DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS
JAPAN INTERNATIONAL COOPERATION AGENCY

MANUAL ON FLOOD CONTROL PLANNING

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Project for the
Enhancement of Capabilities in Flood Control
and Sabo Engineering of the DPWH

MANUAL ON FLOOD CONTROL PLANNING

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MESSAGE

The country has been distressed by the catastrophic natural disasters that have struck the nation over the past decades. The loss of life has been enormous and the social disruption has caused heartrending setbacks for the society. The destruction of properties has been immensely discouraging in the economic system in which capital goods are scarce and the potential for saving and investment is very limited.

The government had undertaken several flood control and disaster prevention measures to reduce the impact of the natural disasters. Both structural and non-structural measures are of utmost importance in improving the level of the quality of alleviating human suffering and advancing development.

However, it has been observed that there are flood control structures that are either frequently damaged or not functioning as intended. One of the causes is the inadequate planning mainly due to budget constraints and lack of reference materials. It is essential that flood control structures are carefully planned and designed with due considerations on local site conditions, river hydraulic principles and phenomena to ensure that structures are effective in mitigating water-induced disasters. The formulation of this “Manual on Flood Control Planning” under the JICA - Assisted Project for the Enhancement of Capabilities in Flood Control and Sabo Engineering (Project ENCA) is an effort by the DPWH through the PMO-Flood Control and Sabo Engineering Center to provide references in planning of flood control structures.

Responsive approaches in planning of flood control structures are envisioned as this manual becomes available to the offices concerned in the Department of Public Works and Highways. Eventually in effect, improved flood control structures built by this department are effective, economical and sustainable.

My deep appreciation to the DPWH engineers and staff who gave time and efforts and the JICA experts dispatched for this Project for the technical assistance in the formulation of this manual. Likewise, my sincerest appreciation to the continuing support extended by JICA in the development of our technical capabilities in dealing with water-related disasters.

MANUEL M. BONOAN
Undersecretary

FOREWORD

Flood damage is categorized in terms of structure damage and inundation damage. One of the countermeasures to prevent inundation is to increase the flow capacity of the river. Although under the local funds, the flood control works by the DPWH are mostly intended to prevent scouring, their final targets are to prevent inundation. It is therefore essential for the DPWH engineers to understand flood control planning which includes (1) the determination of the design discharge and (2) the river improvement plan such as alignment, depth, height and width of the water area of the river and others.

The calculation to convert rainfall to discharge and water level analysis are essential techniques for flood control planning. Thus far, there is no comprehensive guideline for such planning. Therefore, the Manual on Flood Control Planning is introduced with the supplemental volumes listed below.

1. a. Specific Discharge Curve

   b. Rainfall Intensity Duration Frequency Curve

   c. Isohyet of Probable 1 –day Rainfall

2 Manual on Runoff Computation with HEC-HMS

3. Manual on Non-Uniform Flow Computation with HEC-RAS

This manual covers topics such as computation of discharge even when there is no hydrological information in the target area, the importance of river segments in planning, river channel routing, computation of flow capacity of rivers, the appropriate types of structures on specific conditions, the alignment of structures, and others.

The supplemental volumes covering the specific discharge curve, the rainfall intensity duration frequency curve and the isohyet of probable 1-day rainfall and the corresponding specific coefficients are included to help the engineers establish the design discharge. Thus, the discharge in any river in the country can be established where there is no rainfall gauge or there is lack of data using the methods discussed in this manual.

The newly formulated Rainfall Intensity Duration Frequency Curves were revised from previous ones, which were attached to the “Technical Standards and Guidelines for Planning and Design, March 2002” by the strict selection of appropriate data.

To facilitate the computation of the needed parameters in planning and design, the Manual on Runoff Computation with HEC-HMS (Hydrological Engineering Center – Hydrologic Modeling System) and the Manual on Non-Uniform Flow with HEC-RAS (Hydraulic Engineering Center – River Analysis System) are included. HEC-HMS and HEC-RAS are programs developed by US Army Corps of Engineers. The former will enable the engineer to compute design discharge even in a basin with bigger catcment area. The latter will guide the engineers to derive the flow capacity of the river as well as identify and determine the river stretch that needs flood control projects.

Hopefully, the manual and the supplements as references will guide the DPWH engineers on flood control planning.

I would like to thank the DPWH engineers and staff for the hard work and dedication in the formulation of this manual.

HIDEAKI FUJIYAMA
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CHAPTER 1 INTRODUCTION

This manual is intended to assist the engineers of the Department of Public Works and Highways (DPWH) in carrying out flood control planning specifically for the improvement of rivers with small and medium size catchment areas. It was developed to enable the engineers to plan in a more contextual and effective manner.

The Philippines lies in the path of tropical cyclones. An annual average of twenty tropical cyclones enter the Philippine Area of Responsibility and out of this, seven to eight directly hit the country causing enormous water-related damages to human lives and properties. According to the reports of the Office of Civil Defense, approximately 1,000 people every year are killed and/or missing due to floods brought about by cyclones.

Among the mandate of the Department of Public Works and Highways, DPWH, as the engineering arm of the government, is to plan, construct and maintain flood control facilities on major or principal river basins to mitigate the hazards and damages of flood disasters. Generally, flood control projects, which constitute about 10-15% of the total DPWH budget, are classified into the following:

(1) Major Flood Control Projects. These are projects along river basins funded under the foreign assistance. The Planning Service undertakes identification and selection, the foreign consultants undertake the project studies and the Project Management Office (PMO) supervises the implementation of these projects.

(2) Small and Medium Scale Flood Control Projects. These projects are along the principal or major river basins under the regular infrastructure funds or congressional initiative funds. The Regional Offices (ROs) and the District Engineering Offices (DEOs) plan and implement these projects.

(3) Emergency Flood Control Projects. These projects are funded under the quick response funds of the Regional Disaster Coordinating Councils (RDCCs) planned and implemented by ROs and DEOs.

The ROs and DEOs tasked to implement small, medium and emergency flood control projects, sometimes implement projects which are ineffective or inappropriate due to improper or lack of planning. Flood control plans and programs are not properly prepared due to limited technical capabilities, unavailability or insufficient hydrological data and limited budget for the purpose.

To suit the prevailing needs of the ROs and DEOs engineers, the coverage of the manual is carefully crafted to include the following topics:

(1) Systematic explanation of flood control planning including design discharge establishment, alternative study of flood control measures and river improvement

(2) Explanation of topography, climate, rivers in the Philippines and flood damage, which are basic information for river engineers

(3) Graphical method to estimate the design discharge of the rivers in the Philippines

(4) Graphical method to prepare probable design hyetograph, which is necessary for design discharge estimation by the runoff computation

(5) Methods to formulate a river improvement plan by non-uniform flow computation.

This manual is applicable for small and medium size rivers in which probable discharge can be estimated from only a single rainfall station and where sedimentation is ignored. However, topic on river segments, which are closely related to sedimentation, is discussed to serve as reference for the design and repair of damaged structures due to sedimentation problems.

Through this manual, engineers can estimate design discharge in any areas of the country even without rainfall station and established Rainfall Intensity Duration Frequency (RIDF) Curve.

Computations of runoff discharge and non-uniform flow analysis using calculators are quite cumbersome and tedious. By the use of computer programs such as Hydrologic Modeling System (HEC-HMS) and Runoff Analