(m.PWD)
1	2	3	4	5	6	7	8
	3	0	0	0	0	0	4.27
	6	4.04	4.04	4.04	1.40	1.24	4.61
	9	12.52	16.56	17.80	3.24	11.34	4.88
1	12	26.66	39.18	50.52	6.21	38.10	5.21
	15	37.56	64.22	102.32	9.46	83.40	5.52
	18	40.39	77.95	161.36	12.29	136.77	5.75
	21	37.56	77.95	214.75	14.44	185.87	5.92
	24	31.50	69.06	254.93	15.97	222.99	6.04
	3	22.62	54.12	277.11	16.82	243.47	6.10
	6	15.75	38.37	281.84	16.98	247.88	6.11
	9	11.31	27.06	274.94	16.74	241.46	6.09
2	12	8.36	19.67	261.13	16.21	228.71	6.05
	15	5.94	14.30	243.01	15.52	211.97	6.00
	18	4.32	10.26	222.23	14.72	192.79	5.94
	21	3.11	7.43	200.22	13.88	172.46	5.88
	24	2.22	5.33	177.79	13.02	151.75	5.81
	3	1.62	3.84	155.59	12.02	131.55	5.73
3	6	1.17	2.79	134.34	11.00	112.34	5.65
	9	0.85	2.02	114.36	10.04	94.28	5.57
	12	0.61	1.46	95.74	9.14	77.46	5.49
	15	0.44	1.05	78.51	8.03	62.45	5.38
	18	0.30	0.74	63.19	7.03	49.13	5.29
	21	0.20	0.50	49.63	6.15	37.33	5.20
	24	0.10	0.30	37.63	5.11	27.41	5.09
	3	0.05	0.15	27.56	4.16	19.24	4.98
	6	0	0.05	19.29	3.38	12.53	4.90
	9	0	0	12.53	2.70	7.13	4.80
4	12	0	0	7.13	1.90	3.33	4.69
	15	0	0	3.33	1.15	1.03	4.58
	18	0	0	1.03	0.59	0	4.43
	21	0	0	0	0	0	4.27

12.0 POST MONSOON FLOOD ROUTING

As the river stage starts rising the gates of the regulator is closed before the flooding inside the project area exceeds the tolerable inundation level. For our case it is + 6.00m (PWD) which the outfall river attains on 15th June (**Figure** 30). During the period the gates are closed and flood water cannot enter in to the project area provided the project area is completely empoldered, but the excess rainfall runoff will be accumulated inside the basin. Computations of rainfall runoff volume for non-paddy land, paddy land and weighted basin average for different months are presented in Tables 26, 27 and 28 respectively. The basin water surface elevation after each month is computed in Table 29 and presented in **Figure** 30. It may be observed that the basin water level always remains below the level in the peripheral rivers. The difference is the benefit accrued out of the project by reclaiming land which were subjected to annual flooding during pre-project condition. It may also be observed from **Figure** 30 that at falling limb of the outfall hydrograph on 9th October the river water level coincides with the basin water level. The gates are opened on the day and the post monsoon routing begins. The computations are presented in Table 30.

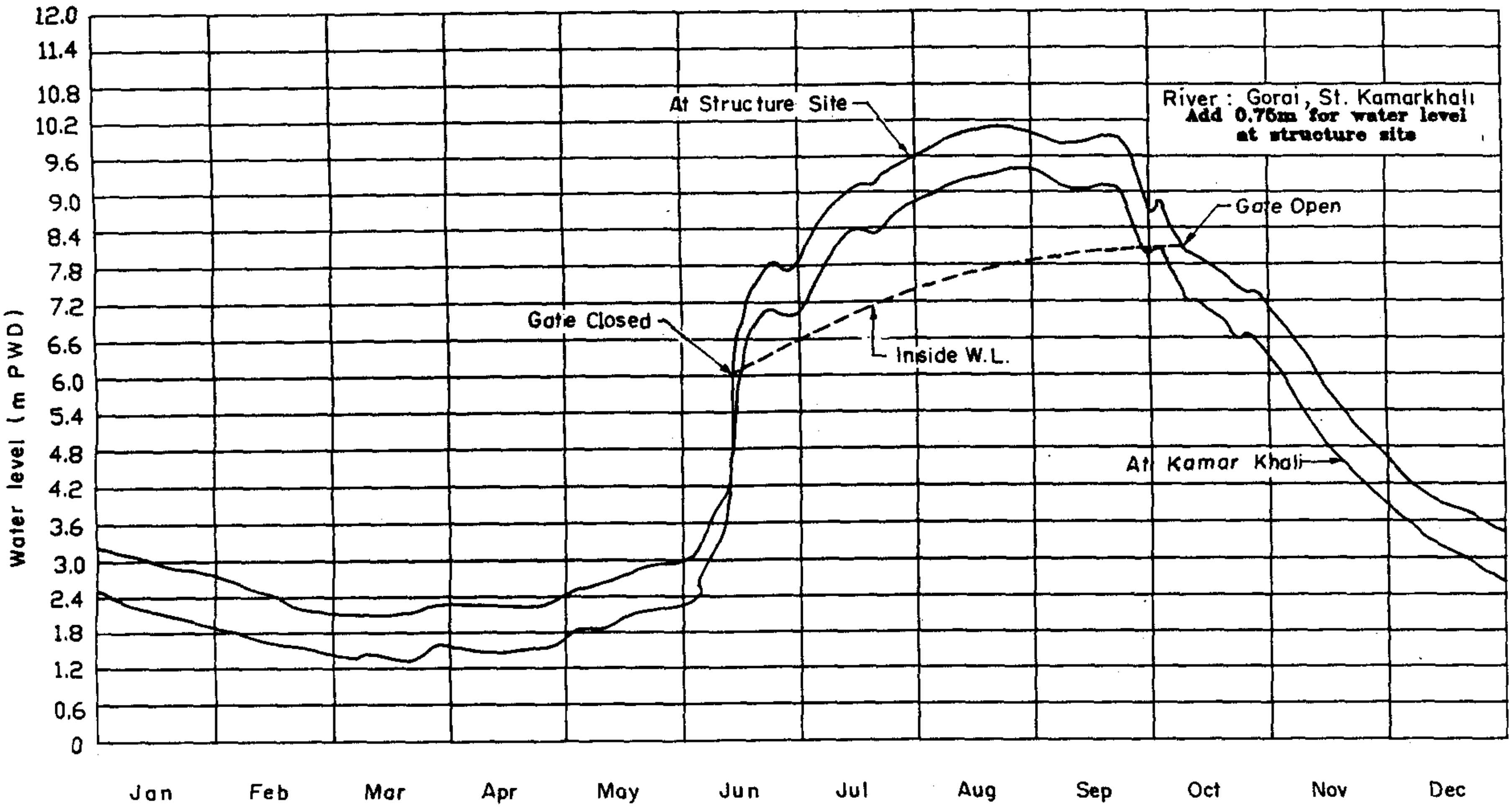


Figure 30. Mrigi Khal Regulator Water Level Hydrograph and Basin Water Level

Table 26. Rainfall Runoff Volume (Non Paddy Land)

Month	Monthly Rainfall (mm)	Monthly evapo-transpiration (mm)	Initial soil Moisture loss (mm)	Depression storage (mm)	Percolation loss (mm)	Runoff (mm)	Runoff volume (ha.m)
June	355	114	13	25	52	151	654.31
July	360	128	-	-	52	180	780.00
August	374	128	-	-	52	194	840.63
Sept.	246	119	-	-	52	75	324.99
Oct.	162	117	-	-	52	0	0.00

Table 27. Rainfall Runoff Volume (Paddy Land)

Month	Monthly Rainfall (mm)	Monthly evapo-transpiration (mm)	Initial soil Moisture loss (mm)	Depression storage (mm)	Runoff (mm)	Runoff volume (ha.m)
June	355	134	-	100	121	524.31
July	360	151	-	-	209	905.63
August	374	150	-	-	224	970.63
Sept.	246	140	-	-	106	459.31
Oct.	162	137	-	-	25	108.33

Table 28. Rainfall Runoff Volume (Weighted Basin Average)

Month	Non paddy land 50%		Paddy land (50%)		Basin Weighted Run off
	Net Runoff (ha.m)	Weighted Run off (ha.m)	Net Run off (ha.m)	Weighted Run off (ha.m)	(ha.m)
June	654.31	327.16	524.31	262.16	589.32
July	780.00	390.00	905.63	452.82	842.82
August	840.63	420.32	970.63	485.32	905.64
Sept.	324.99	162.50	459.31	229.66	392.16
Oct.	0.00	0.00	108.33	54.17	54.17

Table 29. Computation of Inside Storage and Water Level

Date	River W/L (in m.PWD)	Run off volume (ha.m)	Gate position	Inside	
				Storage (ha.m)	Water level (in m.PWD)
15th June	6.0	245.02	Gate closed	245.02	6.00
30th June	7.7	294.66		539.68	6.63
31st July	9.5	842.82		1382.50	7.40
31st August	9.9	905.64		2288.14	7.83
30th Sept.	8.5	392.16		2680.30	7.99
9th Oct.	8.0	15.73	Gate opened	2696.03	8.00

Table 30. Post Monsoon Flood Routing Computations(3-vents 1.52m x 1.83m)

Date	River Stage m(PWD)	Trial Basin Storage Level m(PWD)	Ave. Head Water Depth H above invert (m)	Ave. Tail Water Depth D above invert (m)	h (H-D) (m)	Ave. Out-flow (ha.m)	Computed residual Basin storage (ha.m)	Computed Basin storage level m(PWD)
oct.9	8.00	8.00	3.730	3.730	0	0	2696.03	8.00
10	7.95	7.98	3.720	3.705	0.015	32.07	2663.96	7.98
11	7.90	7.96	3.700	3.655	0.045	55.55	2608.41	7.96
12	7.85	7.94	3.680	3.605	0.075	71.72	2536.69	7.94
13	7.80	7.90	3.650	3.555	0.095	80.71	2455.98	7.90
14	7.75	7.87	3.615	3.505	0.110	86.85	2369.13	7.87
15	7.70	7.83	3.580	3.455	0.125	22.59	2276.54	7.83
16	7.65	7.79	3.540	3.405	0.135	96.22	2180.32	7.79
17	7.60	7.75	3.500	3.355	0.145	99.72	2080.60	7.75
18	7.55	7.70	3.455	3.305	0.150	101.42	1979.18	7.70
19	7.50	7.66	3.410	3.255	0.155	103.10	1876.08	7.66
20	7.45	7.61	3.365	3.205	0.160	104.75	1771.33	7.61
21	7.40	7.56	3.315	3.155	0.160	104.75	1666.58	7.56
22	7.35	7.50	3.260	3.105	0.155	103.10	1563.48	7.50
23	7.30	7.44	3.200	3.055	0.145	99.72	1463.76	7.44
24	7.25	7.39	3.145	3.005	0.140	97.98	1365.78	7.39
25	7.20	7.33	3.090	2.955	0.135	96.22	1269.56	7.33
26	7.15	7.27	3.030	2.905	0.125	92.59	1176.97	7.27
27	7.10	7.20	2.965	2.855	0.110	86.85	1090.12	7.20
28	7.05	7.14	2.900	2.805	0.095	80.71	1009.41	7.14
29	6.97	7.08	2.840	2.740	0.100	82.81	926.60	7.08
30	6.89	7.00	2.770	2.660	0.110	86.85	839.75	7.00
31	6.81	6.90	2.680	2.580	0.100	82.81	756.94	6.90
Nov.1	6.73	6.82	2.590	2.500	0.090	78.56	678.38	6.82
2	6.65	6.73	2.505	2.420	0.085	76.35	602.03	6.73
3	6.57	6.62	2.405	2.340	0.065	66.76	535.27	6.62
4	6.49	6.52	2.300	2.260	0.040	52.37	482.90	6.52
5	6.41	6.44	2.210	2.180	0.030	45.36	437.54	6.44

13.0 DISCUSSION AND CONCLUSION

The results of the pre-monsoon routing for 3 vents of size 1.52 m x 1.83 m are tabulated in Table 31.

Table 31. Pre-Monsoon Flood Routing Results (3 vents 1.52 m x 1.83 m)

Item	Unit Hydrograph Method	Richard's Method	SCS Method
1) Runoff hydrograph peak discharge	36.19 m^3/sec	40.41 m^3/sec	40.39 m^3/sec
2) Time to runoff hydrograph peak	54 hours	36 hours	15 hours
3) Time base of runoff hydrograph	120 hours	72 hours	75 hours
4) Out flow hydrograph peak	16.0 m^3/sec	17.51 m^3/sec	16.98 m^3/sec
5) Basin storage level corresponding to peak discharge	+ 6.04 m (PWD)	+ 6.15(PWD)	+6.11m(PWD)
6) Percentage of area submerged corresponding to peak discharge	8.98 %	9.65 %	9.49 %
7) Basin submergence level for more than 72 hours	+ 5.53 m (PWD)	+ 5.56 m (PWD)	+ 4.82 m(PWD)
8) Percentage of area submerged for more than 72 hours	+ 5.03 %	5.26 %	1.23 %

From Table 31 it is observed that out of the three methods, Richards method is little more conservative & dictates the designs to go for next higher size of the structure.

Considering the simplicity of computation by Richard's method & SCS method when there is time constraint for detail design computation, the Richard's method or SCS method for generating runoff hydrograph may be used.

The computed size of the structure selected by pre-monsoon routing should also be checked by post-monsoon routing. In non tidal zone it is observed that the size of the structure is mostly governed by post-monsoon conditions. However when more sophistication in design computations is required, specially for a large structure the conventional unit hydrograph method for runoff hydrograph computation may be recommended. For Mrigi Khal regulator the results of post-monsoon routing shows that the maximum difference of water level between C/S and R/S is 16 cm for a 3 vents regulator of size 1.52 m x 1.83 m. It may therefore be concluded that a 3 vents drainage regulator of size 1.52 m x 1.83 m placed at the outfall of Mrigi Khal is adequate for Mrigi Basin.

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CHAPTER FIVE

HYDRAULIC DESIGN OF REGULATORS, *DRAINAGE SLUICES AND EMBANKMENT*

G. M. Akram Hossain

HYDRAULIC DESIGN OF REGULATORS, DRAINAGE SLUICES AND EMBANKMENT

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Abstract

This chapter gives an outline of the design considerations usually adopted in the hydraulic design of regulators, drainage sluices and water retention structure with some specific design examples. The design considerations for embankment is also included in this chapter with example.

The hydraulic design for regulators and sluices is divided into eight subsections which are the: i) Design Flood Frequency; ii) Invert level Fixation; iii) Design Head Estimation; iv) Scour Depth and Cutoff Wall; v) Stilling Basin Design; vi) Exit Gradient and Floor Length; vii) Uplift Pressure and Floor Thickness and viii) Loose Protective Works. All these design parameters are discussed in the text briefly and the considerations are applied in design examples. The design procedure is mostly based on the existing Bangladesh Water Development Board (BWDB) design practice. In the design of cut off wall, lacey's depth of scour is considered applying standard safety factor. The design of stilling basin is described in details with different flow conditions. Various alternatives with USBR Stilling Basins, Low Froude Number Basins, Saint Anthony Falls (SAF) Basins and Indian Standard Stilling Basins are discussed. Khosla,s design methodology is adopted in the design for safe exit gradient and floor thickness.

The design of embankment is also divided into five subsections. They are the: i) Design crest level; ii) Design crest width; iii) Provision for berms; iv) Set back distance and v) Stability of embankment. The embankment design procedure described in the text are in line with the existing BWDB design practice. In the design for crest level, considerations are given to the design flood level (normally 1:20 year level), wave run-up, wind set-up and shrinkage and settlement for the embankment material. Design crest width, berms, set back distances and stability analysis for earthen embankment are also discussed briefly and the considerations are applied in examples.

HYDRAULIC DESIGN OF REGULATORS, DRAINAGE SLUICES AND EMBANKMENT

1.0 INTRODUCTION

Structures designed to control and or to modify the flow of water in channels; natural or man made and, to control the flow of water into or out of a particular area (e.g. a coastal embankment polder) and to control the forces imposed by the flowing water, are called hydraulic structures. Bridges and offshore structures e.g. jetties, quaywalls etc. are also called hydraulic structures but those are not included in this text. Size of structures may vary from small installations (e.g. farm turnouts, small canal structures) handling less than a cubic meter per second to large structures capable of controlling and withstanding flows in excess of 3000 cubic meter per second. This chapter presents in brief the basic guide lines and criteria to be followed in the hydraulic design of small hydraulic structures like regulators, water retention structure and sluices including design of earthen embankment.

2.0 SMALL WATER CONTROL STRUCTURE

Small Scale Flood Control, Drainage and Irrigation Project is involved in the identification, planning, design and construction monitoring of small water control structures projects e.g. regulators, sluices, weirs and water retention structure. A brief description of the functions and utility of these structures are given in the following subsections:

2.1 Regulators

Regulators are gated structure which permit passage of water in and out of a particular area at a controlled rate. These structures are provided with vertical lift gate with mechanical hoisting devices for easy operation. For small regulators, the gates are usually operated manually.

2.2 Sluices

Sluices are structures provided with gated opening to permit passage of water in one direction only when the difference in water level permit. These type of structures are provided with flap gates for automatic operation and are mostly used in coastal region to prevent saline water intrusion and to allow drainage at low tide.

2.3 Weirs

Weirs are structures usually located in open channel and are normally used for the following purposes:

- Raising the upstream water levels
- Controlling the flow in the channel
- Measuring discharge in the channel

The weirs can be given a variety of shapes, each suitable for a specific purpose. The three basic types (based on the profile) are : i) broad crested; ii) sharp crested and iii) ogee-crested weirs. The flow characteristics and relationship between upstream and downstream of the broad crested weir are described below. For flow characteristics and water level relationship for other two types, the reader should refer to standard text books.

Broad Crested Weir (1)

A weir is called broad crested, if the crest has a breadth (B) long enough in the direction of flow to maintain essentially parallel stream lines. The significance of this is that it ensures hydrostatic pressure distribution in a cross section over the weir. This condition could be achieved when $0.05 < y_1 / B < 0.50$ for rounded leading edge and when $0.08 < y_1 / B < 0.33$ for sharp leading edge. The longitudinal profile of a broad crested weir with a rounded leading edge is shown in figure 1.

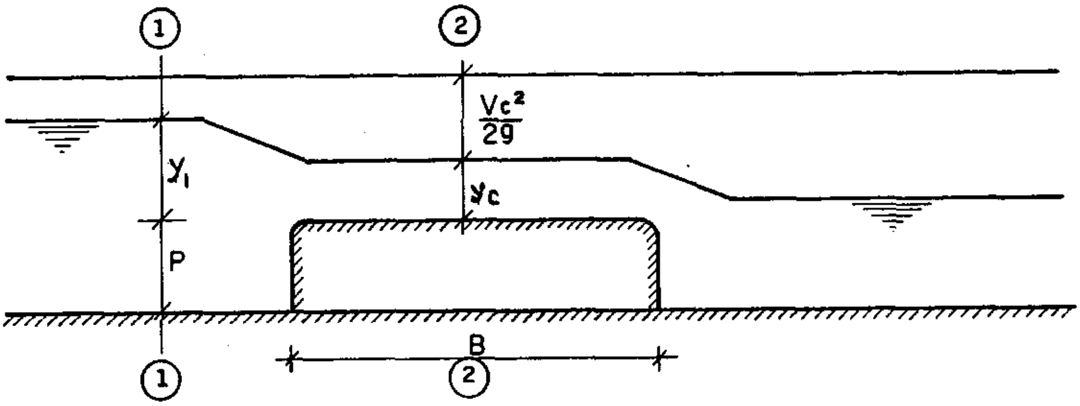


Fig. 1 : Definition sketch for a broad crested weir.

The discharge formula for the broad crested weir, neglecting upstream velocity head and friction loss, can be derived from figure 1 as follows: Applying Bernoulli's energy equation between Sections 1-1 and 2-2,

$$\frac{V_1^2}{2g} + Y_1 = \frac{V_c^2}{2g} + y_c = E_c \qquad .....(1)$$

For a deep, relatively slow moving flow in the upstream section,

$$\frac{V_1^2}{2g} << y_1 \qquad \text{hence} \qquad y_1 = E_c$$

Again using definition of specific energy,

$$E_c = \frac{3}{2} y_c \qquad \text{and} \qquad y_1 = \frac{3}{2} y_c$$

Therefore $y_c = \frac{2}{3} y_1$, again $y_c = (q^2/g)^{1/3}$

or $(y_c)^3 = (\frac{2}{3} y_1)^3 = q^2/g$

or $q^2/g = (2/3)^3 \; y_1^3$

therefore $q = (g)^{(1/2)} \; (2/3)^{(3/2)} \; (y_1)^{(3/2)}$

Now $Q = b.q$

$= b \; (g)^{(1/2)} \; (2/3)^{(3/2)} \; y_1^{(3/2)}$

for $g = 9.8 \text{ m/sec}^2$

$$Q = 1.70 \; b \; y_1^{(3/2)} \quad \ldots\ldots \qquad (2)$$

$$q = 1.70 \; y_1^{(3/2)}$$

The above equation is theoretical because it assumes no friction along the crest. Thus to allow for friction, figures 2 and 3 can be used to obtain the discharge co-efficient "C" for various ratios of y_1 / B. Equation (2) is valid for free flow condition. If the tail water level rises to the point where the downstream head becomes 0.6 times the upstream head, the flow becomes submerged. The discharge co-efficient for the submerged flow condition in sharp leading edge can be determined using figure 3.

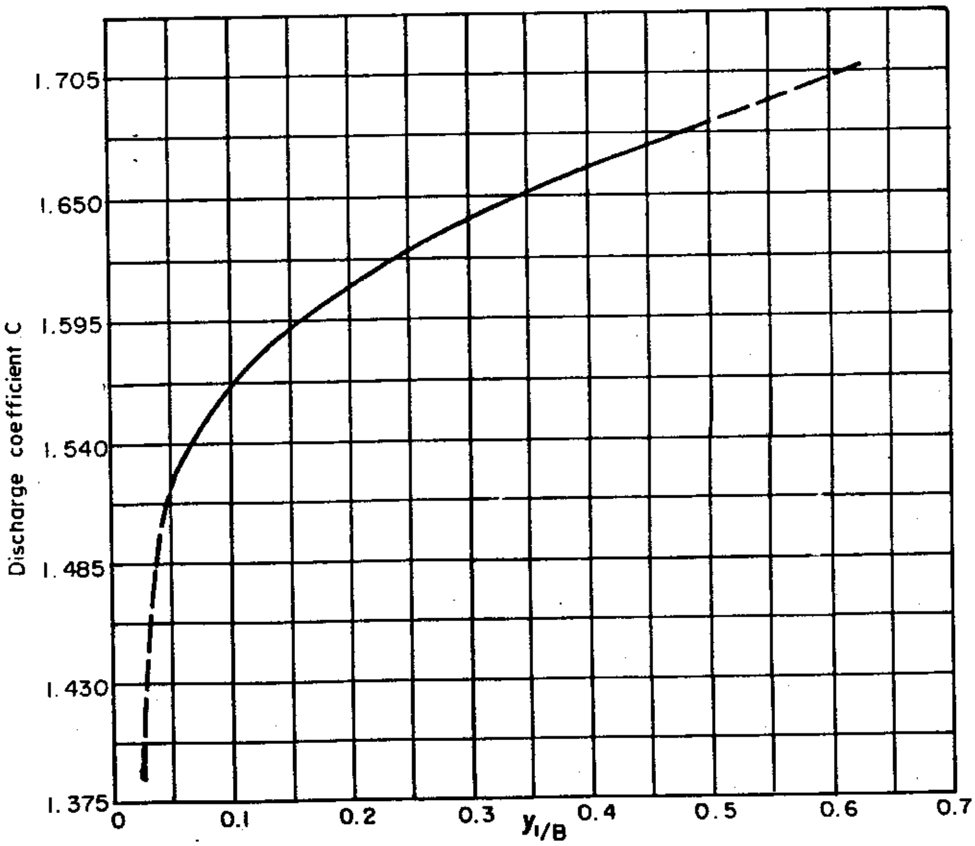
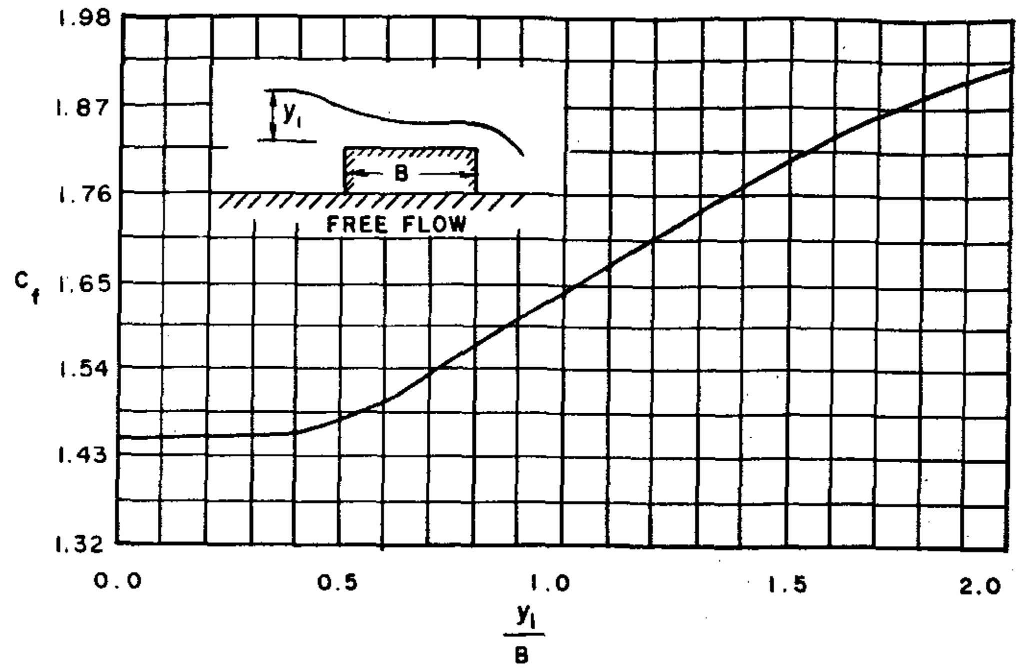
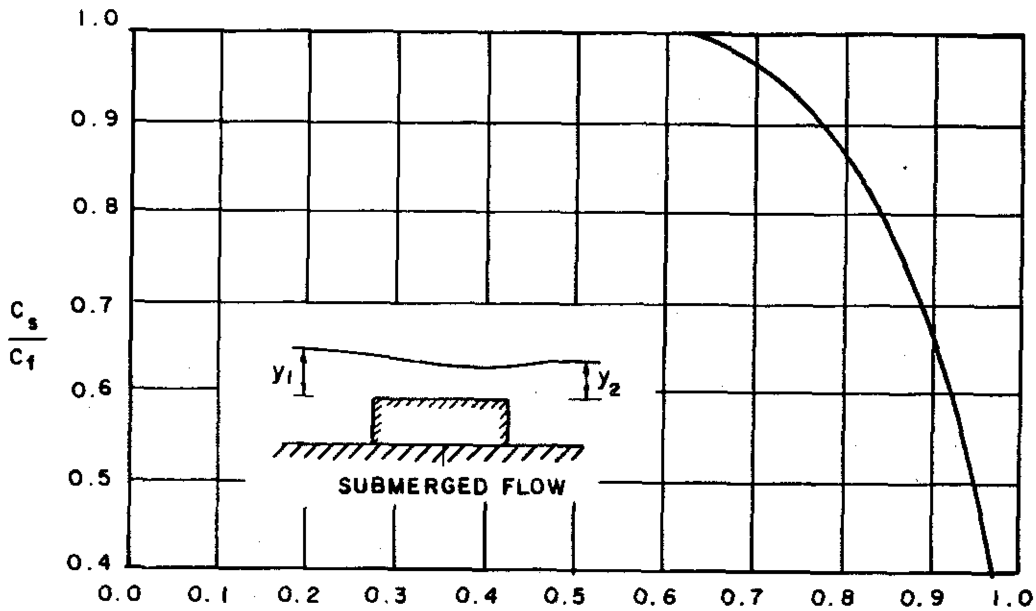


Fig. 2. Broad-Crested Weir Discharge co-efficient for rounded leading edge





C_s = submerged flow coefficient

C_f = free flow coefficient

Source: Engineering Manual 1110-2-1602, U.S. Army Corps of Engineers, 1963, p.III-15

Fig. 3 : Broad-Crested Weir Discharge co-efficient for Sharp Leading Edge

2.4 Water Retention Structure

Water retention structures are designed and constructed on perennial streams to retain water for dry season irrigation and to check over drainage of beel areas. They are submersible low height structure (masonry or reinforced cement concrete) and are designed either with vertical side walls or sloping side walls depending on the economy and stability of the structure. They are mostly built on small perennial hill streams to head up water for gravity irrigation or by Low Lift Pumps (LLPs) to irrigate the adjoining areas. The vertical side walls are more stable against any possible erosion due to overflanking of the structure during flash floods. A typical section of a water retention structure is shown in figure 4.

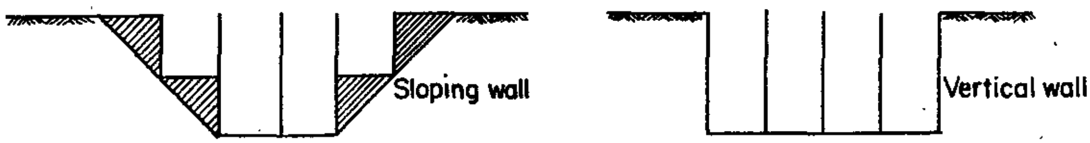


Fig. 4 : Typical Water Retention Structure

3.0 DESIGN CONSIDERATIONS FOR HYDRAULIC STRUCTURES

The considerations generally adopted in the hydraulic design of regulators, sluices, weirs and water retention structures are as follows:

- Design Flood Frequency
- Invert level Fixation
- Design Head
- Scour Depth and Cutoff Wall
- Stilling Basin Design
- Exit Gradient and Floor Length
- Uplift Pressure and Floor Thickness
- Loose Protective Works

3.1 Flood Frequency (2)

The 1 in 10 year 5 day pre-monsoon rainfall is normally used in determining the size of the structure for which the area inundated to a depth greater than 300 millimetres for a period of 3 days should be less than 5 percent of the drainage area. To ensure safety of the structure the riverside stilling basin will be designed for 1 in 25 year 5 day pre-monsoon sto[illegible] water level in the outfall channel. But if the [illegible]sin has the same hydrological characteri[illegible]y may be used. Post-monsoon routing using 1 in [illegible] rainfall within the drainage basin area and 1 in 10 year water level in the outfall channel is carried out to check the adequacy of the regul[illegible] post-monsoon drainage. The difference in water level betw[illegible] and riverside across the regulator during post-mons[illegible] [illegible]ally limited to 225 millimetres for a prolonged period and [illegible] millimetres not exceeding 3 days. Where flushing is a requirement, country side apron will be designed as a stilling basin using 80% dry year rainfall within the basin and a in 5 year river level in the out[illegible] channel.

3.2 Invert level Fixation

The following considerations are generally adopted in fixing the invert level for the regulators and sluices:

- Drainage requirements
- Hydraulic considerations
- Bed level of drainage channel.
- Riverside low water level (tail water level)

Drainage Requirement

The invert level for a drainage regulator is guided by the lowest basin level to be drained. From the lowest drainage level, an acceptable energy gradient (depending on topography) should be drawn to the structure point to fix the invert level. For tidal sluices, this level should be as low as the minimum polder level in the dry season.

Hydraulic Considerations

The discharge through a regulator or a sluice increases if the invert level is lowered. If the regulator/sluice is designed for flushing, a box with lower invert will give higher discharges and flow velocities since maximum head difference at inlet condition might be higher than at drainage condition. But lowering the invert will result in higher foundation cost requiring high wing walls and expensive dewatering system.

Bed level of drainage channel

The third criteria for fixing the invert level is the consideration to the existing bed level or the proposed re-excavation level of the drainage channel. For a flushing regulator, the invert level should correspond with the bed level. If the invert level is higher than the bed level, flushing will be restricted. In case of drainage regulator, the downstream basin level should be fixed at the natural bed level to avoid scour or siltation.

Riverside low water level

The fourth criteria to decide the sluice invert is the riverside low water level (tail water level). During pre-monsoon drainage, the sequent depth is required to match the tail water level to avoid sweeping out of the jump. Therefore the invert level and the downstream stilling basin is adjusted so that the depth of flow after the jump matches the river water level.

3.3 Design Head

Estimation of design head is very important in calculating the uplift pressure and floor thickness downstream. The criteria generally adopted to calculate design head for hydraulic structure are as follows:

Regulators

Two conditions are generally considered:

- Pre-monsoon
- Monsoon

In pre-monsoon, the design head is the difference between the maximum countryside water level attained in pre-monsoon routing and minimum riverside water level. In monsoon, the design head is the difference between 1 in 10 year river water level and countryside storage level using 1 in 10 year rainfall.

Tidal Sluices

In determining the design head for tidal sluices, two conditions are generally considered:

- Difference between riverside high tide level and countryside minimum water level.
- Difference between countryside retention level and riverside low tide level.

Water Retention Structure

For water retention structure, the design head is the difference between upstream water level and downstream minimum water level (dry bed).

3.4 Scour Depth and Cutoff Wall

3.4.1 Silt Factor

Scour depth is calculated from Lacey's formula. The most important factor in the use of Lacey's formula is the determination of the correct value for the silt factor "f" which depends upon rugosity of channel bed and silt grade. According to Lacey, f is proportional to V_o^2/R.

where,

V_o = regime velocity

R = hydraulic mean depth

A rough qualitative formula for determining "f" for the predominant type of silt transported, is

$$f = 1.76 (d_m)^{\frac{1}{2}} \qquad .....(3)$$

where d_m is the mean diameter of silt in millimeter corresponding to a maximum size of silt from 0.01 millimeter to 0.257 millimeter (3).

3.4.2 Scour Depth (4)

The depth of scour is determined by Lacey's regime formula

$$R = 1.35 \, (q^2/f)^{1/3} \qquad ...(4)$$

where R is the normal depth of scour below high water level in meter and q is the discharge per unit width in m^3/sec. The total depth of scour below high water level (HWL) at the downstream and upstream of regulator and sluices, is taken as X times R where R is Lacey's normal scour depth and the values of X (safety factor) for different classes of scour are shown in Table 1.

Table 1. Lacey's safety factor against scour depth.

Scour Type	Reach	Mean value of X	D = XR - Water depth (y)
A	Straight	1.25	1.25R-y
B	Moderate bend	1.50	1.50R-y
C	Severe bend	1.75	1.75R-y
D	Bend	2.00	2.00R-y

A value of X is taken as 1.25 on upstream and 1.50 on downstream side of the regulator. The depth of cut off wall for regulators and drainage sluices, is calculated from scour depth formula and then checked against exit gradient consideration. If the exit gradient criteria is not satisfied, adjustment is made between the length of floor and depth of cutoff wall ~~(see section 3.6)~~.

3.5 Energy Dissipation and Stilling Basin Design

Before designing the stilling basin, the type of flow through the structure are to be ascertained:

3.5.1 Flow Conditions (5)

The flow through a hydraulic structure, e.g., a culvert, a regulator or a sluice, may be classified into two general types depending upon the head water and tail water conditions. But for practical purposes, the two general types are broken down to six specific types of flow as indicated in figure 5.

Flow Type	Flow Conditions
1	Inlet and Outlet submerged $h_1 > d$ $h_2 > d$ Full flow (outlet control)
2	Inlet submerged Outlet not submerged $h_1 > 1.5d$, $h_2 < d$ Vent hydraulically long $(L/d > 20)$ Full flow
3	Inlet submerged Outlet not submerged $h_1 > 1.5d$, $h_2 < d$ Vent hydraulically short Partly full (inlet control)
4	Inlet not submerged Outlet not submerged $h_1 < 1.5d$, $h_2 > h_{cr}$ Subcritical flow
5	Outlet not submerged $h_1 < 1.5d$, $h_2 < h_{cr}$ (outlet control)
6	Outlet not submerged $h_1 < 1.5d$ $h_2 < h_{cr}$ Inlet control

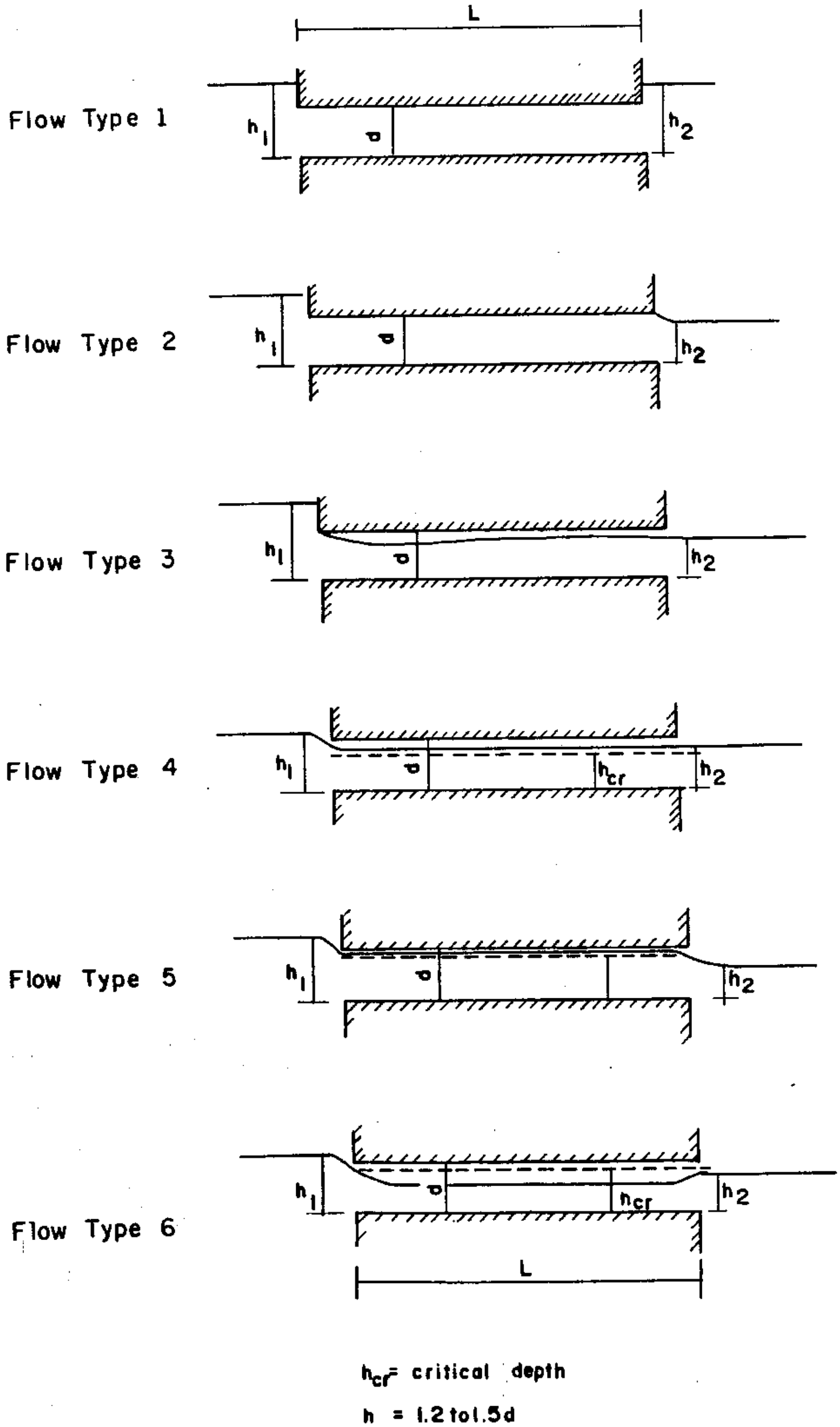


Fig. 5. Types of flow through hydraulic structure (5)

The discharge for each type of flow can be derived from the basic orifice and weir formula:

$$Q = C.A.\ (2g.\ H)^{\frac{1}{2}} \qquad \text{.... (5)}$$

in which Q = discharge in m^3/sec

C = discharge coefficient

A = cross sectional area in square meter

g = gravity acceleration (9.81 m/sec^2)

H = energy head loss in meter

The discharge formula for six flow conditions are summarised in table 2. below:

Table 2. Discharge formula

Flow Condition	Discharge formula	Remarks
1 and 2	$Q = C_1\ A\ \sqrt{(2g\ \Delta\ H)}$	$C_1 = \left[\dfrac{1}{1.5 + 2.L.n^2.g/R^{4/3}}\right]^{\frac{1}{2}}$ (for square cornered entrance & outlets)
3	$Q = C_3\ A_b\ \sqrt{(2g.\ H_1)}$	Value of C_3 can be obtained from figure 5a
4	$Q = C_4.b.h_2.\sqrt{[2g(h_1-h_2)]}$	Only if $h_2 > h_{cr}$ $h_{cr} = (q^2\ /g)^{1/3}$ C_4 = 0.82 for square entrances
5 and 6	$Q = C\ b\ h_1{}^{3/2}$	for value of C refer table 3.

where,

b = width of barrel in meter

h_1 = headwater depth above invert level in meter

H_1 = energy head of headwater above invert level in meter

L = length of barrel in meter

n = Manning's roughness coefficient

g = gravity acceleration (9.81 m/s^2)

R = hydraulic radius in meter

A = cross sectional area of flow (m^2)

A_b = cross sectional area of barrel (m^2)

q = discharge per unit width in m^3/s

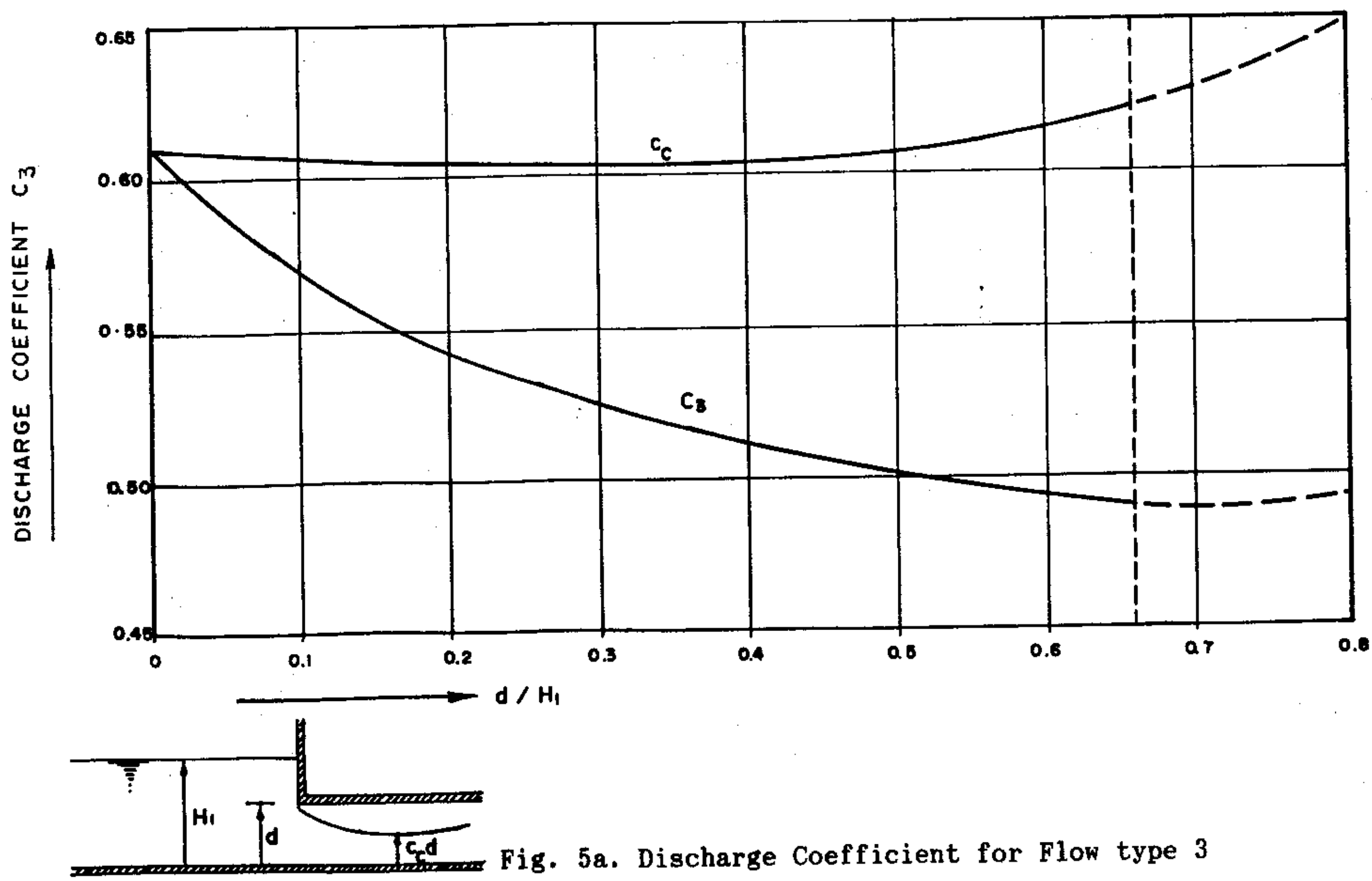


Fig. 5a. Discharge Coefficient for Flow type 3

Table 3 : Discharge co-efficient flow condition 5

Entrance type	Co-efficient
warped transition	1.62
cylindrical quadrant	1.58
simplified straight line	1.56
straight line transition	1.48
square ended transition	1.35

3.5.2 Types of Stilling Basin (6,7)

Stilling basin may be defined as a structure in which the energy of the high velocity flow jet is dissipated. If the phenomenon of hydraulic jump is used for dissipating the energy, the basin is called the hydraulic jump type of stilling basin. In this section ,the design of hydraulic jump type basin is described.

Hydraulic jump is the most effective way of energy dissipation. When a stream of water moving with a high velocity and low depth (supercritical flow), strikes another stream of water moving with a low velocity and high depth (subcritical flow), a sudden rise in the surface of former takes place. This phenomenon is called the hydraulic jump which creates a large scale turbulence dissipating most of the kinetic energy of the supercritical flow. The energy dissipation in the jump depends upon the froude number (F_1) of the incoming flow. Different types of jumps have been described for different values of Froude numbers. If the Froude number of the incoming flow is higher, the greater will be the energy dissipation. An approximate percentage loss of energy for various values of F_1 are given in table 4.

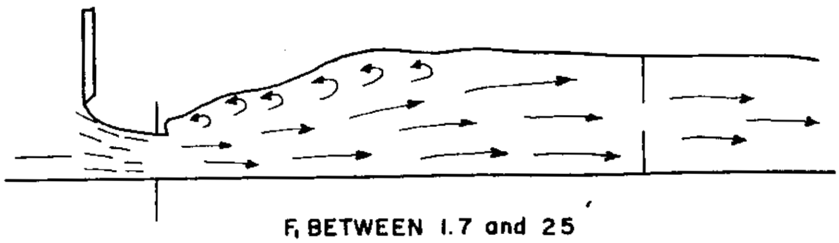
Table 4 : Loss of energy for various values of F_1

F_1	per cent loss of energy
2.5	17
4.5	45
9.0	70
14.0	80
20.0	85

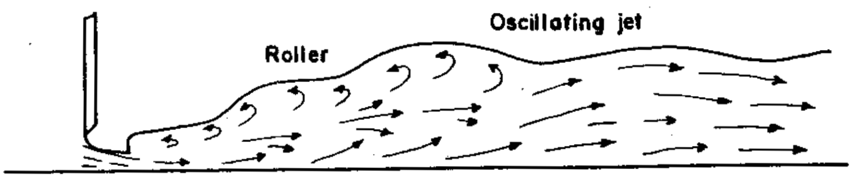
The hydraulic jump basins need to be designed in such a way that the energy of exit velocity is to be dissipated to a tranquil state before the flow is returned to downstream channel bed. USBR Stilling basin designs suitable to provide stilling action for various forms of jump are described in the following paragraphs and the characteristic forms of jumps related to Froude numbers are shown in figure 6.

$F_1 < 1.7$

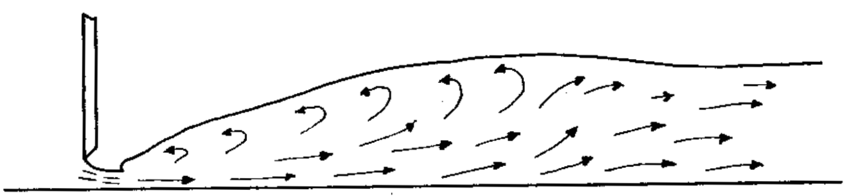
The conjugate depth y_2 is about twice the incoming depth or about 40 percent greater than the critical depth. The exit velocity v_2 is about one-half the incoming velocity or 30 percent less than the critical velocity. No special stilling basin is required to still flows except that the channel lengths beyond the point where the depth starts to change,



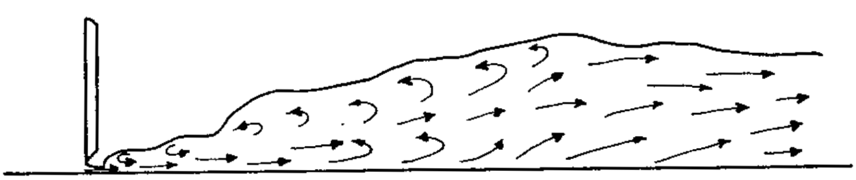
F_1 BETWEEN 1.7 and 2.5
FORM A-PREJUMP STAGE



F_1 BETWEEN 2.5 and 4.5
FORM B-TRANSITION STAGE



F_1 BETWEEN 4.5 and 9.0
FORM C-RANGE OF WELL-BALANCED JUMPS



F_1 GREATER THAN 9.0
FORM D-EFFECTIVE JUMP BUT ROUGH
SURFACE DOWN STREAM

shall not be less than $4y_2$. No baffles or other dissipating devices are needed. This basin is designated as type I.

<u>F_1 between 1.7 to 2.5</u>

This is a prejump stage where flows are not attended by active turbulence and therefore baffles or sills are not required. The basin should be sufficiently long to contain the flow prism. Conjugate depths and basin lengths shown in figure 7 (properties of jump in relation to froude number) will provide acceptable basins (7). These basins are also designated as type I.

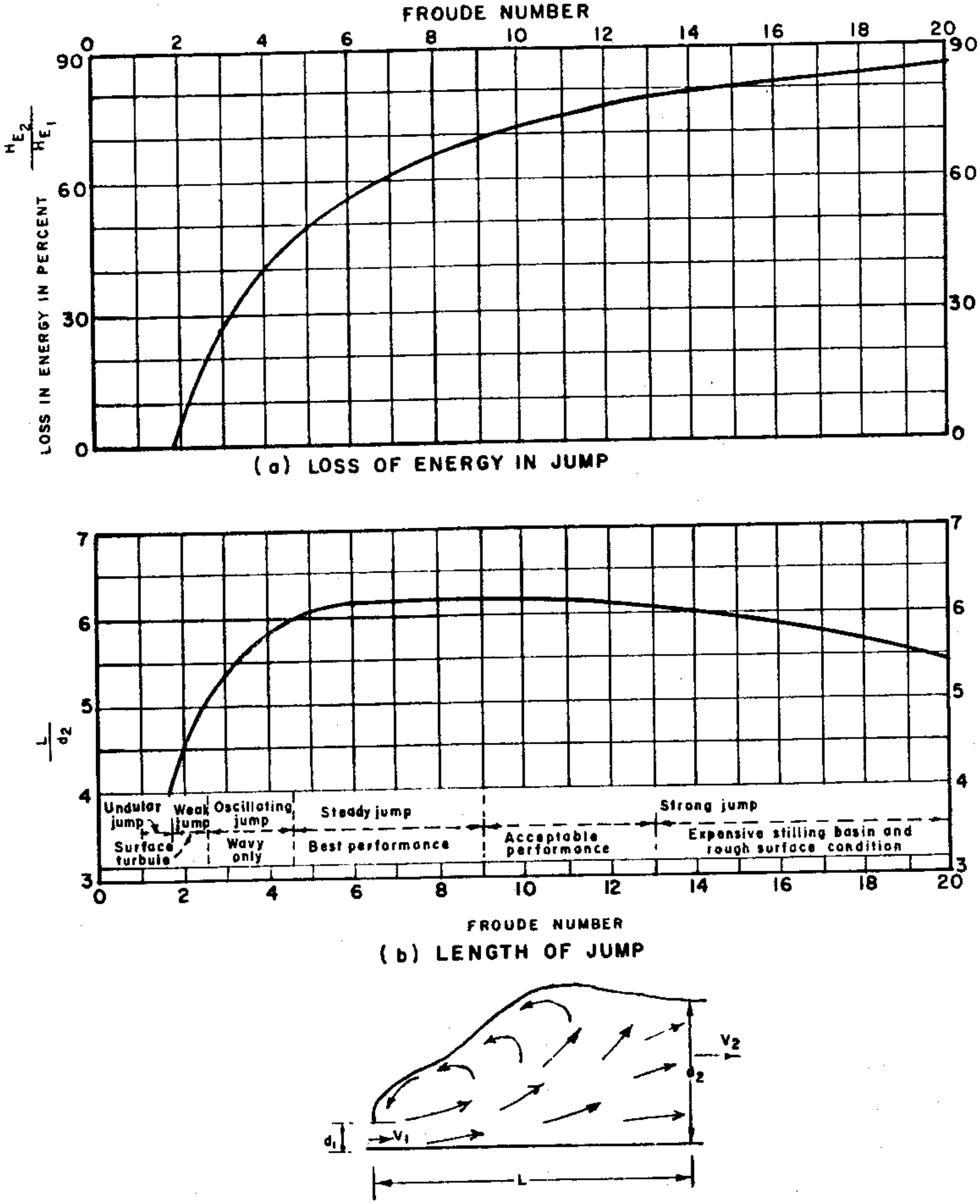
<u>F_1 between 2.5 to 4.5</u>

This is a transitional flow stage because a true hydraulic jump does not fully develop. The jump is troublesome and oscillating giving rise to heavy surface waves which persist beyond the end of the basin. Stilling device must be provided to dissipate flows for this range of froude number. The basin shown in figure 8, designated as type IV, has proved to be effective for dissipating the bulk of the energy of flow. Because of the tendency of the jump to sweep out and as an aid in suppressing wave action, the water depth in the basin should be about 10 percent greater than the computed conjugate depths. Design of this type of basin can be avoided by selecting stilling basin dimensions that will provide flow conditions outside the range of transition flow. The froude number can be

raised by widening the basin but it depends on the economy as well as hydraulic performance of the basin.

<u>F_1 higher than 4.5</u>

A true hydraulic jump will form in this range with the best jump performance. The elements of the jump will vary according to froude number as shown in figure 9. The installation of accessory devices such blocks, baffles and sills along the floor of the basin produce a stabilizing effect on the jump, which permits shortening of the basin and provides a safety factor against sweep out due to inadequate tailwater depth. The basin shown in figure 9 which is designated as type III, can be adopted where the incoming velocities do not exceed 18.3 m/sec (60 ft/sec). This type of basin uses chute blocks, impact baffle blocks, and end sill to shorten the jump length and to dissipate the high velocity flow within the shortened basin length. Where the incoming velocities exceed 18.3 m/sec (60 ft/sec), or where impact baffle blocks are not used, the type II basin (figure 10) may be adopted. Because the dissipation is primarily accomplished by hydraulic jump action, the basin length will be greater than type III. However, the chute blocks and dentated end sills will effectively reduce the basin length. Because of the reduced margin of safety against sweep out, the water depth in the basin should be about 5 percent greater than the computed conjugate depth.



Sources: "Design of Small Dams" by U.S. Department of the Interior.

"Research Studies on Stilling Basins, Energy Dessipators and Associated Appurtenances," Hydraulic Lab. Report No. Hyd-399 by U.S. Department of the Interior.

Fig. 7. Properties of Jump in Relation to Froude Number (7)

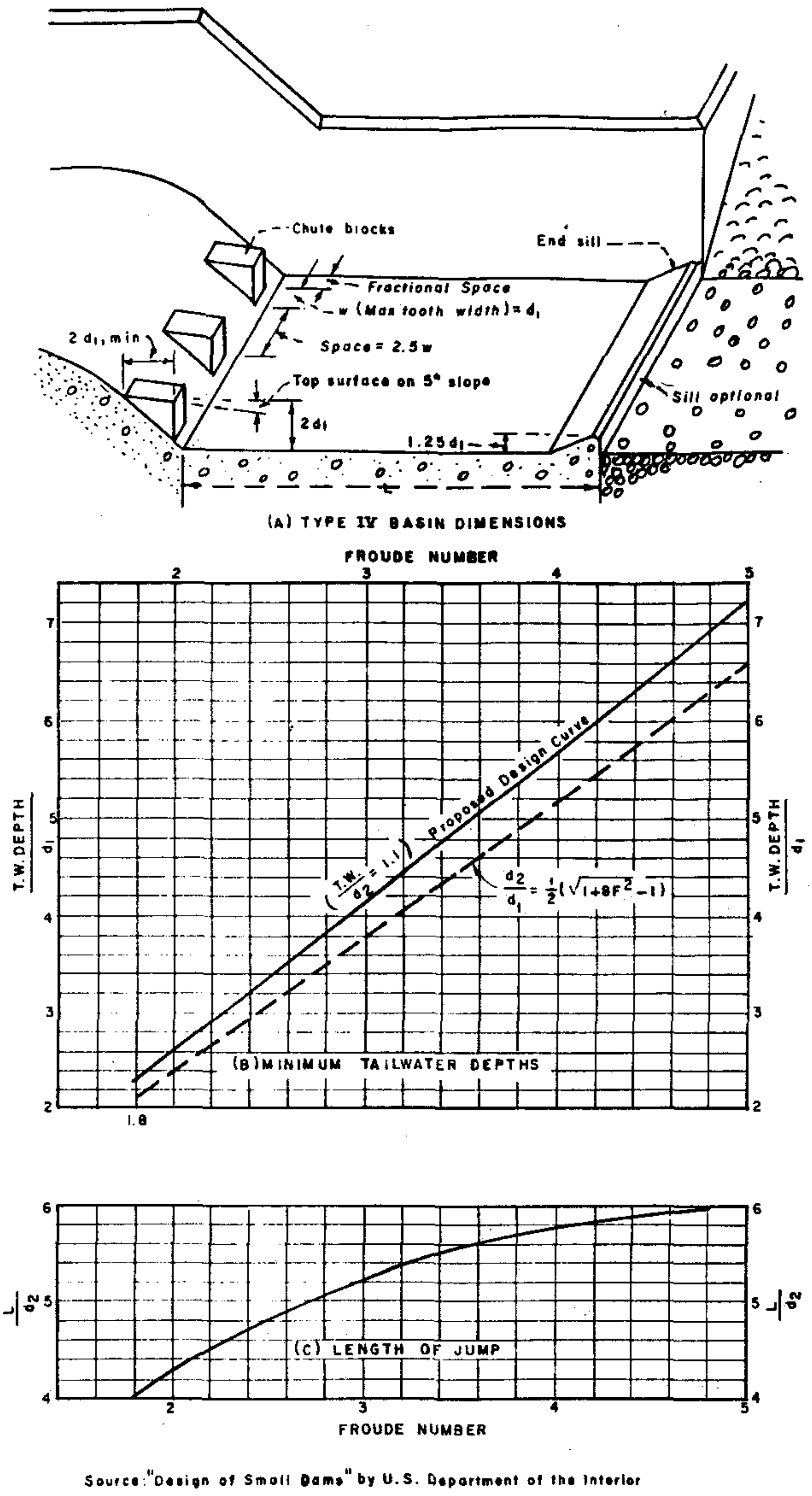
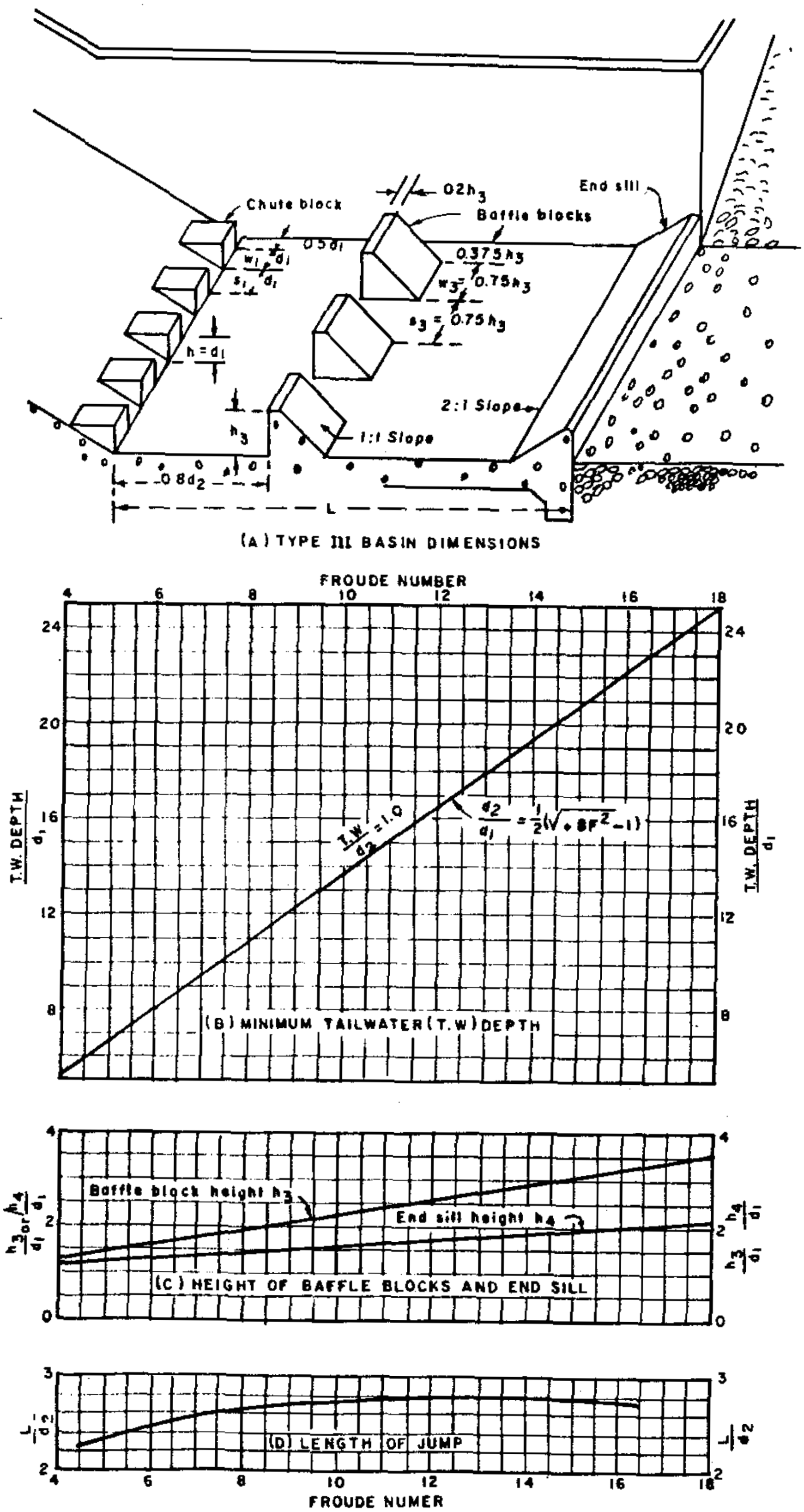


Fig. 8. USBR Stilling Basin Type IV



Source: "Design of Small Dams" by U.S. Department of the Interior

Fig. 9. USBR Stilling Basin Type III

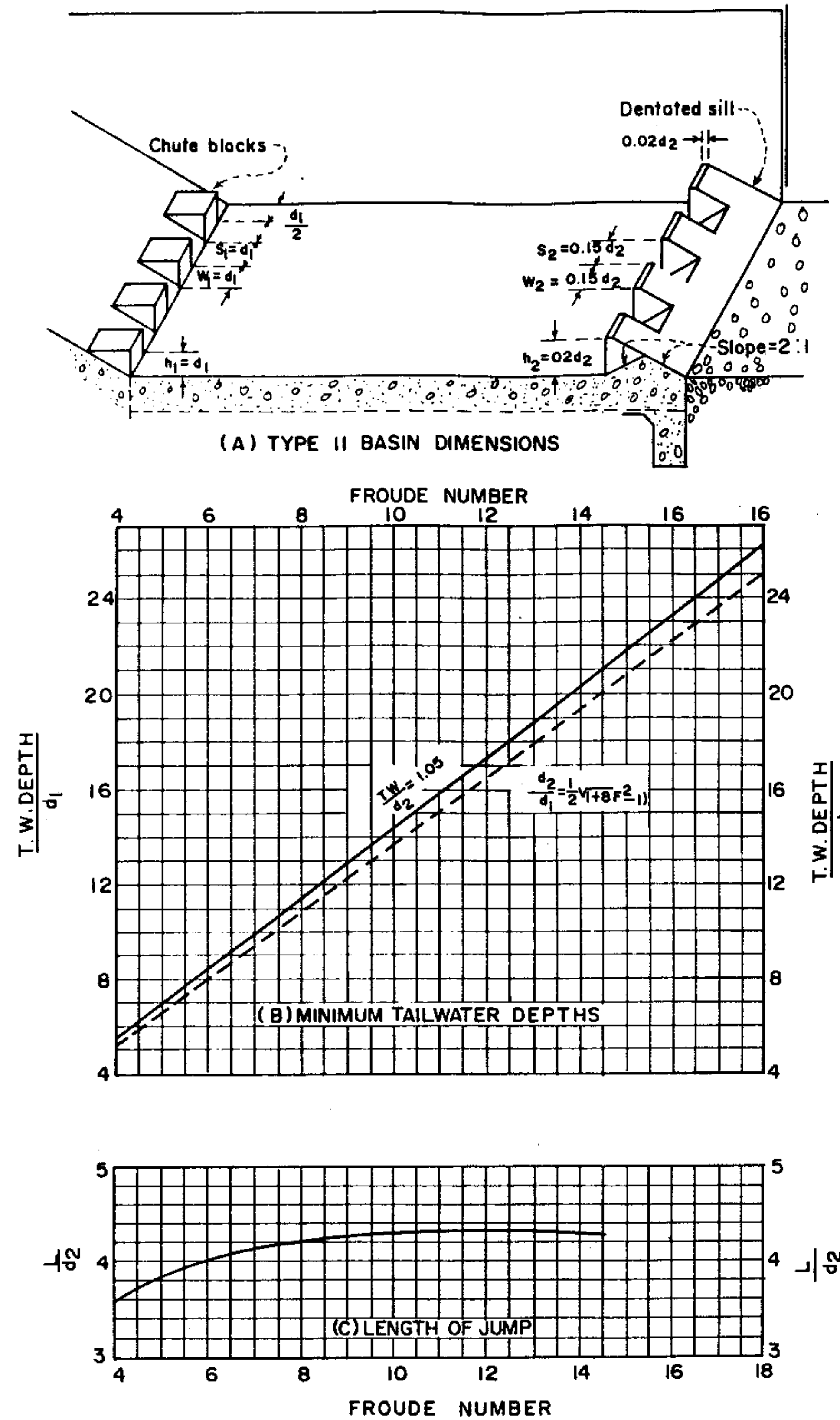


Fig. 10. USBR Stilling Basin Type II

Low Froude Number Stilling Basin

Type IV basins are fairly effective at low froude number flows for small canals and for structures with small unit discharges. However, model tests have developed designs quite different from type IV basin. It should be noted that a hydraulic jump stilling basin is not an efficient energy dissipator at low froude number; the efficiency of a hydraulic jump basin is less than 50 percent in this range of froude number (2.5 to 5.0). Alternative energy dissipators, such as the baffled apron, chute or spillway, should be considered for these conditions. The recommended design has chute blocks, baffle piers, and a dentated end sill. All design information are presented in figure 11.

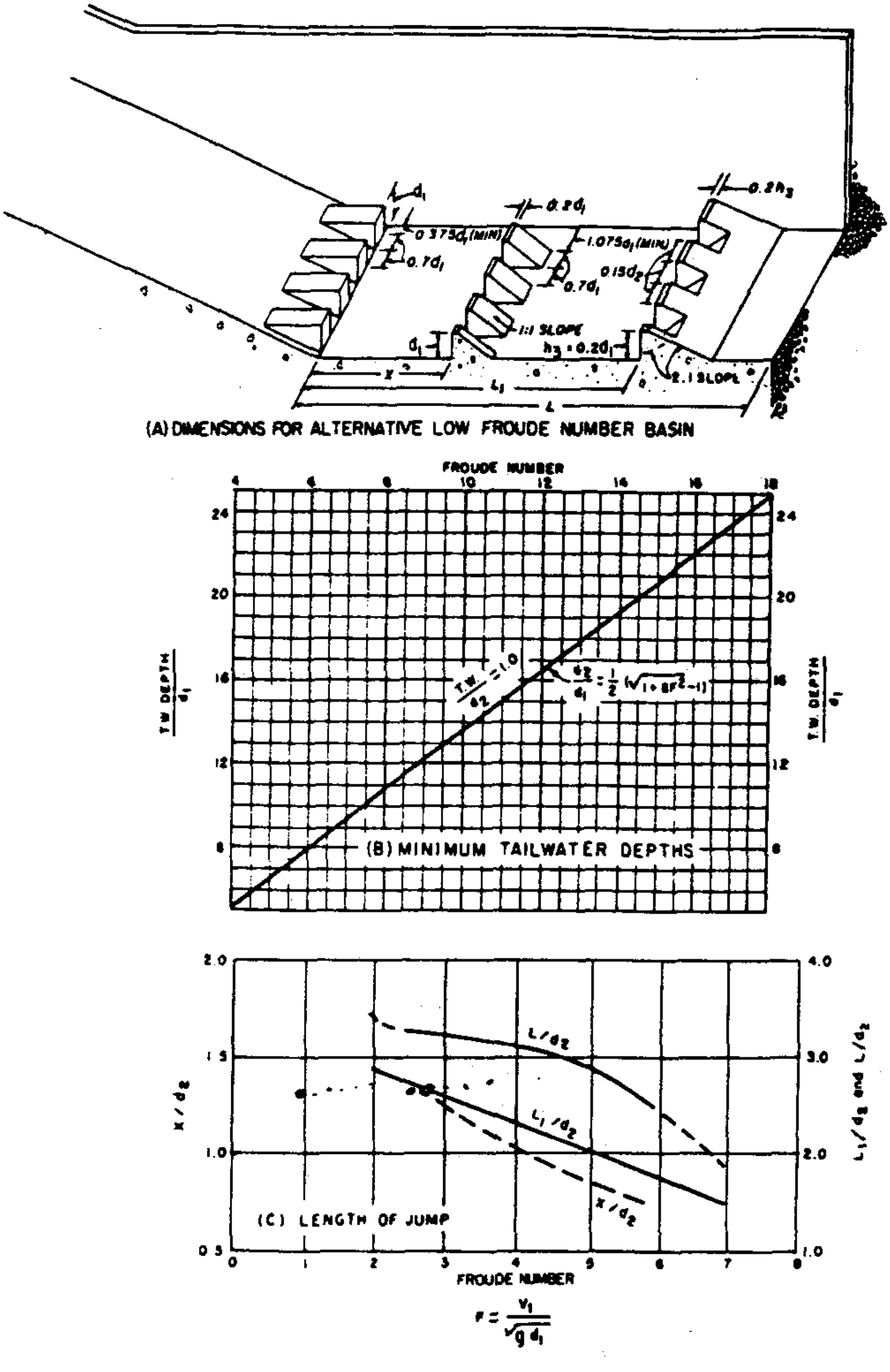
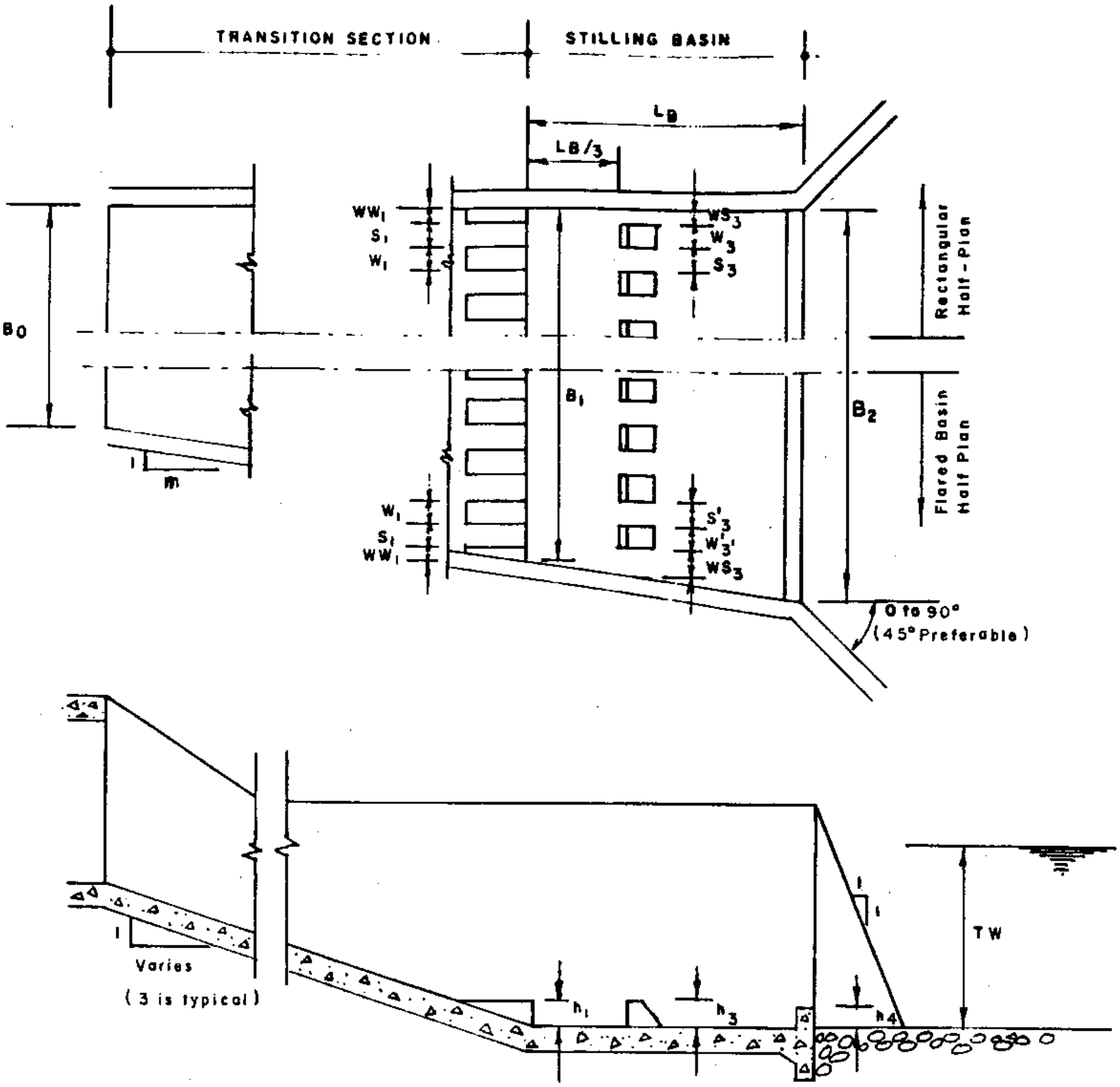


Fig. 11. Low Froude Number stilling Basin

The length of the basin is rather short, approximately three times d_2 (the conjugate depth after the jump). The size and spacing of the chute blocks and baffle piers are functions of d_1 (incoming depth) and froude number. The end sill is placed at or near the downstream end of the stilling basin. A tail water depth of d_2 maintained the jump at the intersection of the horizontal apron and the chute. However, sweep out did not occur for the recommended design when tail water was 0.8 of d_2. The tail water should be maintained at or slightly higher than d_2 (about 5 percent). Additional depth increases the safety factor against sweep out and decreases the flow velocity.

SAF (Saint Anthony Falls) Stilling Basin.

The SAF stilling basin (figure 12) was developed at the Saint Anthony Falls hydraulics laboratory for canal and diversion structures on agricultural distribution systems and small streams having low heads and small unit discharges. SAF basin is so short that a significant amount of energy dissipation must occur downstream from the end sill. The downstream channel can be allowed to erode until a stable scour hole occurs or suitable protection can be provided downstream from the stilling basin to minimize scour. The design parameters for this type of basin are as follows:



Basin Floor: Upstream width, $B_1 = K\sqrt{Q}$ ($K = 0.6$ is commonly used)

Wall flare rate, $m = 3F_o$

Basin length, $L_B = 4.5\, d_2 / F_1^{0.76}$

Tailwater depth, $TW = (1.1 - F_1^2/120)\, d_2$ for $1.7 < F_1 < 5.5$

$TW = 0.85\, d_2$ for $5.5 < F_1 < 11$

$TW = (1 - F_1^2/800)\, d_2$ for $11 < F_1 < 17$

Chute Blocks: $h_1 = d_1$ $\quad w_1 = s_1 = 0.75\, d_1$ $\quad ww_1 > 0.375\, d_1$

Baffle Blocks: $h_3 = d_1$ $\quad w_3 = s_3 = 0.75\, d_1$ $\quad ws_3 > 0.375\, d_1$

$$w_3' = s_3' = w_3\left(1 + \frac{2m}{3B_1}\right)$$

End sill: Height, $h_4 = 0.07\, d_2$

Fig. 12. Saint Anthony Falls Basins

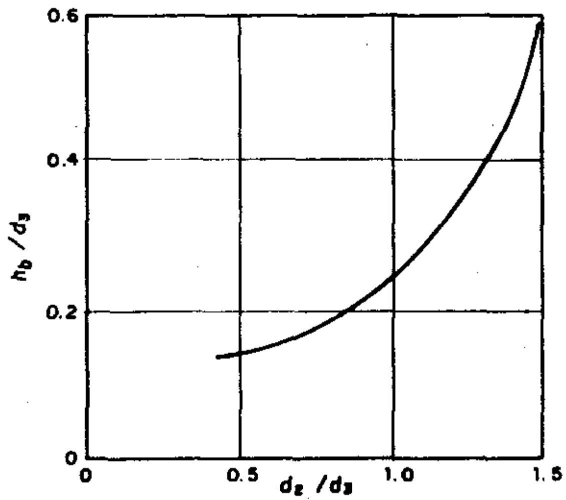
- Basin can be designed for froude number between 1.7 and 17.0
- Length of stilling basin (L) = 4.5 y_2 / $F_1^{0.76}$
- Height of chute blocks and floor blocks is y_1 (pre-jump depth) and the width and spacing are approximately 0.75 y_1
- The distance from the toe of glacis to floor blocks is L/3. The floor blocks are placed staggered with chute blocks and clear space between side wall and floor block is $3y_1/8$.

Indian Standard Stilling Basin I

Definition sketch and dimension sketch for the Indian Standard Stilling basin I, is given in figure 13. This basin is suitable for froude number between 2 and 4.5. The length of the basin for different values of froude number is given below:

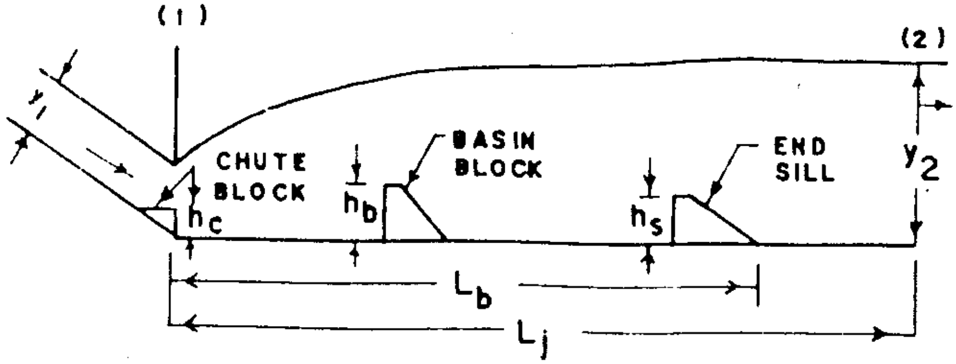
Where	**F_1**	**is**	2.0	3.0	4.0	4.5
Then	**L_b/Y_2**	**is**	3.15	4.3	4.75	5.0

The height of floor block h_b is determined from a relation h_b/d_3 and d_2/d_3 (figure 13a) where d_2 is the conjugate depth and d_3 is the tail water depth.



HEIGHT OF FLOOR BLOCKS IN RELATION TO d_2 DEPTH

Figure 13a



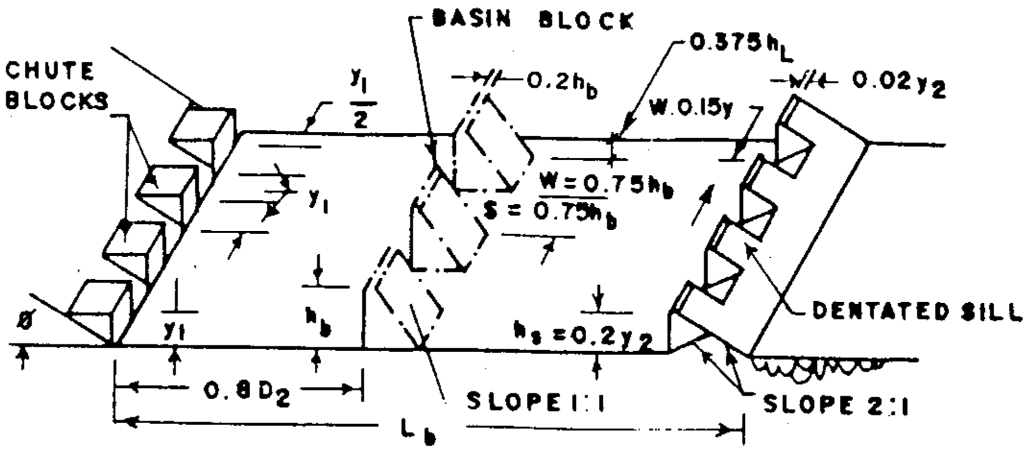


Fig. 13. Indian Standard Stilling Basin Type I

3.6 Exit Gradient and Floor Length (4)

According to Khosla's theory, for a standard form consisting of floor length b with a vertical cut off depth d (figure 14), the exit gradient at its downstream end is given by

$$G_E = \frac{H}{d} \times \frac{1}{\pi \sqrt{\lambda}} \qquad .....(6)$$

in which,

$$\lambda = \frac{1 + \sqrt{(1 + \alpha^2)}}{2}$$

$$\alpha = \frac{b}{d}$$

H = maximum seepage head

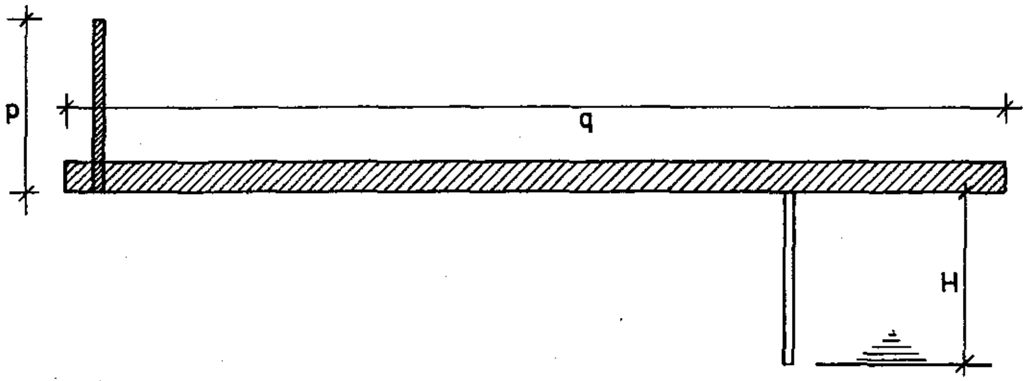


Fig. 14. Khosla's standard form

The exit gradient is critical when the upward disturbing force developed by difference in pressure head on the material grain at the exit is just equal to the submerged weight of the grain. For the structure to be safe against piping, an exit gradient equal to 1/6 to 1/7 of the critical exit gradient should be ensured. Value of Khosla's safe exit gradient for different type of material is shown in Table 5.

Table 5. Khosla's safe exit gradient

Type of material	Khosla's safe exit gradient
Shingle	0.25 to 0.20 (1/4 to 1/5)
Coarse sand	0.20 to 0.17 (1/5 to 1/6)
Fine sand	0.17 to 0.14 (1/6 to 1/7)

Bligh's Creep Theory

According to Bligh's theory, the percolating water follows the outline of the foundation base of the hydraulic structure. In other words, water creeps along the bottom contour of the structure. The length of the path thus traversed by water is called the length of the creep. Further it is assumed in this theory that the loss of head is proportional to the length of the creep. If H is the total head loss between the upstream and downstream, and L is the length of the creep, loss of head per unit of creep length (i.e. H/L) is called the hydraulic gradient. Bligh's theory made no distinction between horizontal and vertical creep.

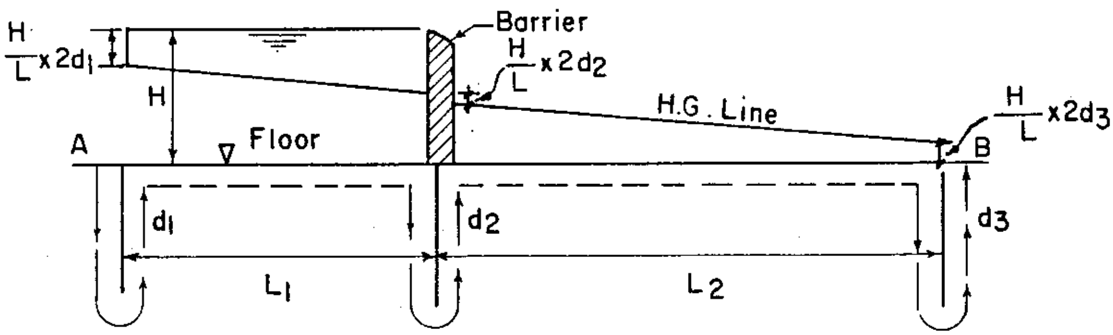


Fig. 15. Bligh's Creep theory.

In figure 15, H is the difference of water levels between upstream and downstream ends (downstream bed is dry). Water will seep along the bottom

contour of the profile as shown and the total length of creep (L) is

$$L = d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3$$
$$L = d_1 + L_1 + 2d_2 + L_2 + 2d_3$$
$$= (L_1 + L_2) + 2 (d_1 + d_2 + d_3)$$

Hydraulic gradient

$$= H/(b+2(d_1+d_2+d_3) = H/L$$

Head losses equal to:

$(H/L) \times 2d_1$, $(H/L) \times 2d_2$ and $(H/L) \times 2d_3$

will occur respectively in the planes of three vertical cut offs. The spacing between two adjacent cutoff shall be not less than twice the depth of cut off.

Safety against piping

Safety against piping can be ensured by providing creep length L = CH where C is Bligh's creep co-efficient for soil. Different values of C for different types of soil are given in table 6 below:

Table 6. Bligh's creep co-efficient

Soil type	Value of C	Safe Hydraulic gradient should be less than
Fine sand	15	1/15
Coarse sand	12	1/12
Sand mixed with boulder and gravel	5 to 9	1/5 to 1/9
Light sand & mud	8	1/8

Lane's Weighted Creep Theory

As stated above, Bligh made no distinction between horizontal and vertical creep but Lane, based on the study carried out on about 200 dams all over the world, stipulated that the horizontal creep is less effective in reducing uplift (or in causing loss of head) than that of the vertical creep. He, therefore, suggested a weightage factor of 1/3 for the horizontal creep as against 1.0 for the vertical creep. Referring to figure 15, the total Lane's creep length,

$$Le = (d_1 + d_1) + 1/3\ (L_1 + L_2) + (d_2 + d_2) + (d_3 + d_3)$$

$$Le = 1/3\ (L_1 + L_2) + 2\ (d_1 + d_2 + d_3)$$

$$Le = 1/3\ b + 2\ (d_1 + d_2 + d_3)$$

To ensure safety against piping, the creep length Le must not be less than C.H where H is the head causing flow and C is Lane's creep co-efficient given in table 7.

Table 7. Lane's creep co-efficient

Soil type	Value of Lane's co-efficient "C"	Safe Lane's Hyd. gradient should be less than
Very fine sand or silt	8.5	1/8.5
Fine sand	7.0	1/7
Coarse sand	5.0	1/5
Gravel and sand	3.5 to 3.0	1/3.5 to 1/3
Boulders, gravel	2.5 to 3.0	1/2.5 to 1/3
Clayey soils	3.0 to 1.6	1/3 to 1/1.6

3.7 Uplift pressure and floor thickness (4)

Uplift pressure under a structure needs to be checked for two conditions:

- Uplift pressure under steady seepage when the pond is full
- Uplift pressure in the jump trough

3.7.1 Pressure under steady seepage

The steady seepage flow below the foundation of a hydraulic structure, is precisely known by plotting flow net diagram. This can be solved either mathematically or graphical sketching by adjusting the stream lines and equipotential lines with respect to boundary condition. These are complicated approach and are time consuming. Khosla has given a simple, quick and accurate approach, called the Method of Independent Variables to design hydraulic structures on pervious foundation. In this method, complex profiles of a regulator or a weir is broken into a number of simple profiles. Khosla's simple profiles are shown in figure 16.

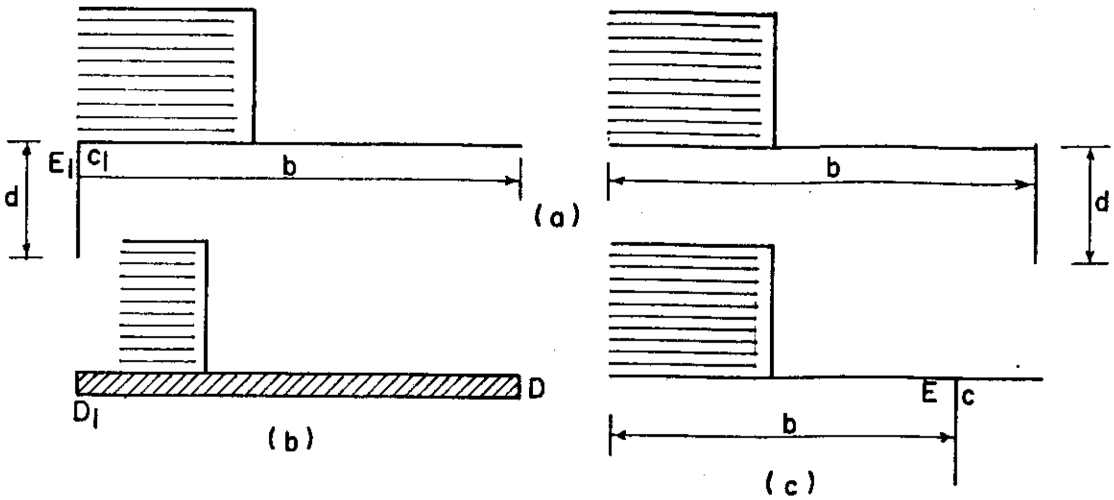


Fig. 16. Khosla's simple profiles

- A straight horizontal floor of negligible thickness with a sheet pile on the upstream end or downstream end.
- A straight horizontal floor depressed below the bed without any vertical cutoffs.
- A straight horizontal floor of negligible thickness with a sheetpile line at some intermediate points.

Mathematical solutions of flow nets for these profiles are presented in the form of pressure curves in figure 17 which can be used for determining percentage pressure at various key points under the structure. The key points are the junctions of the floor and pile lines on either side and the bottom points of the pile line and the bottom corners in the case of depressed floor.

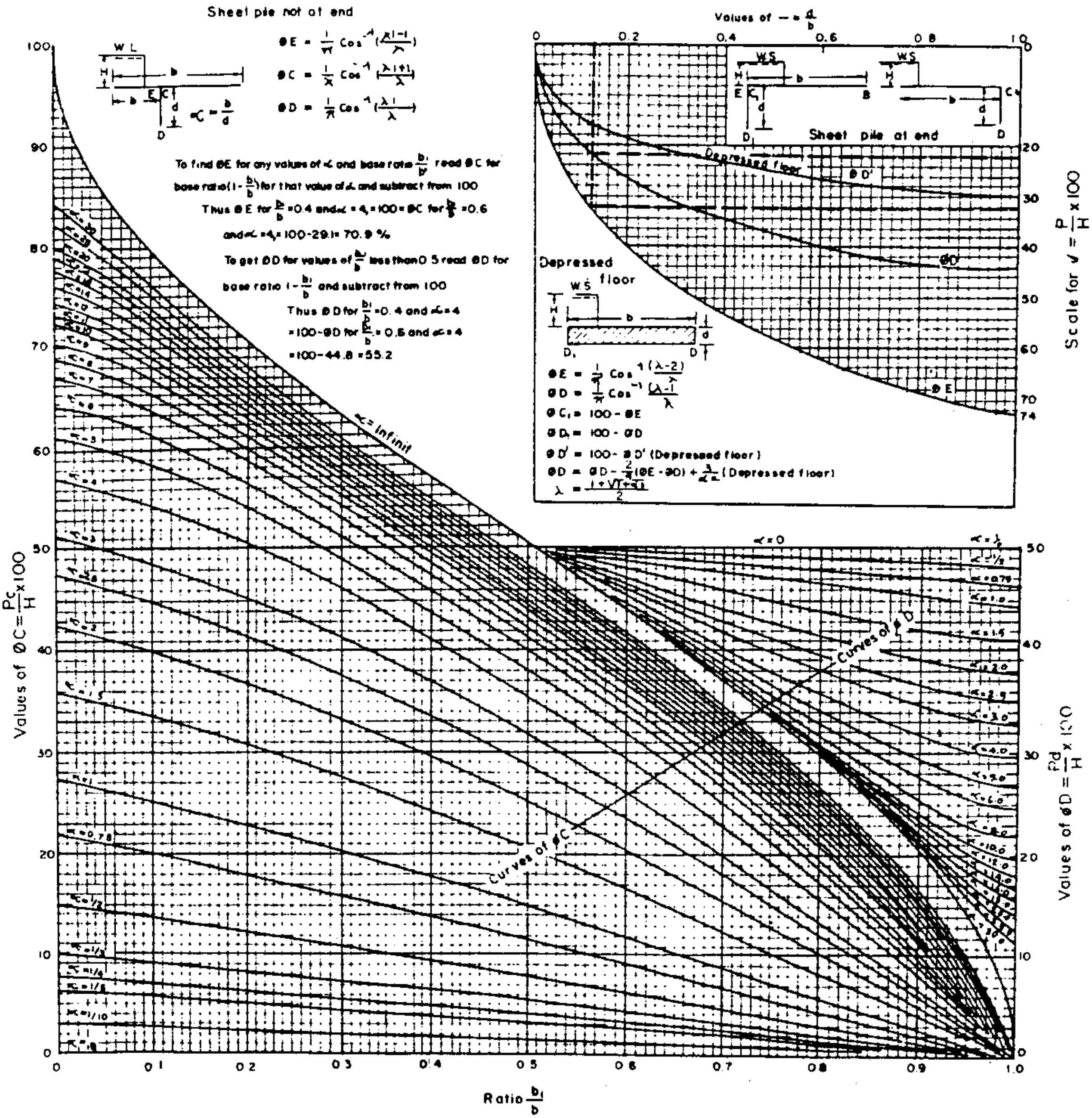


Fig. 17. Khosla's pressure curves

The percentage pressures at various key points are valid for the complex profiles if the pressures are corrected for:

- Mutual interference of pile
- Thickness of floor
- Slope of the floor

<u>Correction for mutual interference of piles</u>

$$C = 19 \ [D/b']^{1/2} \ [(d+D)/b] \qquad \ldots\ldots(7)$$

where,

C = correction in per cent

b' = distance between two piles in meter

D = depth of pile line in meter, the influence of which is to be determined on the neighbouring pile of depth d. D is to be measured below the level at which interference is desired.

d = depth of pile in meter on which effect is considered.

b = total length of the floor in meter.

This correction is positive for points in the rear or backwater and negative for the points forward in the direction of flow. This equation does not apply to the effect of an outer pile on an intermediate pile, if the intermediate pile is equal to or smaller than the outer pile and is located at a distance less than twice the length of the outer pile.

Correction for floor thickness

Khosla's standard profile assumed negligible thickness of the floor slab and percentage pressure calculated, shall pertain to top levels of the floor while the junction points E and C are at the bottom of the floor (figure 18).

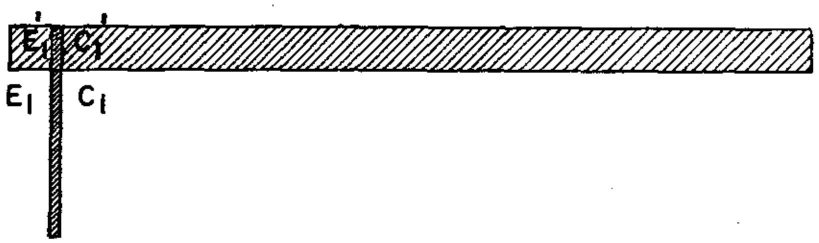


Fig. 18. Definition sketch for correction to floor thickness.

Since the corrected pressure at E_1 should be less than the calculated pressure E_1', the correction to be applied for the point E_1 shall be negative. Similarly pressure calculated at C'_1 is less than the corrected pressure at C_1' and hence, the correction to be applied at C_1 is positive. The pressures are found out by assuming linear pressure distribution.

Correction for slope of floor

A correction is applied for sloping floor and is taken as positive for the down and negative for the upslopes following the direction of flow. Values of correction for standard slopes are given in table 8.

Table 8. Slope correction factor

Hor:Vert	Correction factor
1:1	11.2
2:1	6.5
3:1	4.5
4:1	3.3
5:1	2.8

The correction factor given above is to be multiplied by the horizontal length of the slope and divided by distance between two pile lines between which the sloping floor is located. This correction is only applicable to the key points of the pile line, fixed at the beginning or the ends of the slope.

3.7.2 Uplift pressure in a jump trough (4)

The maximum uplift pressures are imposed on a structure when water is ponded upto the highest level on the upstream side without any discharge passing through the regulator. The hydraulic gradient line under such a situation is shown in figure 19.

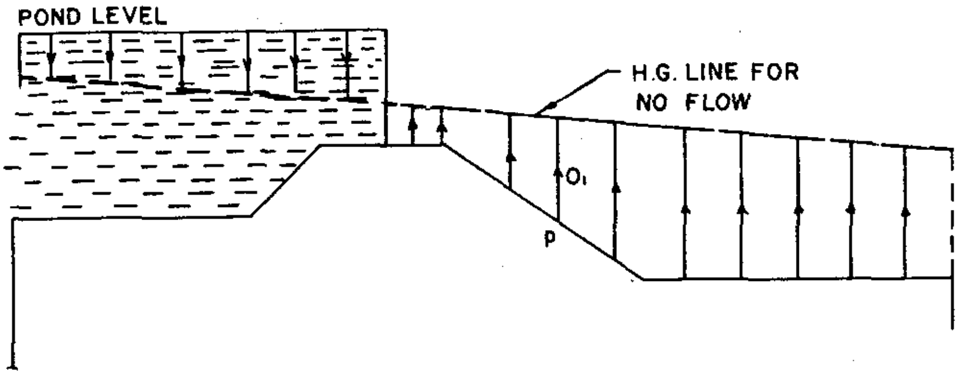


Fig. 19. Uplift pressure in jump trough for no flow condition.

When discharge is passing through the regulator with the formation of a hydraulic jump, seepage head is the difference in water level upstream and downstream, which is much larger than the seepage head in static condition. The hydraulic gradient line along with the water surface profile for this situation, is plotted in figure 20.

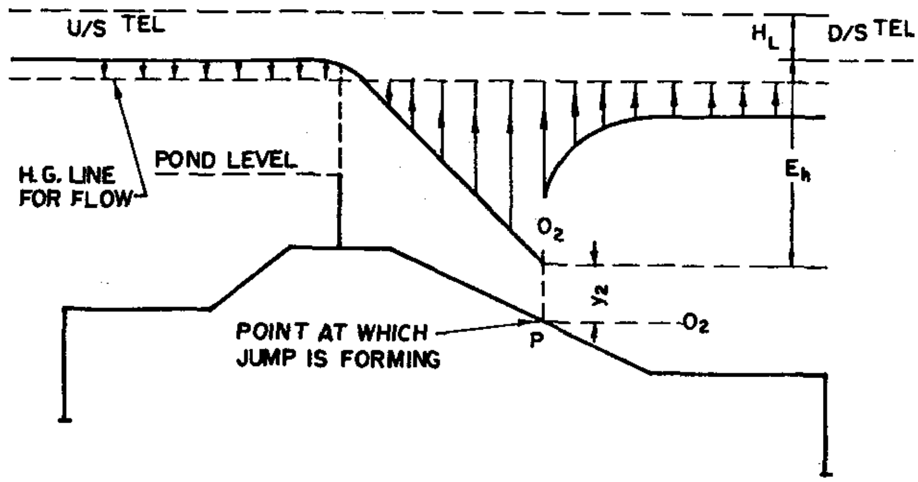


Fig. 20. Uplift pressure in jump trough with flow

The ordinate of the uplift pressure is measured from the hydraulic gradient line to the water surface as the rest of the uplift is counterbalanced by the weight of water standing on the floor. It is quite evident from the figure that there is no uplift pressure on the upstream floor as the hydraulic gradient line is below the water surface and a nominal floor thickness is required upstream.

Further if figures 19 and 20 are superimposed, it is found that the uplift ordinate O_2 is much larger in the second case than that of O_1 in the first case at the corresponding point P. Since the point of jump is likely to shift with the variation in discharge, the entire glacis may have to be designed for the second condition; while most of the remaining floor may have to be designed for the first condition. In fact, the floor thickness should be designed for the larger of the two ordinates. The uplift pressure due to dynamic action (hydraulic jump), is further reduced due to the following reasons:

- The backward rolling flow of water in the trough

- Uplift pressure is maximum at the point of jump formation but reduces rapidly on either side. As the floor has beam action, it may be designed for average uplift ordinate instead of the maximum ordinate

- The vertical component of the momentum remains unaffected in the jump, which exerts a downward pressure in the vertical direction.

Due to reasons stated above, the uplift pressure due to dynamic action is reduced to about two-thirds of it's theoretical value, and the design is done for maximum of either the two-thirds of the largest ordinate due to jump action or the largest ordinate due to steady seepage.

3.8 Loose Protective Work (inverted filter)

Protective works are required on the upstream and downstream of the regulators or sluices in order to alleviate the possibility of scour hole travelling close to the concrete floor of the regulator/sluice and to relieve any residual uplift pressure through the filter. The arrangement consists of :

- Inverted filter and
- Launching apron

<u>**Inverted filter (downstream)**</u>

At the end of concrete floor, an "inverted filter", usually recommended as 1.5 times D to 2.0 times D long is generally provided, where D is the depth of scour below original river bed. An inverted filter invariably reduces the possibility of piping as it allows free flow of seepage water through itself without allowing foundation soils to be lifted upward. The filter consists of layers of materials with increasing permeability from bottom to top. The gradation should be such that while it allows free flow of seepage water, the foundation material does not penetrate or clog the filter. The following limits are recommended to satisfy filter stability criteria and to provide ample increase in permeability between the base and filter. To prevent filter from dislocation under surface flow, concrete or masonry blocks are laid over the filter material.

(i) $\dfrac{D_{15} \text{ of the filter}}{D_{15} \text{ of base material}} \geq 5$, provided that the filter does not contain more than 5 percent of material finer than 0.074 mm after compaction.

(ii) $\dfrac{D_{15} \text{ of the filter}}{D85 \text{ of base material}} \leq 5$

D_{15} is the size at which 15 percent of the total soil particles are smaller; the percentage is by weight as determined by mechanical analysis. The D85 size is that at which 85 percent of the total soil particles are smaller. If more than one filter layer is required, the same criteria are followed; the finer filter is considered as the base material for selection of the gradation of the coarser filter.

The recommended size of blocks for regulators and drainage sluices, is 380 mm x 380 mm x 300 mm but the size depends on the velocity of flow at the exit. The size of block can be determined from the well known formula in California Highway Practice (CHP).

$$W = \frac{0.00002\ V^6\ sg_r\ \text{cosec}^3\ (\Phi - \theta)}{(sg_r - 1)^3} \qquad \ldots.(8)$$

where,

W = weight of stone in pounds (two-third of stone should be heavier)

Φ = 70 degree constant for broken stone

sg_r = specific gravity of hard material

θ = slope of material measured from horizontal face slope

V = mean velocity in feet per second

<u>Launching apron (down stream)</u>

At the end of inverted filter, a loose apron is provided for a length, generally equal to 1.5 times D, where D is the depth of scour from original river bed (figure 21).

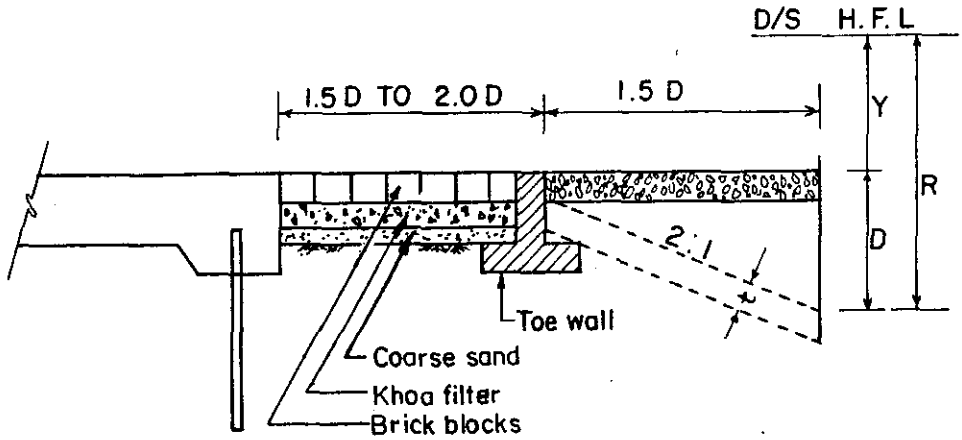


Fig. 21. Details of Loose Protective Works

The apron generally launches to a slope of 2:1, and if t is the thickness of the apron in the launched position, the design thickness of launching apron shall be equal to:

$(2^2 + 1^2)$ t/1.5 = (2.24 t/1.5) = 1.5 t

The empirical formula for the thickness of stone pitching as suggested by Inglish is,

$$t = 0.06\ Q^{1/3} \qquad \text{.....(9)}$$

where Q = discharge in cubic feet per second

t = thickness of stone in feet

Loose Protective Works (upstream)

Block protective work is also needed at the upstream of the regulators or sluices. General practice is to lay brick blocks over packed stone, for a length equal to D (D = XR - y, where X = 1.0 to 1.5, generally taken as 1.25), R is Lacey's normal scour depth and y is the depth of water above bed. Upstream of the blocks, launching apron is provided in the same way as described for the downstream portion, except that the proper value of

X should be chosen. Toe walls are generally constructed in between the 'inverted filter' and the "launching apron".

3.9 Design Examples

3.9.1 Mrigi Khal Drainage Regulator (3 - 1.52 m x 1.83 m)

Detail hydrological analysis for this regulator has been given in the Chapter 4 of this volume. A 3 vent 1.52 m x 1.83 m structure is selected with a design discharge of 16 cubic meter per second. The step by step computations required for the hydraulic designs are as follows :

3.9.1.1 Invert Level Fixation

The invert level for the regulator has been fixed at 4.30 m (PWD) from drainage consideration. The lowest level of the beel to be drained is about 5.5 m PWD which is about 5 kilometers away from the structure. Based on topography, a water surface gradient of 0.25 meter per kilometer is considered during drainage and the invert level fixes at 4.30 meter PWD.

3.9.1.2 Design Head Calculation

Pre-monsoon

Maximum countryside water level = 6.04 meter PWD (pre-monsoon routing)

Minimum riverside water level = 4.30 meter PWD (dry bed)

Design head C/S to R/S = 1.74 meter**Post-monsoon**

Country side water level = 7.80 meter PWD (post-monsoon routing)

Riverside water level = 9.94 meter PWD (outfall river level)

Design head R/S to C/S = 2.14 meter

3.9.1.3 Stilling basin Design

Flow Condition

h_1 = 6.04 - 4.30 (head water level - invert level)

= 1.74 meter

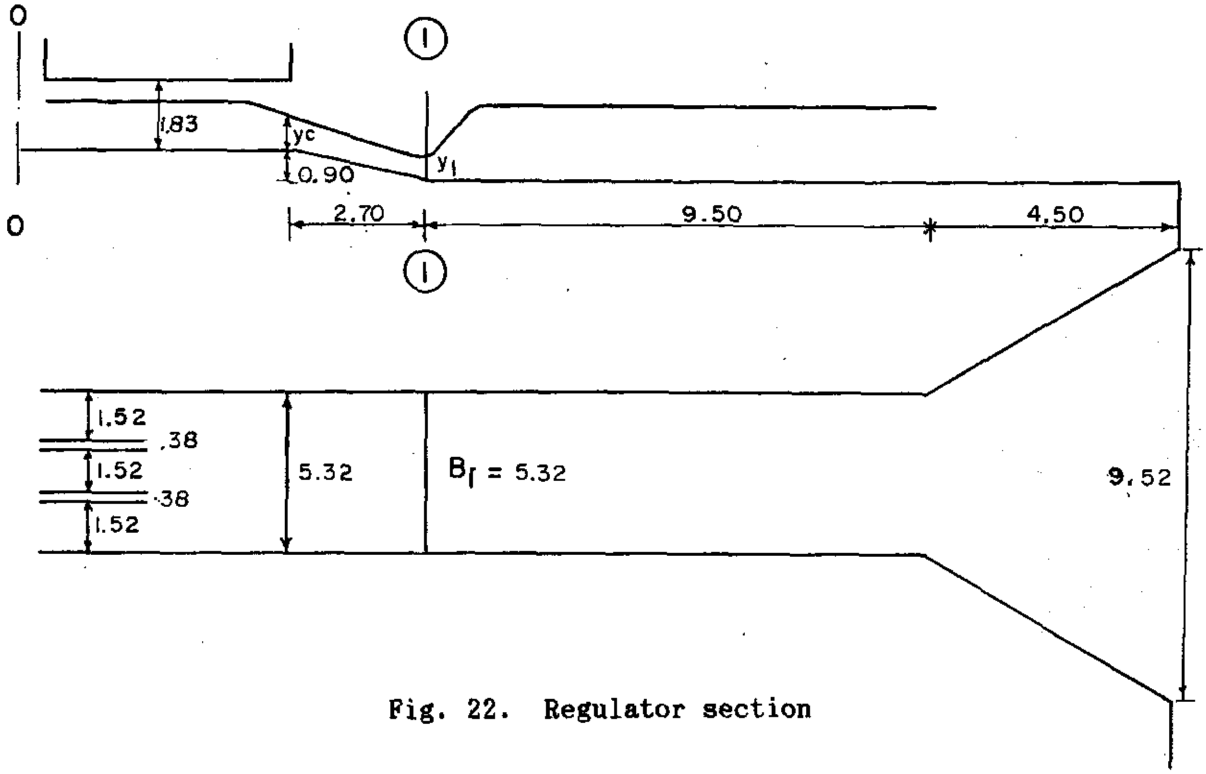


Fig. 22. Regulator section

D	= barrel height	(h_1 < 1.5 d and h_2 < 2/3 h_1)
	= 1.52 meter	The flow condition is type 5
Number of vent	= 3	
Size of vents	= width 1.52 meter; height 1.83 meter	
Discharge (Q)	= 3 x 1.52 x 1.48 x $1.74^{3/2}$	
	= 16.0 m^3 /sec	
Pier width	= 0.38 meter	
Flow width	= 3 x 1.52 + 2 x 0.38 = 5.32 meter.	
Flow/meter(q)	= 16.0/5.32 = 3.0 m^3 /sec/meter	

Depth (y_c) = $(q^2/g)^{1/3}$ = $(3.0^2/9.81)^{1/3}$ = 0.97 meter

Applying Bernoulli's energy equation between section (0 - 0) and (1 - 1) in figure 22, we can write,

$y_0 + V_0^2/2g + Z = y_1 + V_1^2/2g$ (neglecting entrance loss and conduit loss)

$y_0 = 6.04 - 4.3 = 1.74$ m (6.04 is the highest pre-monsoon level)

$Z = 0.60$ m (assumed)

or $1.74 + 0.60 = y_1 + 3.02^2/(2 \times 9.81 \times y_1^2)$ (neglecting velocity head)

or $2.34 = y_1 + 0.46/y_{12}$

By trial $y_1 = 0.50$ meter

$v_1 = 3.0/0.50$

$v_1 = 6.0$ m/sec.

$F_1 = 6.0 /(9.8 \times 0.5)^{\frac{1}{2}}$

$F_1 = 2.70$

$y_2 = (y_1/2)\ [(1 + 8F_1^2)^{\frac{1}{2}} -1]$

$y_2 = 0.50/2\ [(1 + 8 \times 2.70^2)^{\frac{1}{2}} - 1]$

$y_2 = 1.67$ meter

This results in a downstream water level = (4.30 - 0.60 + 1.67) = 5.37 meter. The corresponding tail water level (riverside water level) is 5.20 meter, and the jump may sweep out.

2nd trial with 0.90 meter drop of the river side apron

$q = 3.0\ m^3/sec$

$v_1 = 3.02/y_1$

substituting, $1.74 + 0.9 = y_1 + 3.02^2/(2 \times 9.81 \times y_1^2)$

$$2.64 = y_1 + 0.46 / y_1{}_2$$

By trial $y_1 = 0.46$ meter

$v_1 = 6.52$ m/sec

$F_1 = 6.52/(9.81 \times 0.46)^{\frac{1}{2}}$

$= 3.07$

$y_2 = (y_1/2)[(1 + 8F_1^2)^{\frac{1}{2}} - 1]$

$= (0.46/2)\ [(1 + 8 \times 3.07^2)^{\frac{1}{2}} - 1]$

$= 1.78$ meter

This results in a downstream water level = (4.30 - 0.90 + 1.78) = 5.18 meter, which almost matches the tail water level of 5.20. This situation will stabilize the jump within the basin.

Selection of stilling basin

Selection of the type of stilling basin depends on the value of froude number of the incoming flow. In this case, the froude number is 3.07 and we can use either USBR basin type IV, low froude number stilling basin or Indian Standard stilling basin type I.

Using Indian Standard Stilling basin I which has chute blocks, Baffle piers and dentated end sill (figure 13),

$L_1/y_2 = 4.30$

$L_1 = 4.30 \times 1.78$

$= 7.7$ meter

Increasing the length of the floor by about 20 per cent, the length provided is 9.5 meter. The total length of basin including glacis is 2.70 + 9.50 = 12.20 meter.

$v_2 = q / y_2$

$= 3.0 / 1.78$

$= 1.72$ m/sec

which is higher than the allowable velocity of 1.0 m/sec on the downstream channel. Therefore a 4.5 meter transition length with a flare angle of 25 degree has been provided to reduce the downstream channel velocity. After the transition, the exit velocity (v_2) is,

$v_2 = 16.0 / (9.52 \times 1.78)$

$= 0.94$ m/sec $<$ 1.0 m/sec okay.

For the design of auxiliary devices, e.g. the chute blocks, baffle piers and dentated end sills, standard text book, " Theory and Design of Irrigation Structure", volume II, by Varshney, S.C. Gupta and R.L. Gupta may be consulted.

3.9.1.4 Cutoff Wall Design

Downstream basin floor width between return walls

$= 3 \times 1.52 + 2 \times 0.38 + 2x\,(4.5 \tan 25^o) = 9.52$ meter

Discharge per meter at the end of downstream floor

$= 16/9.52 = 1.68\ m^3$ /sec

Lacey's depth of scour

$R = 1.35\ (q^2/f)^{1/3}$

$f = 1.76\ (dm)^{1/2}$

$= 1.76\ (0.015)^{1/2}$

$= 0.21$

$= 1.35\ (1.68^2/0.21)^{1/3} = 3.21$ meter

Considering a safety factor of 1.5 for the downstream scour, the required bottom elevation of riverside cutoff,

$= (3.4 + 1.78) - 3.21 \times 1.5 = 0.365 \sim 0.40$ meter PWD

Therefore, the required depth of downstream cutoff wall,

$= 3.40 - 0.40$

$= 3.00$ meter

Similarly, considering a safety factor of 1.25, for the upstream scour protection, the required depth of country side cutoff wall,

$= [4.00 - (4.00 + 1.74 - 3.21 \times 1.25)]$

$= 2.27$ meter (provided 3.0 meter)

3.9.1.5 Computation for exit Gradient

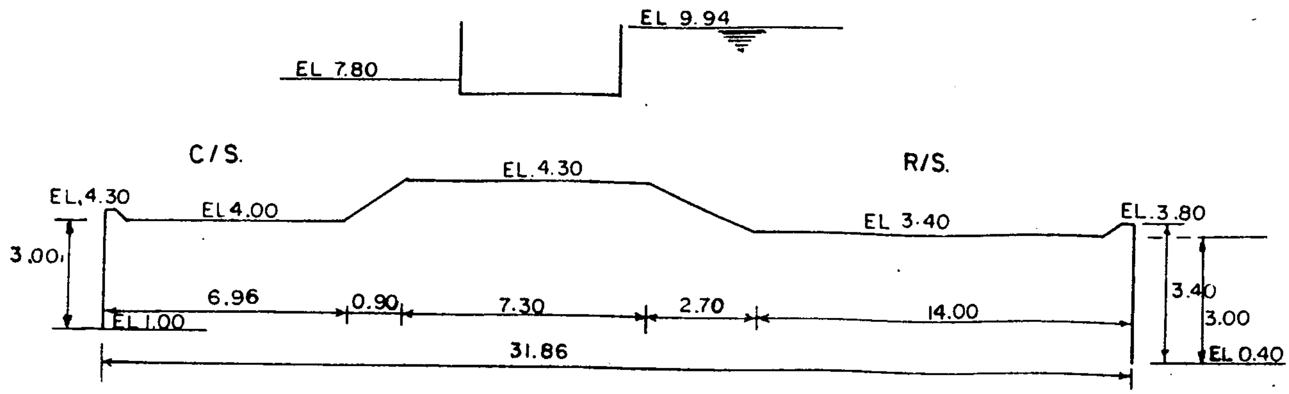


Fig. 23. Computation for exit Gradient

Maxm seepage head , H = 2.14 meter

Depth of C/S cutoff d = 3.00 meter

Total floor length b = 31.86 meter

α = b/d = 31.86/3.00 = 10.62

= $[1+ \sqrt{(1+\alpha^2)}]/2 = [1+ \sqrt{(1+ 10.62^2)}]/2 = 5.83$

<u>Exit Gradient.</u>

GE = $(H/d) x 1/\pi\sqrt{\Lambda}$

= 2.14/3.00 x 1/7.58 = 0.09 < 1/7 okay (figure 23).

3.9.1.6 Computation for uplift pressure and floor thickness

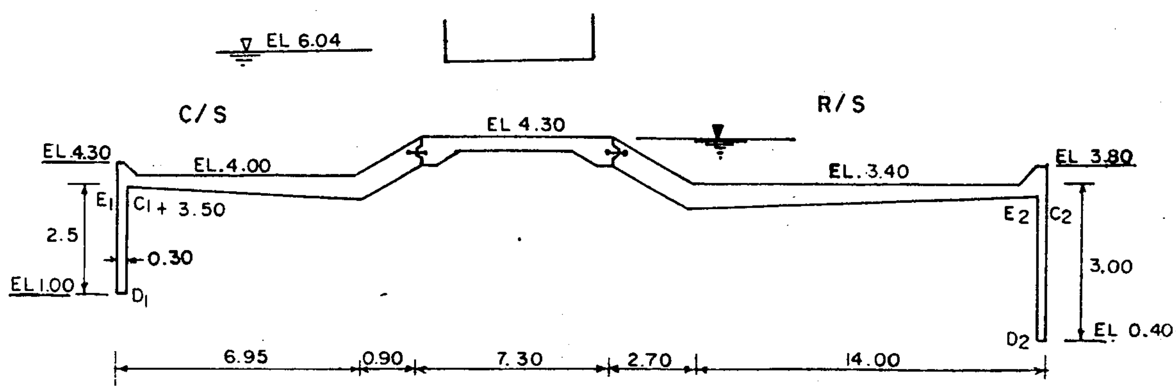


Fig. 24. Computation for uplift pressure

River side floor (figure 24)

Countryside cutoff No.1

Total length of the floor b = 31.86 m

Depth of C/S cutoff d = 4.00 - 1.00 = 3.00m

α = b/d = 31.86/3.00 = 10.62

$1/\alpha$ = 0.094

From curve (Figure 17)

ϕ_{E1} = 100%

ϕ_{C1} = 100% - ϕ_C = 100% - 32 = 68%

ϕ_{D1} = 100% - ϕ_D = 100% - 22% = 78%

The value of ϕ_C must be corrected for three corrections stated below :

Correction due to the effect of cutoff No.2 on cutoff No.1

Correction % = $19\ [D/b']^{1/2}\ [(d+D)/b]$

where,

D = depth of cutoff 2 below the point C_1 i.e, the point at which interference is desired.

= 3.50 - 0.40 = 3.10 meter

d = depth of cutoff No. 1 = 4.00 - 1.00 = 3.00 meter

b' = distance between two cutoffs = 31.26 meter

b = Total length of floor = 31.86 meter

$$\text{Correction} = 19\ [3.10 / 31.26]^{1/2}\ [(3.00 + 3.10)/31.86]$$

$$= 1.14\ \%\ (+ve)$$

Correction due to thickness of floor

Pressure calculated from curve for ϕC is at the top of floor but we want to know the pressure at the bottom of slab.

Correction = $[(\phi D_1 - \phi C_1)$/distance between C_1 & $D_1]$x floor thickness

= [(78 % - 68 %)/(4.00 - 1.00)]x0.50 = 1.66 % (+ve)

Correction due to slope = nil

Corrected ϕC_1 = 68 % + 1.14 % + 1.66 % = 70.8 %

Riverside cutoff wall No. 2

$1/\alpha$ = d/b = 3.00/31.86 = 0.094

ϕC_2 = 0 %

ϕE_2 = ϕE = 32 %

ϕD_2 = ϕD = 22 %

Correction for ϕE_2

Correction due to the effect of cutoff No.1 on cutoff No.2

Correction % = $19\ [D/b']^{1/2}\ [(d+D)/b]$

D = 2.90 - 1.00 = 1.90 meter

d = 3.40 - 0.40 = 3.00 meter

b' = 31.26 m

b = 31.86 m

Correction = $19\ [1.90\ /\ 31.26]^{1/2}\ [(3.00 + 1.90)/31.86]$

= 0.72 % (-ve)

Correction due to thickness of floor

= [(32 % - 22 %)/(3.40 - 0.40)]x0.50 = 1.66 % (-ve)

Correction due to slope = nil

Corrected ϕE_2 = 32 % - 0.72 % - 1.66 % = 29.62 %

Design Head, H = 1.74 m

Maximum uplift pressure at sec.1 = 51.60 % of 1.74 = 0.89 m of water

Maxm uplift pressure at sec.2 = 29.62 % of 1.74 = 0.51 m of water (figure 26). This can be more accurate done by drawing the hydraulic gradient line on the structure configuration.

Submerged weight of concrete/Weight of water = (23.6 -9.81)/9.81 = 1.40

Required thickness of floor at sec.1 = 0.89/1.40 = 0.64 m ≈ 700 mm

Required thickness of floor at sec.2 = 0.51/1.40 = 0.36 m ≈ 500 mm

Countryside floor (figure 25)

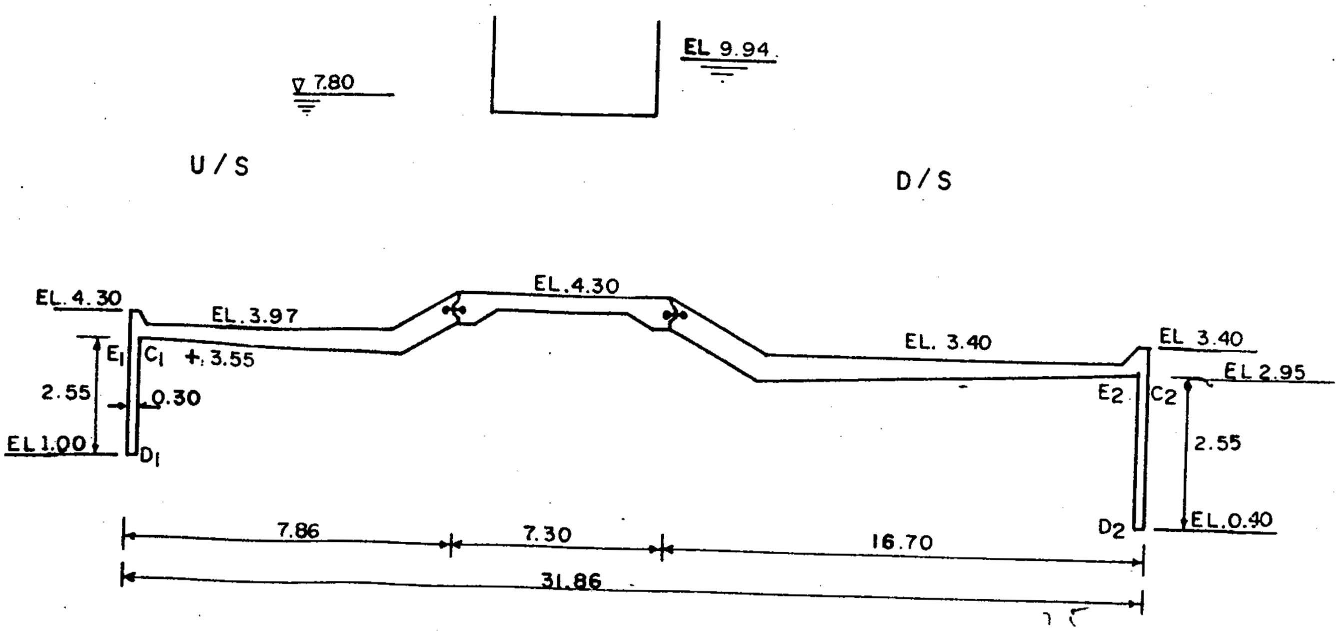


Fig. 25. Computation for uplift (c/s floor)

Riverside cutoff No.1

Total length of the floor b = 31.86 meter

Depth of R/S cutoff d = 3.40 - 0.40 = 3.00 meter

α = b/d = 31.86/3.00 = 10.62

$1/\alpha$ = 0.094

From curve (Figure 17)

ϕE_1 = 100 %

ϕC_1 = 100 % - ϕ_E = 100 % - 32 % = 68 %

ϕD_1 = 100 % - ϕ_D = 100 % - 22 % = 78 %

Correction for ϕC_1

Correction due to the effect of cutoff No. 2.

Correction % = $19\ [D/b']^{1/2}\ [(d+D)/b]$

D = 2.90 - 1.00 = 1.90 m

d = 3.40 - 0.40 = 3.00 m

b' = 31.26 b = 31.85 m

Correction = $19\ [1.90 / 31.26]^{1/2}\ [(3.00 + 1.90)/31.86]$

= 0.72 % (+ ve)

Correction due to thickness of floor

= (78 % - 68 %)/(3.40 - 0.40)x0.50 =1.66 % (+ve)

Correction due to slope of floor = nil

Corrected ϕC_1 = 68 % + 0.72 % + 1.66 % = 70.38 %

Countryside cutoff wall No. 2

$1/\alpha$ = d/b = 3.00/31.86 = 0.094

ϕC_2 = 0 %

ϕE_2 = ϕE = 32 %

ϕD_2 = ϕD = 22 %

Correction for ϕE_2

Correction due to the effect of cutoff No.1 on cutoff No.2

Correction % = 19 $[D/b']^{1/2}$ $[(d+D)/b]$

D = 3.50 - 0.40 = 3.10 m

d = 4.00 - 1.00 = 3.00 m

b' = 31.26 m

b = 31.86 m

Correction = 19 $[3.10 / 31.26]^{1/2}$ $[(3.00 + 3.10)/31.86]$

= 1.14 % (- ve)

Correction due to thickness of floor

= (32 % - 22 %)/(4.00 - 1.00)x0.50 = 1.66 % (-ve)

Corrected ϕE_2 = 32 % - 1.14 % - 1.66% = 29.2 %

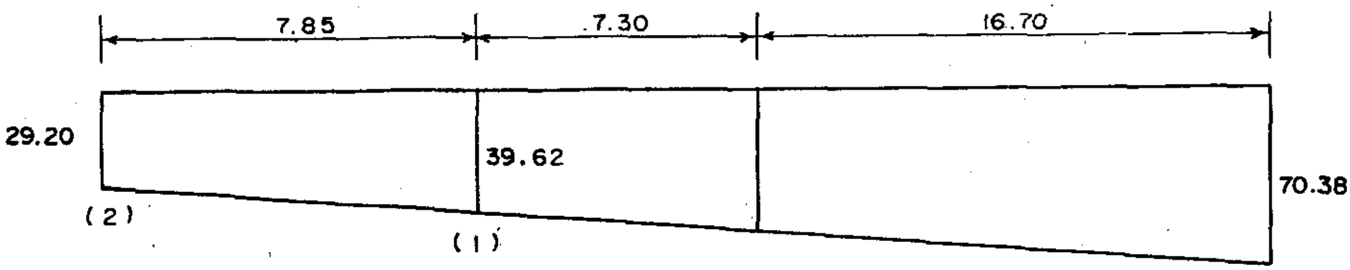


Fig. 26. Uplift Pressure Diagram.

Design Head, H = 2.14 m

Maxm uplift pressure at sec.1 = 39.35 % of 2.14 = 0.84 m of water

Maxm uplift pressure at sec.2 = 29.2 % of 2.14 = 0.68 m of water (figure 26).

Required thickness of floor at sec.1 = 0.84/1.40 = 0.60 m ≈ 0.630 mm

Required thickness of floor at sec.2 = 0.62/1.40 ≈ 500 mm

3.9.2 Typical tidal drainage sluice

Design data:

Drainage basin area	:	1300 hectares
Maxm. riverside monsoon water level (average design conditions)	:	2.70 meter (PWD)
Minm. riverside monsoon water level (for average design condition)	:	(-) 0.60 meter (PWD)
Bed level of natural channel at the proposed site	:	(-) 0.30 meter (PWD)
Bed width of natural channel at the proposed site	:	8.0 meter
Average ground level of the basin	:	1.20 meter (PWD)
Rainfall excess (1 in 10 year)	:	45.70 millimeter/day
Embankment crest level	:	4.88 meter (PWD)

<u>Sluice Size Determination (approximate method)</u>

Design countryside water level = 1.5 meter (average GL + 0.3 m)
Tidal range is 3.30 meter in 6 hour
Assuming a linear water level fluctuation, the water level rises or falls at a rate of 0.55 meter per hour.
Invert level of the sluice is at (-) 0.60 meter PWD
Ratio of head water depth to barrel height (Hw/D) = 2.1/1.83 =1.15
Discharge per unit barrel width (Q/b) from design chart (Fig. B-11 Page 588 Design of Small Dam, Third Edition) is 4.66 m^3/sec/ m (50 ft^3/sec/ft)
Total discharge for a box, 1.52 meter wide = 4.66 x 1.52 = 7.08 m^3/sec
For rectangular section critical depth $y_c = (q^2/g)^{1/3}$

$$y_c = (4.66^2/9.81)^{1/3} = 1.30 \text{ meter}$$

The sluice will be discharging in inlet-control as long as the tailwater is at or below the elevation,

$(yc + D)/2 = (1.30 + 1.83)/2 = 1.57$ meter (tail water el.+ 0.97 meter)
Discharge in inlet control would remain constant at 7.08 cubic meter per sec as the headwater is assumed constant. From the elevation of 0.97 meter PWD to 1.50 meter PWD, the control will be shifted to outlet and the discharge will linearly be reduced to zero as shown in figure 27.

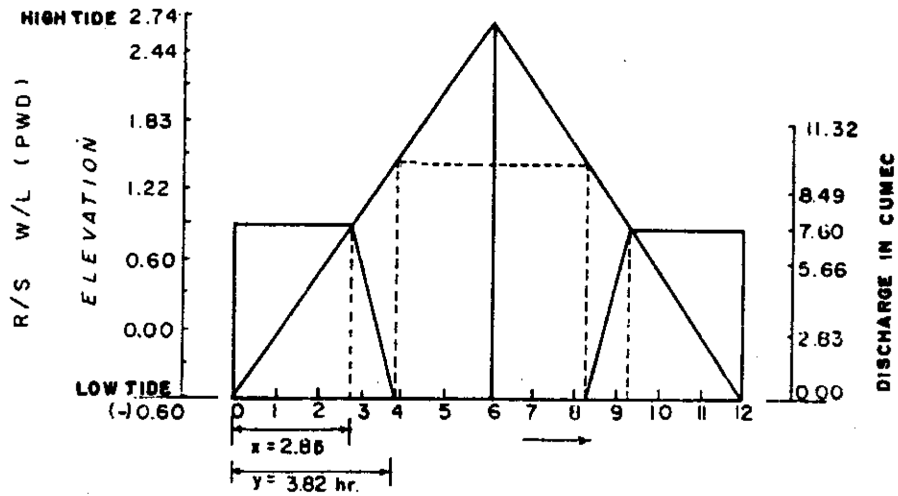


Fig. 27. Tidal Discharge Curve

Drainage is constant, by similar triangles, for a period

$t_1 = (6/3.3) \times 1.57 = 2.85$ hour

Drainage reaches a discharge rate of zero from change of tide to tide blockage. $t_2 = (6/3.3) \times 2.1 = 3.82$ hour

Volume of water discharged under one drainage curve in 12 hours

$$V = \left[2\,(2.85 \times 7.08) + \frac{2\,(3.82 - 2.85) \times 7.08}{2}\right] \times 3600$$

$= [\ 40.35 + 6.87\ ]\ 3600$

= 169992 cubic meter in 12 hours

= 339984 cubic meter per day

= 3.94 m^3/sec

Therefore average discharge for 24 hours period by one vent 1.52 m x 1.83 m sluice is 3.94 cubic meter per second.

45.7 millimeter of rainfall excess over an area of 1300 hectares will generate a total drainage discharge for the basin

(45.7/1000) x 1300 x 10000 = 594100 cubic meter per day

So the number of vents required = 594100/339984 = 1.75

Provide a 2 vent 1.52 m x 1.83 m for the drainage basin.

3.9.3 Typical Water Retention Structure

A water retention structure is to be constructed on a channel with the following design parameters.

Natural Bed Width	:	20 meter
Natural Depth of Channel	:	3 meter
Side Slopes	:	2:1 (H:V)
Discharge (bankful)	:	70 cumec

The channel is perennial and the structure will retain water in lean period for winter irrigation to cultivable land bordering the channel. The structure will be designed to permit little or no backwater effect upstream of the structure during flood flow. The following calculation determines the minimum opening required to pass the flood discharge through the structure.

Hydraulic Design Steps

For a channel flowing at a discharge of 70 m^3 /sec and a depth of flow 3.0 meter, the average velocity in the cross section is 0.9 m/sec

$$\text{Specific energy of flow } (E) = y + \frac{v^2}{2g} \qquad \ldots\ldots(10)$$

$$= y + \frac{Q^2}{2g\ A^2}$$

in which

y = depth of flow from mean bed level

Q = discharge is m^3/sec

A = cross-sectional area (m^2)

g = gravity acceleration (9.8 m/sec^2)

E = specific energy in meter above bed level

$$\text{Thus} \quad E = 3.0 + \frac{(70)^2}{2x9.8x(78)^2}$$

$$= 3.04 \text{ meter}$$

Compute the minimum width of a rectangular and a trapezoidal section which will convey this flow without "chocking" the flow. For the rectangular section the procedure is :

Assuming critical depth will occur at the constriction and the depth of flow "y_c" is then two-thirds of the specific energy of the flow upstream.
Thus y_c = 2/3 x 3.04 = 2.03 meter
For rectangular channels

Critical depth (y_c) $= (q^2/g)^{1/3}$ where $q = Q/b$

$$y_c{}^3 = (q^2/g)$$

$$= 70^2/(9.81 x b^2)$$

$$b = (59.8)^{1/2} = 7.73 \text{ meter}$$

$$\sim 8.0 \text{ meter}$$

Thus an opening with a clear span of 8 meter will just permit the flood flow to take place without noticeable back water effect. However, if piers are installed in this opening to facilitate the placement of 1.5 meter long timber fall boards, the opening need to be increased to accommodate the piers plus some allowance for contraction of the flow at the piers. Thus for a design utilizing a rectangular opening and no step-up in the bed, minimum recommended opening distance from abutment to abutment is about 10.50 meter as shown in figure 28.

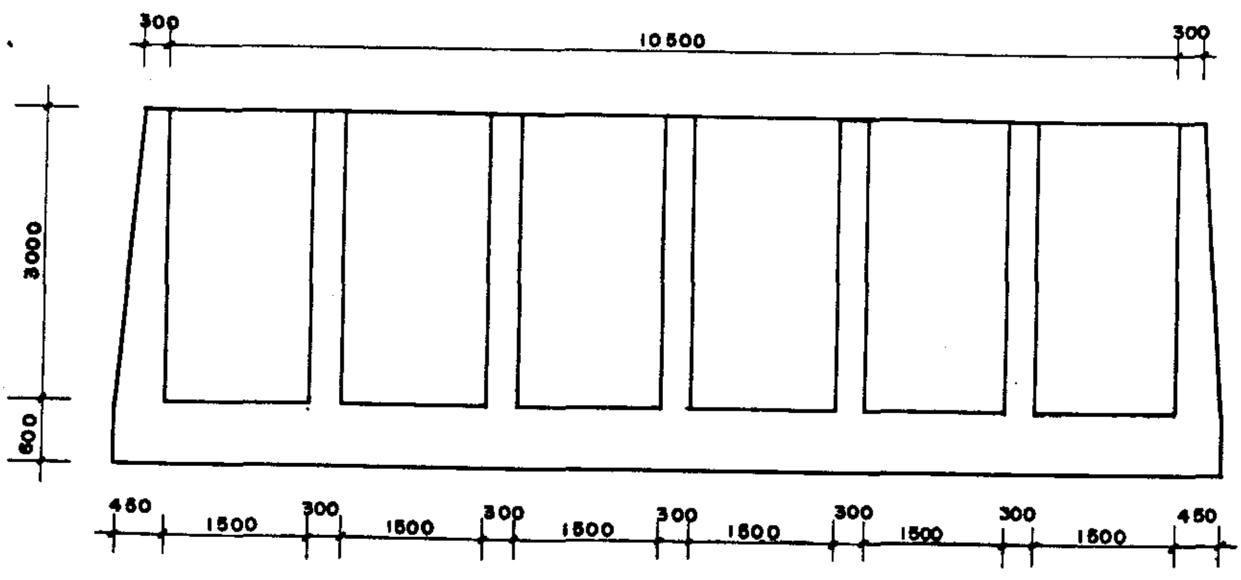


Fig. 28. Typical Design of Water retention structure

4.0 DESIGN OF EMBANKMENT (5)

4.1 Introduction

The design of the embankment is based on the utility of the embankment. The following questions need to be answered in selecting the design criteria for the embankment:

- How safe the embankment should be and what damages with respect to total investment costs are acceptable.
- Comparison of total costs (construction costs, maintenance costs and repair costs) with economic losses resulting from damage to land, crops, structures and population in case of failure of embankment.

The cost of embankment will increase with the increase in height of embankment in view of the anticipated damages. But it is difficult to assess how this anticipated damage will decrease with the increase in embankment height. Generally a flood level with a return period of 20 years (i.e. chance of occurring once every 20 years) is adopted for the crest height of the embankment. In special areas, where significant capital investments are planned, with regard to socio-economic and agricultural development, a higher period is considered.

4.2 Embankment Design Criteria (5,6)

The basic principle of embankment design is to produce a satisfactory, functional structure at a minimum cost. Proper consideration must be given to maintenance so that the savings achieved in the initial cost of construction do not result in excessive maintenance cost. The following design steps should be taken into consideration:

- Design crest level

- Design crest width
- Provision for berms
- Set back distance
- Stability of embankment

4.2.1 Design Crest Level

The design criteria, to fix the design crest level, are related to the stability of the embankment under all loading conditions. The loading conditions differ between sea dykes and river embankments. The following loading conditions are considered in fixing the design crest level of embankment:

- Design flood level and duration
- Wave run-up
- Wind set-up
- Shrinkage and settlement

<u>Design Flood Level</u>

The flood level which the embankment has to retain, is the major criteria for the design of embankment. As highlighted in paragraph 4.1, design flood level should be adopted which occurs once in 20 years in normal condition and higher frequency for special areas where the increased cost of construction can be justified. Design flood level is determined by extreme value analysis or the frequency analysis of the extreme annual value of flood. The procedure is as follows in Gumbel method:

- rank maximum annual observed values, X_i in descending order
- calculate return period,"T" for each rank "m" using the formula

$T = (n+1)/m$, where

m = rank of the event in order of magnitude

n = number of years of record

- plot T versus X_i on Gumbel probability paper

- calculate arithmetic mean $\bar{X}$ of the observed values, $\bar{X} = Xi/n$

- calculate standard deviation, $= \Sigma(Xi-\bar{X})^2 / n-1$

- calculate the theoretical distribution by $X_T = \bar{X} + K_T$, where

  X_T = extrapolated variate for a return period, T

  K_T = frequency factor for a return period, T

  $K_T = -\sqrt{6}/\pi\ [0.57721 + \text{Ln Ln}(T/(T-1))]$

- plot theoretical distribution on probability paper, compare with observed distribution and check the goodness of fit

- If it is not a good fit, other types of extreme value distribution, eg. log normal, log pearson III etc, may be tried.

<u>Wave run-up (5)</u>

The height of wave generated by winds blowing over water depends on the fetch. A relation between fetch (F), wind velocity (w), wind duration (t), wave height (H) and wave period (t), is presented in figure 29.

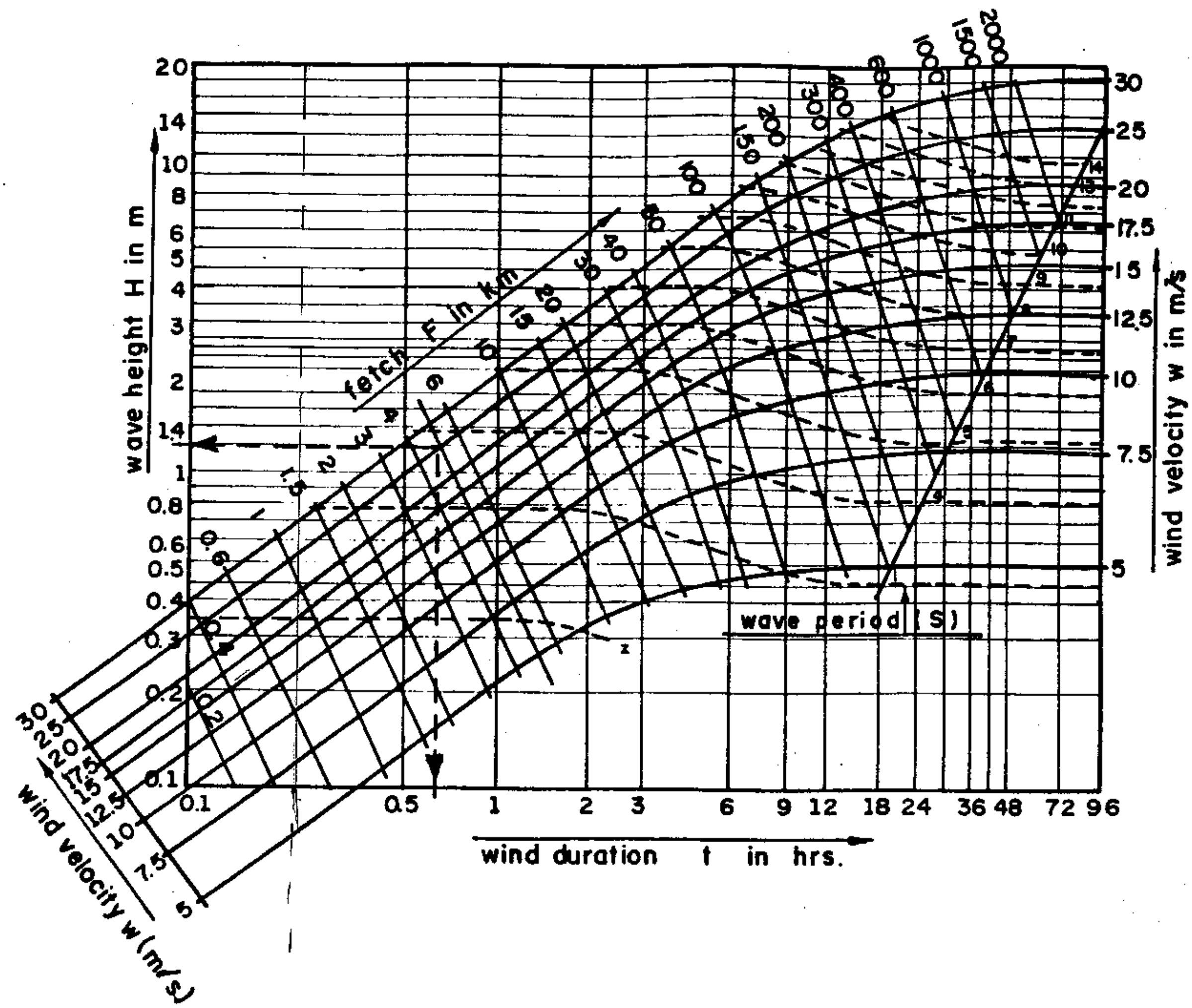


Fig. 29. Relation between wind velocity, fetch and wave height

Wind set-up

The wind set-up, will cause a slope of water surface and water will flow back along the bottom. This return flow exerts a friction force along the river bed as shown in figure 30.

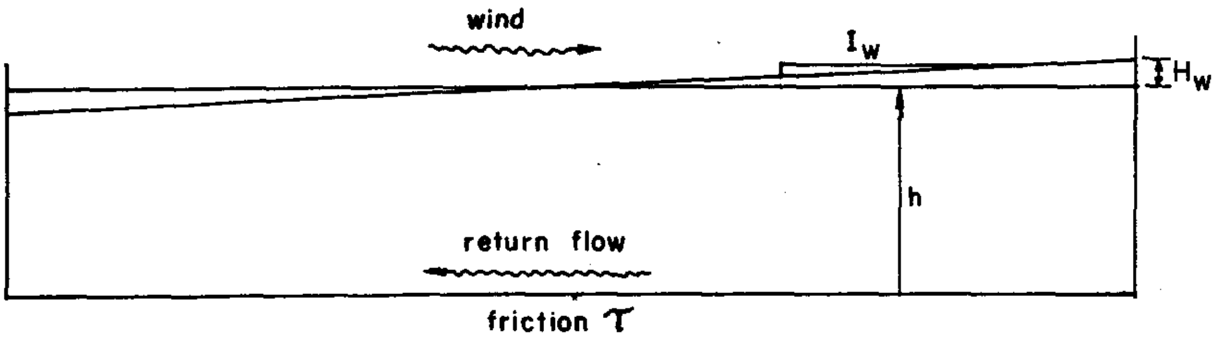


Fig. 30. Wind setup

The friction force $\Gamma = ghi \times F \times q$, ...(11)

in which

= density of water

g = gravity acceleration

h = average water depth

i = slope

F = affected area

q = constant factor

The force that the wind is exerting on the water surface is expressed as:

$$W = p.F.w^2,$$ ....(12)

in which

p = constant factor

F = water surface area affected by wind

w = wind velocity m/sec (6 meter above water level)

Equating 11 and 12, we get

$p.F.w^2 = ghi.F.q$ and $i = (p/q).\ w^2/gh$ where p/q is a dimensionless constant with a value of 4×10^{-6}

The wind setup gradient can thus be expressed as: $I_w = 4 \times 10^{-6} \times w^2/gh$

The general formula for calculating the wind setup is then:

$$H_w = 4 \times 10^{-6} \times w^2 \times l \times \cos\phi/gh \qquad ....(13)$$

where,
- w = wind velocity m/sec (6 meter above water surface)
- l = length of water area over which the wind is blowing
- g = gravity acceleration (9.81 m/sec^2)
- h = average water depth along stretch l
- ϕ = angle at which the wind is approaching coast

Shrinkage and Settlement

Shrinkage and settlement need proper consideration while fixing the crest level of embankment. From field experience, the shrinkage of a newly constructed embankment, averages about 10 per cent. So the fill height immediately after construction should be 10 per cent higher than the design height of embankment with a minimum allowance of 0.20 meter. For organic soil layers a settlement of 50% is considered and added to the 10% allowance for fill shrinkage. Peat soil must not be used for embankment construction. Settlement of the subsoil compressible strata under the body of the embankment is calculated by the formula,

$$S = (C_c H/(1+e_o)) \log_{10} [(P_o + P)/P_o] \qquad ....(14)$$

where,
- S = settlement in centimeters
- C_c = compression index
- H = subsoil depth from G.L to centre of compressible layer in cm.

P_0 = present overburden pressure

e_0 = initial void ratio

P = increase in pressure due to embankment surcharge

The increase in the height of flood level due to wave action and wind setup, is marginal for river embankments. In such cases, the design crest level is the sum of design flood level, shrinkage and settlement and an additional free board of 1 meter.

4.2.2 Design Crest Width

The minimum crest width of an embankment should be 2.4 meter. Crest width smaller than 2.4 meter, is vulnerable to more damage in case of overtopping due to erosion of crest and inner slope. If embankment inspection by vehicle is considered, the crest width shall be 4.3 meter. Where the embankment will serve as a road, the crest width shall be the width of the road plus 1.5 meter berm on either sides.

4.2.3 Berms

Berms may be defined as a shelf that breaks the continuity of a slope. Embankments are designed with berms on both sides, when required, for increased stability and additional safety against erosion. When artificial protection of the riverside slope is required, a small berm is provided to facilitate the transition between two different types of slope revetments.

4.2.4 Set Back for Embankment

The set back distance for the embankment can be determined as follows:

- provide enough fore land to locate borrow pit for the embankment
- provide additional space to increase height of embankment
- for sea dyke, additional fore land to locate a second borrow pit for repairs and additional land for afforestation to reduce wave actions
- at eroding bank, set back should include an extra margin equivalent to 10 years of the present erosion rate. Figure 31 shows typical example of set back distance for river embankments and sea dykes.

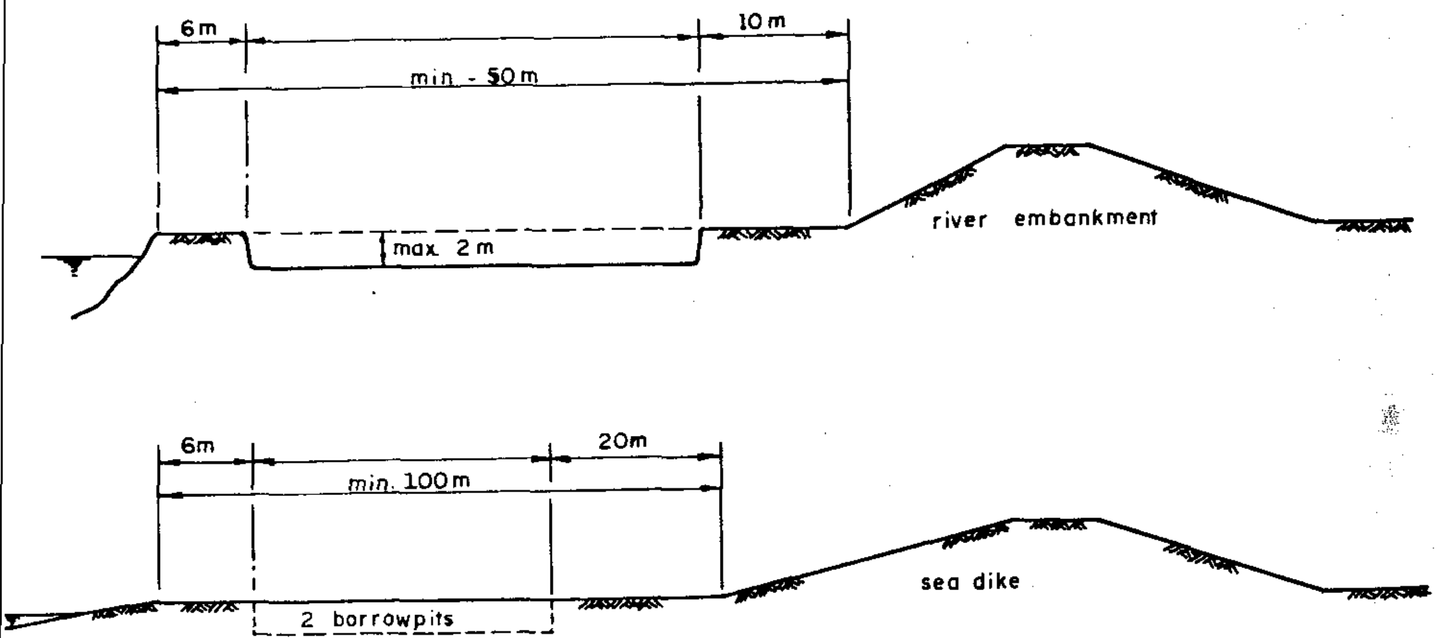


Fig. 31. Set back for river embankments and sea dykes

4.2.5 Slope of Embankment

While designing the slopes of embankment, distinction has to be made between sea dykes and river embankments. Sea dykes are subjected to flood level for short periods only, while river embankments resist flood for longer duration. Two important requirements for the selection of side slopes are:

- stability of the slope against adverse seepage flow
- stability of the section against shear failures.

Generally the side slopes for river embankment vary from 2:1 (H:V) to 3:1 (H:V) on both riverside and countryside. Sea dykes are subjected to severe wave attack and need proper design of the seaside slope. The stability of embankment slopes are checked by slip circle method and slopes are adjusted to satisfy the requirements. In general the sea dykes are designed with a seaside slope of 1:7. Before doing the stability analysis, the line of seepage or the phreatic line is drawn and the section is checked for uplift and seepage quantity.

Phreatic Line

Phreatic line or the line of seepage is defined as the line within the embankment section below which the hydrostatic pressure is positive. The flow through the embankment body below the phreatic line, reduces the effective weight of the soil due to pore water pressure. For the stability analysis of embankment, determination of phreatic line is very important. The phreatic line is assumed to be a base parabola with it's focus at F (figure 32), the downstream toe of the embankment. The equation of the base parabola is given by $\sqrt{(x^2 + y^2)} = x + s$, where s is the distance of the point F from the directrix, called the focal distance. The following procedure is adopted to locate the phreatic line:

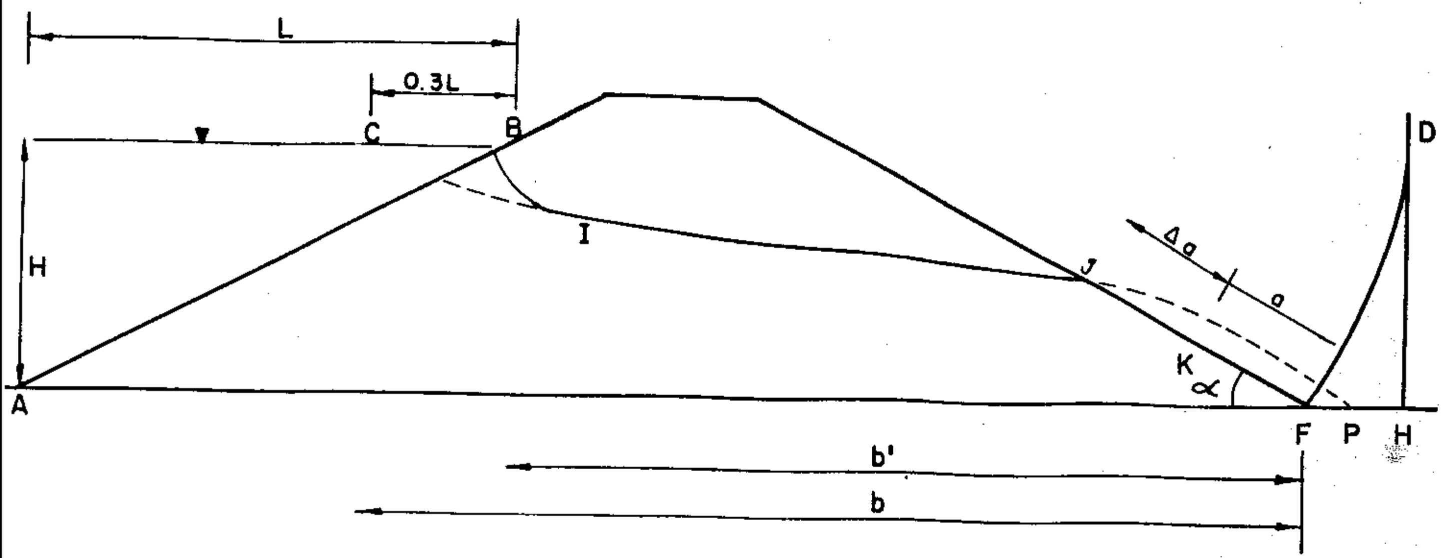


Fig. 32. Construction of Phreatic Line

- Assume the horizontal projection of the upstream face of the embankment AB (figure 32) as L. On the water surface, measure a distance BC = 0.3L. The point C is the starting point of the base parabola.

- To locate the directrix of the parabola, utilize the principle that any point on the parabola is equidistant from the focus as well as directrix. Draw an arc from the point C as centre and with radius CF to cut the horizontal line through CB at D. The vertical line DH is the directrix.

- The co-ordinates of the point C with respect to F is obtained from the geometry of the embankment section. Substituting these ordinates in the equation of the base parabola we get the value of S. The vertex 'P' of the base parabola shall be situated at a distance equal to S/2 from F beyond the downstream stream toe of the embankment.

- Using the equation of base parabola with the value of S determined in the previous step, a few more co-ordinates of the base parabola at known distances (X) are computed and the parabola is drawn.

- Now, this parabola has to be corrected at entry and exit points. The phreatic line is a flow line and must start from B and not from C as it is perpendicular to the upstream face AB which is an equipotential line. Hence the portion of the phreatic line from B to I is sketched freehand as a reverse curve in such a way that it starts perpendicularly to AB.

- At the exit the base parabola will cut the downstream slope at J and extend beyond the limits of the embankment as shown by the dotted line. But according to exit conditions, the phreatic line must emerge at some points K, meeting the downstream slope tangentially. The portion of the embankment KF is known as discharge face and always remains wet. The correction a by which the parabola is to be shifted downwards can be easily obtained by using the equation

$$a = (a + \ \ a) \ (\ (180^\circ - \alpha)/400^\circ) \qquad .....(15)$$

  where, $(a + \ \ a)$ and α are measured from the figure 32.

Knowing a, the point K is plotted and the phreatic line BIK is completed. In another way the point K can be located by finding the value of 'a' from the following equation ($\alpha < 30^0$)

$$a = (b'/\cos\alpha) - \sqrt{\ (b'/\cos\alpha)^2 - (H/\sin\alpha)^2} \qquad .....(16)$$

where b, and H are shown in figure 32.

Seepage Through Embankment

It is assumed that the embankment soil is homogeneous and isotropic in nature (horizontal permeability K_h = vertical permeability K_v). The flow net diagram of the embankment section under the phreatic line is drawn by free hand sketching and adjustments are made until the flow lines and equipotential lines intersect at right angles as shown in figure 33.

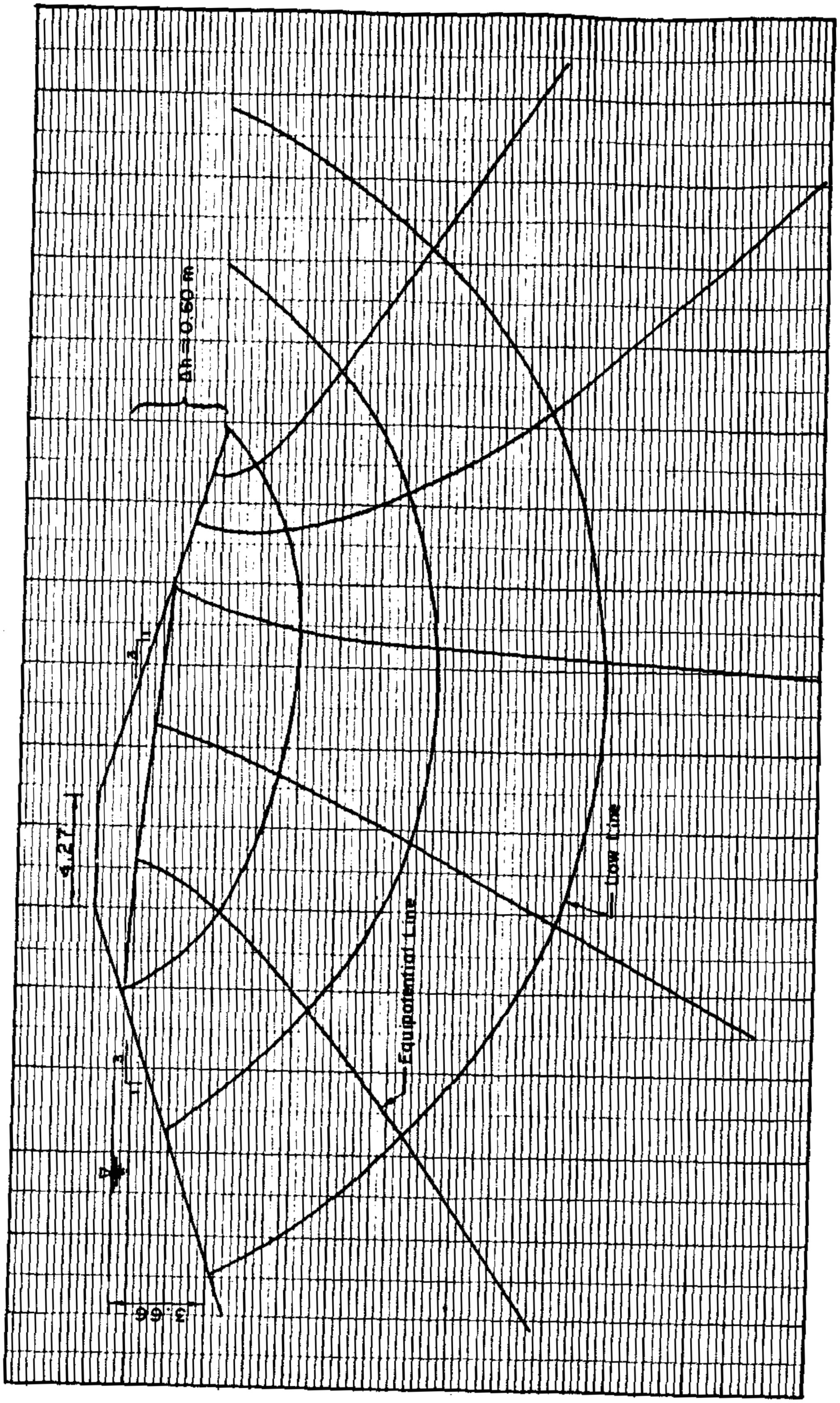


Fig. 33. Flow Net Diagram

The seepage rate, q can be computed from the flow-net using Darcy's Law. The required expression representing discharge passing through a flow channel is given by the equation,

$$q = (KH/N_d)\,N_f \qquad .....(17)$$

$K = (K_h . K_v)^{1/2}$

H = Total head loss

N_d = Total number of drops in the complete flow net.

N_f = Number of significant flow lines

Uplift Pressure and Seepage Quantity

The uplift pressure and seepage quantity is best explained by an specific example. Referring to figure 33, maximum upward hydraulic gradient into countryside, $i_{max} = h/L$

Where, h = head difference

L = length of flow line

Therefore, $i_{max} = 0.60/1.5 = 0.40$

Maximum uplift $F = i_{max}\ w\ v$

$= 0.4 \times 9.81 \times 1 = 3.92$ KN

Submerged weight of soil $s = 19.6 - 9.81 = 9.79$ KN/ m^3

Safety factor against uplift= $s/F = 9.79/3.92$

$= 2.50 > 1.5$ okay

Seepage Quantity

Seepage Quantity, $q = KH\ x(Nf/Nd)$

where

$K = (Kh.Kv)^{1/2} = 0.44$ meter/day

$H =$ Total Head loss = 3.66 meter

$N_f =$ Significant flow lines = 3 [Figure 33]

$N_d =$ No of potential drop = 6

$q = 0.44 \times 3.66 \times 3/6$

$= 0.805\ m^3$/day/meter of embankment width

$= 0.00000094\ m^3$/sec/meter of embankment

The seepage is very low (less than 1 cubic meter per day per meter of embankment) and the embankment section is safe against boiling and seepage.

Slope Stability Analysis

An earthen embankment usually fails due to sliding of a large soil mass along a curved surface. It has been established by actual investigation of railway embankments slides in Sweden that the surface of slip is usually close to cylindrical(an arc of a circle in cross-section). The method used to analyze the stability of slope of an earthen embankment is called swedish slip circle method. To locate the centre of the possible failure arc angle α and β are taken from figure 34 and the point of intersection of the angles α & β is approximately the center of the most dangerous circle.

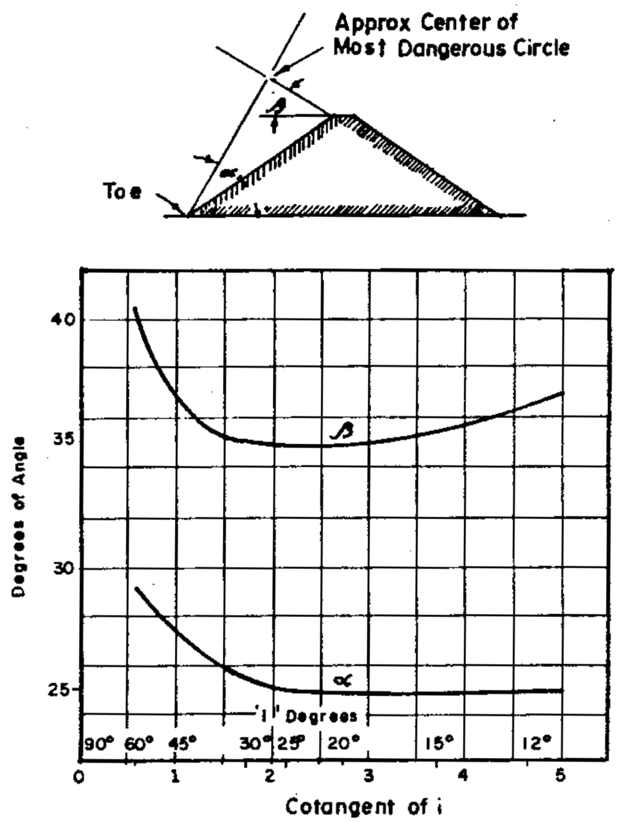


Fig. 34. Angles α and β for Different Slopes of Embankment

The earth mass within the arc is subdivided into six to twelve equal vertical segment called slices as shown in figure 35. The forces between these slices are neglected and each slice is assumed to act independently as a vertical column of soil of unit thickness and width.

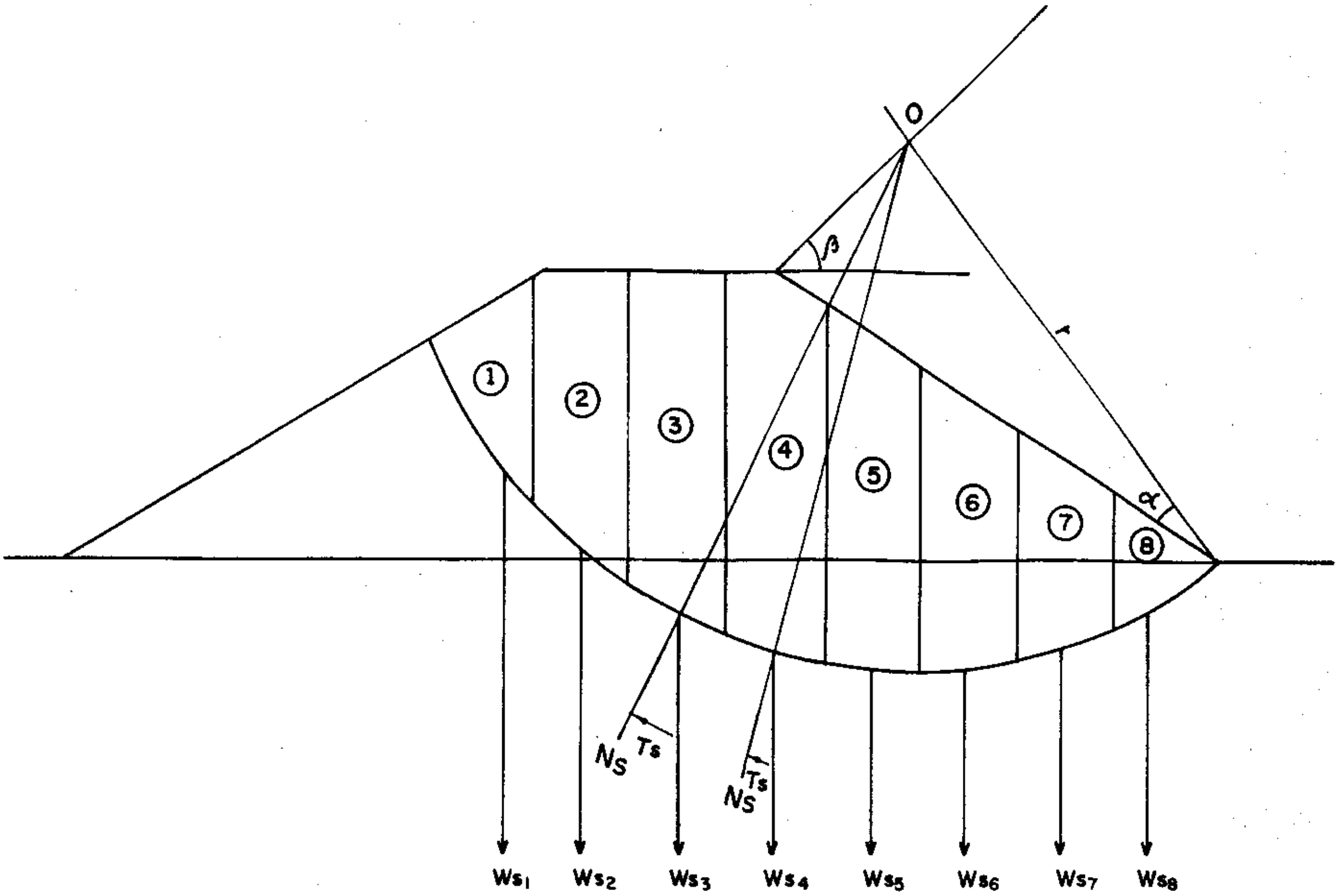


Fig. 35. Swedish Slip Circle

The weight Ws of each slice is assumed to act through it's centre of gravity. The weight Ws of each slice can be resolved into two components; a normal component Ns and a tangential component Ts as shown in figure 35 such that $Ns = W_s \cos\alpha$ and $Ts = W_s \sin\alpha$, where α is the angle which the slope makes with the horizontal.

The normal component (Ns) will pass through the centre of gravity of the segment and does not create any moment on the slice. However, the tangential component (Ts) causes a disturbing moment equal to T times r, where 'r' is the radius of the circle. The tangential components of few slices may create resisting moments and in that case Ts is considered as negative. Total disturbing moment will be the algebraic sum of all the tangential moments. The resisting moment is supplied by the development of shearing strength of the soil along surface of the arc. The magnitude of shear strength developed in each slice will depend upon the normal component (Ns) of that slice and is equal to C L + (N_s-u) tan ϕ where u is the pore water pressure. The factor of safety (FS) for the entire slip circle is then given by the equation:

$$FS = \frac{CL + (\Sigma N_s - \Sigma u) \tan \phi}{\Sigma T_s} \qquad .....(18)$$

where ΣT_s = summation of tangential forces of all the slices tending to produce moment of soil along the circumference of the arc.

ΣNs = Summation of normal forces for all the slices.

ϕ = Angle of internal friction of the material used for the construction of the embankment.

L = Length of arc intersecting the embankment in meter.

C = Cohesion per square meter

ΣU = Total pore water pressure on all the slices.

The factor of safety (FS) against sliding is normally taken as 1.5.

Location of the Centre of Critical Slip Circle

In order to find out the centre of the critical slip circle or the most dangerous circle, numerous slip circles are drawn and the factor of safety is calculated for each circle as explained above. The slip circle which produces the minimum factor of safety is the most dangerous circle or the critical slip circle. In order to reduce the number of trials, Felenius has suggested a method of drawing a line (PQ), representing the locus of the critical slip circle. The point P is obtained with the help of directional angles from figure 34 while the point Q is determined is such a way that its coordinates are (4.5H,H) from the toe as shown in figure 36.

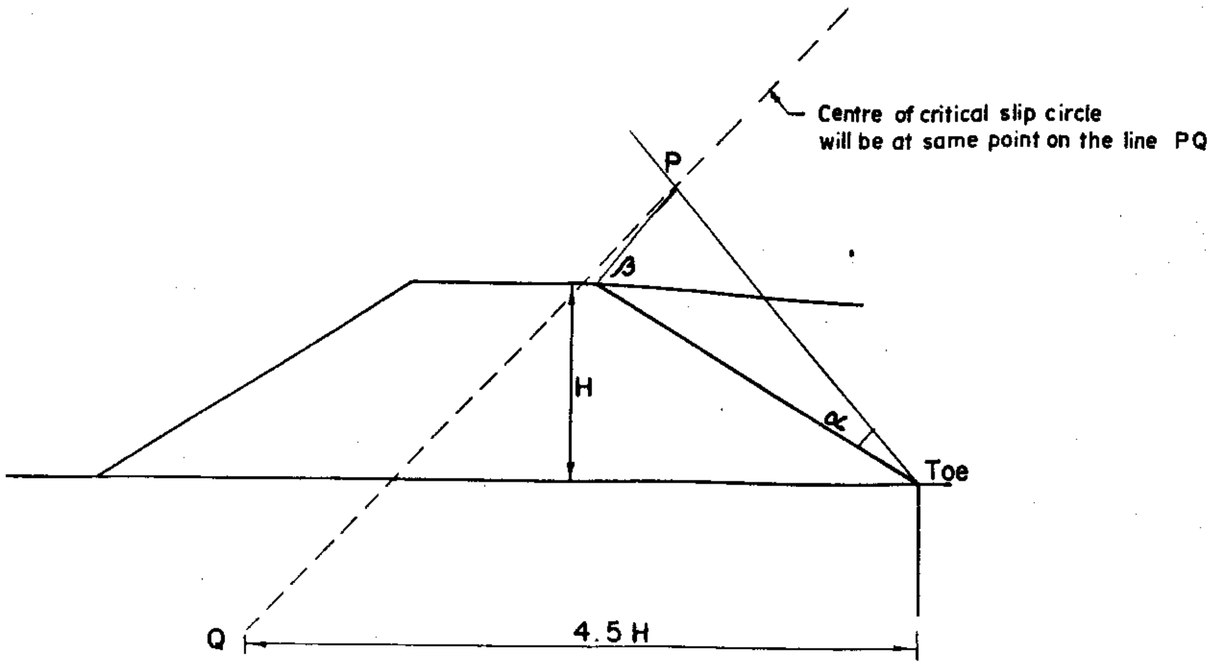


Fig. 36. Locus of Critical Circle for Downstream Slope

4.3 Design Example for Typical Embankment

An embankment with a crest width of 4.3 meter is to be constructed on the bank of a river. Determine the design crest level for the embankment considering 10 year return period using the yearly maximum water level data supplied:

Design Data

Wind velocity, w = 5 meter per second (10 knots)

Average water depth, h = 2 meter

Fetch length, F = 2 km

Embankment side slope:

R/S = 3:1 (H:V)
C/S = 3:1 (H:V)

Gravity acceleration $g = 9.81\ m/sec^2$

Water level data is given in table 9 below:

Table 9. Water Level Data

Year	Maxm W.L (m PWD)
1964-65	17.10
1965-66	16.55
1966-67	16.84
1967-68	17.29
1968-69	16.86
1969-70	17.27
1970-71	16.65
1971-72	18.29
1972-73	16.34
1973-74	17.35
1974-75	17.55
1975-76	17.55
1976-77	17.90
1977-78	17.35
1978-79	18.00
1979-80	16.63
1980-81	18.26
1981-82	17.35
1982-83	17.92

4.3.1 Computation of Design Flood Level

The frequency analysis is carried out by Gumbel Method. The annual maximum water levels are arranged in descending water and their frequency of occurring are calculated in table 10.

Table 10. Frequency Analysis

Rank (m)	Observed Annual maxm. water level (m)	Plotting Position (T= n+1/m)
1	18.29	20.00
2	18.26	10.00
3	18.00	6.00
4	17.92	5.00
5	17.90	4.00
6	17.55	3.33
7	17.55	2.86
8	17.35	2.50
9	17.35	2.22
10	17.35	2.00
11	17.29	1.80
12	17.27	1.67
13	17.10	1.54
14	16.86	1.43
15	16.84	1.33
16	16.65	1.25
17	16.63	1.18
18	16.55	1.11
19	16.34	1.05

$$\bar{X} = 17.32$$
$$\sigma = 0.56$$

Plotting position versus observed annual maximum water levels are plotted in figure 37. The value of K_T, the frequency factor for T year return period is given in equation below:

$$K_T = - \sqrt{6}/\pi[\gamma + \text{Ln Ln } (T/(T-1))] \qquad \text{.....(19)}$$

$$X = \bar{X} + \sigma K_T = 17.32 + 0.5649 \text{ Kt} \qquad \text{.....(20)}$$

The value of K_T and design flood level for different return period are calculated in table 11.

Table 11. Calculation of Design Flood

T (years)	K_T	X (m PWD)
5	0.719	17.73
10	1.305	18.07
20	1.866	18.37
50	2.592	18.78
100	3.137	19.10

Design flood level for 10 year frequency is 18.07 meter PWD.

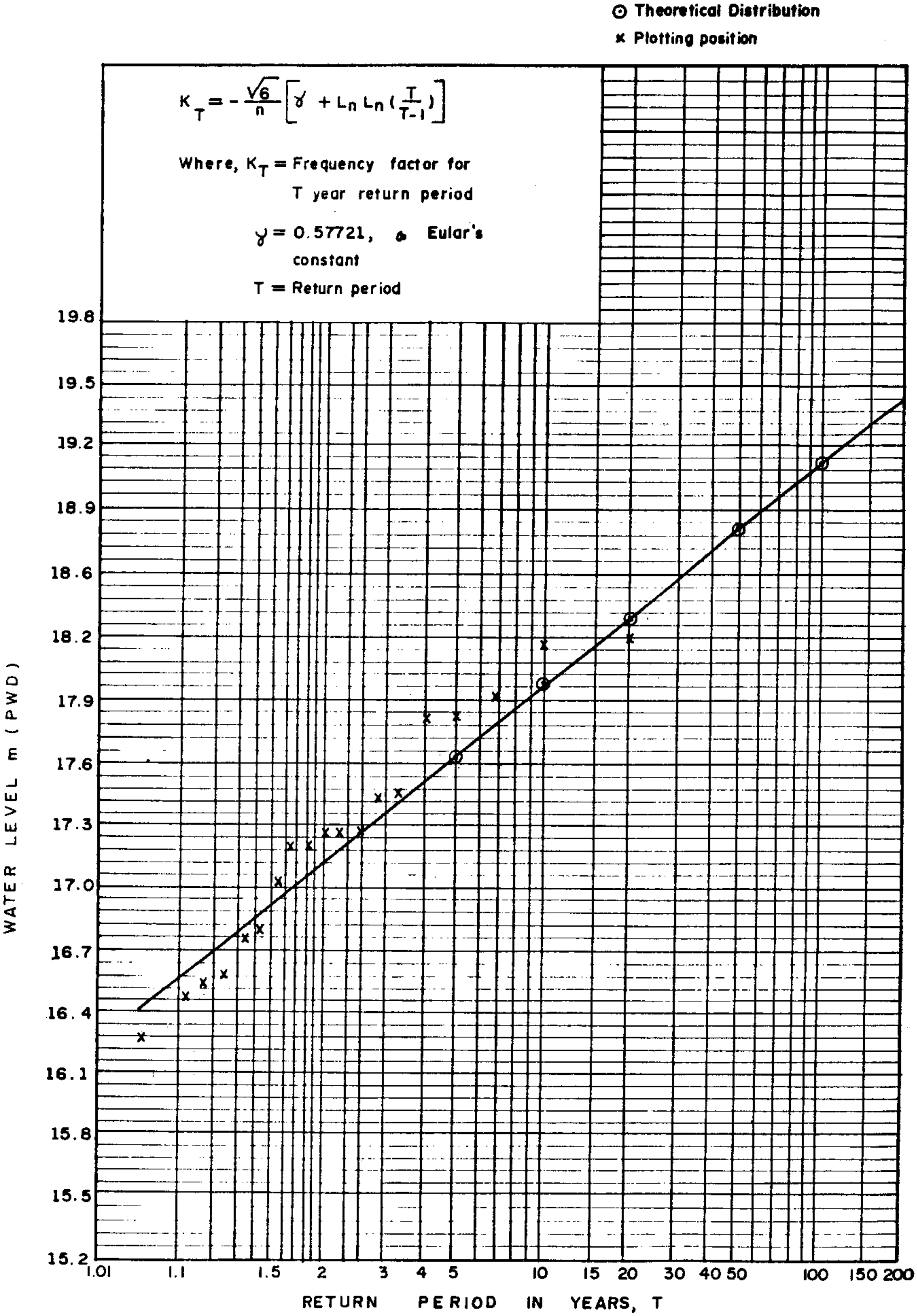


Fig. 37. Return Period vs Water level

4.3.2 Computation of Freeboard

When wind effect is significant

Given Data:

- Wind velocity (w) = 5 meter/sec (10 knots)
- Av.water depth (h)= 2 meter
- Fetch Length (F) = 2 Kilometer
- Embankment side slope
  Riverside 3:1, Countryside 3:1
- Gravity acceleration (g) = 9.81 meter/sec^2

Free board = wind setup height + wave runup height

Wind Setup

Wind setup Hw = Iw x Fetch Length

where, Iw = wind setup gradient = $4 \times 10^{-6} \times w^2/gh$

$$Hw = 4 \times 10\text{-}6 \times (\overset{20^2}{\cancel{5^2}}/(2 \times 9.81)) \times \overset{6000}{\cancel{2000}}$$

$$= 0.01 \text{ meter} \qquad 0.49$$

Wave runup

Wave runup, $Hz = 8\ f\ H \tan\alpha\ \overset{\text{Cos}}{\cancel{\text{Sin}}}\ \beta\ (1 - B/L)$

where,

f = constant = 1.1 (for turfed slope)

H = wave height = $0.0555 \sqrt{W^2 F}$

F = fetch Length in nautical mile

L = wave period = 0.5 $(W^2F)^{1/4}$

α = slope angle of Embankment

β = angle between the approach direction of incident waves.

B = berm width in meter

F = 2 km = 1.079 nautical mile

20 km = 10.79

H = 0.055 $\sqrt{W^2F}$ 60
= 0.0555 $\sqrt{\cancel{10^2}x\cancel{1.079}}$ 10.79
= 0.577

7.29

p-76

tanα = 1/3

β = 0
B = 0

Hz = 8 x $\cancel{1.1}$ x $\cancel{0.577}$ x 1/3 .5 79
= 1.69 m 9.7

Free board = 0.01 + 1.69 = 1.70 m (5.5 feet)

Where wind effect is not significant a minimum freeboard of 1 meter should be used above design flood level.

4.3.3 Determination of phreatic line

a = b'/cosα - $\sqrt{}$[(b'/cosα)2 - (H/sinα]2 = 20.9/cosα - $\sqrt{}$[(20.9/cosα)2 - (3.67/sinα)2]
= 21.82 - $\sqrt{(476.07 - 134.75)}$ = 3.34 m

s = $\sqrt{(b^2 + h^2)} - b = \sqrt{(24^2 + 3.66^2)} - 24 = 0.27$ m

Taking focus F as the origin, the equation of parabola is $\sqrt{(x^2+y^2)}=x+s$

$y = \sqrt{(2sx + s^2)} = \sqrt{(0.54 x + 0.073}$

where x is,	0.0	3.0	6.0	9.0	12.0	15.0	18.0	21.0
value of y is,	0.28	1.33	1.85	2.26	2.60	2.91	3.19	3.44

4.3.4 Settlement computation

Data given :

sat = 18.9 KN/m³ e_o = 1.0

dry = 16.5 KN/m³ C_c = 0.20

P, unit load at the base of embankment = 4.57 x 18.9 = 86.37 KN/m²
a = 13.72 m b = 2.10 m
side slope = 3:1 (H:V)

The vertical stress under the embankment at varying depth Z is calculated in table 12. The influence value for vertical stress is taken from figure 38.

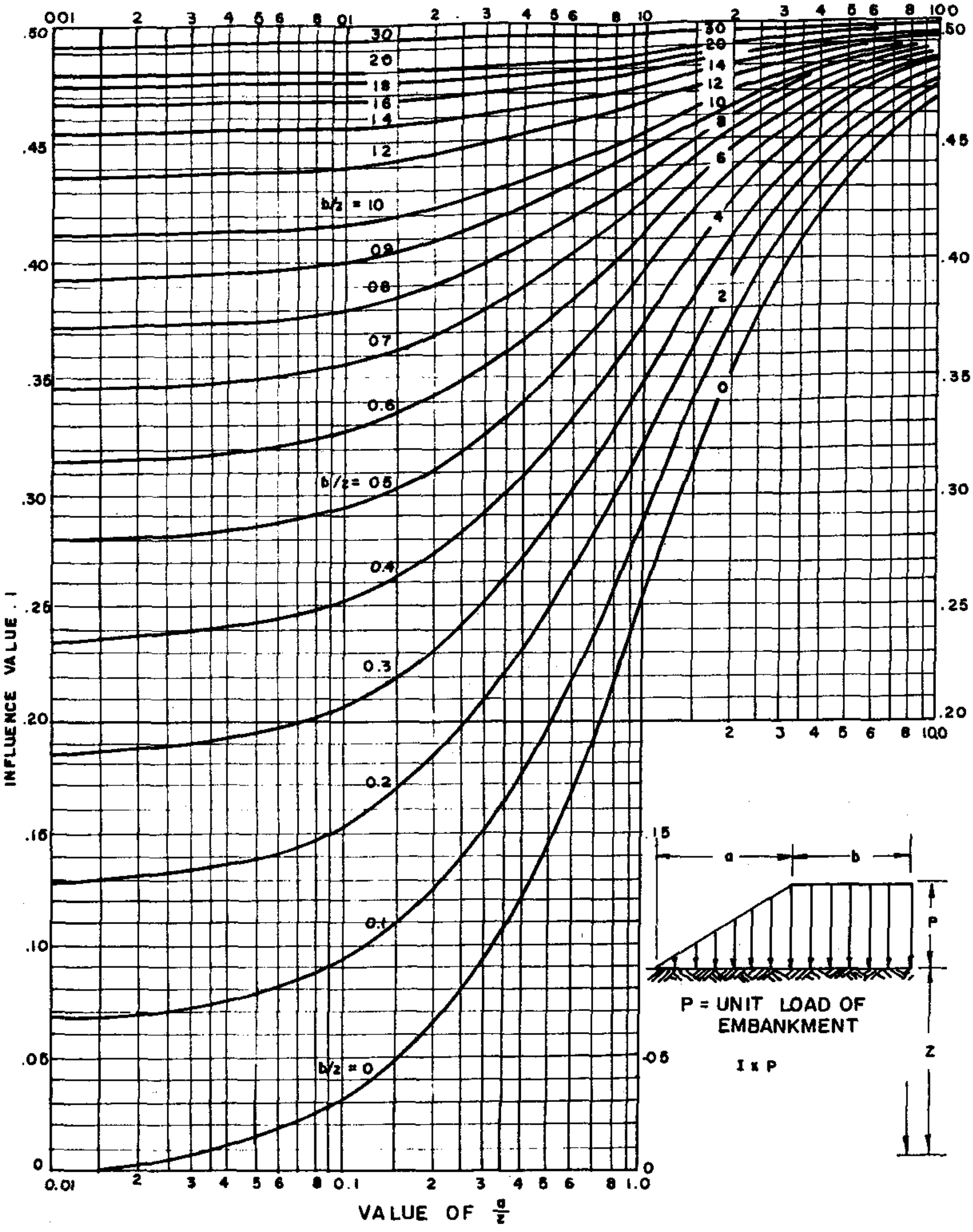


Fig. 38. Influence Value for Vertical Stress under Embankment

Table 12. Vertical stress under embankment for depths Z

Z (meter)	a/Z	b/Z	Influence value I (from Fig.38)	2I(for total embank.load)	stress p=2Ix (KN/sq meter)
1.5	9.15	1.40	0.490	0.98	84.64
3.0	4.57	0.70	0.475	0.95	82.05
6.0	2.29	0.35	0.425	0.95	73.41
9.0	1.52	0.23	0.370	0.74	57.52
12	1.14	0.18	0.320	0.64	49.75
15	0.91	0.14	0.290	0.58	45.08
18	0.76	0.12	0.260	0.52	40.42
21	0.65	0.10	0.225	0.45	34.98
24	0.57	0.09	0.200	0.40	31.09
27	0.50	0.08	0.190	0.38 0.38	29.54
30	0.46	0.07	0.170	0.31 .34	26.25

Settlement, $H = (C_c \times H/(1+e_0)) \times \log_{10} (P_0 + P)/P_0$ $C_c = 0.20$, $e_0 = 1.22$

For a vertical interval of 3.0 meter, $C_c x H/(1+e_0) = 0.20x3/(1+1.22) = 0.27$

For a vertical interval of 1.5 meter , $C_c x H/(1+e_0) = 0.20x1.5/(1+1.22) = 0.14$

$P_0 = (18.90 - 9.81)\ Z = 9.09\ Z\ KN/m^2$. The computation for settlement is carried out in table 13.

Table 13. Computation for settlement

Z(meter)	$P(KN/m^2)$	P0	log10(P0+ p)/P0	H (meter)
1.5	84.64	13.61	0.86	0.12
3.0	82.05	27.27	0.60	0.08
6.0	73.41	54.54	0.35	0.10
9.0	57.52	81.81	0.23	0.06
12	49.75	109.08	0.16	0.04
15	45.08	136.35	0.12	0.03
18	40.42	163.62	0.10	0.03
21	34.98	190.89	0.07	0.02
24	31.09	218.16	0.06	0.02
27	29.54	245.43	0.05	0.01
30	26.25	272.70	0.05	0.01
			Total =	0.52

Total settlement is 0.52 meter which should be taken into consideration during construction of the embankment i.e. an allowance of 0.52 meter in addition to the shrinkage allowance should be added to the height of the embankment. Hence crest level of embankment during construction is the total of design crest level + 10% allowance for shrinkage + settlement of subsoil. The design crest level of the embankment = (18.07 +1.0) + (4.57 x 0.10) + 0.52 = 20.04 m (PWD) .

4.3.5 Stability Analysis (Slip Circle Method)

Embankment height	=	4.57 m
Riverside slope	=	3:1 (H:V)
Countryside slope	=	3:1 (H:V)
Free board	=	1.0 meter
sat	=	18.9 KN/m^3
dry	=	16.5 KN/m^3
ϕ	=	10°
C	=	19.1 KN/m^2

For side slope 3:1 (H:V), From Fig. 35.

α = 25°

β = 37°

α & β are drawn and the point O, the centre of the slip circle is obtained (Fig. 35). Rest of the computation is done as explained in slope stability analysis in para 4.2.5 and the factor of safety is found equal to 2.38. To find the most dangerous circle, more slip circle are required to be drawn taking centres on the line PQ as shown in Figure 36. The circle which will produce the minimum factor of safety will be the most dangerous circle.

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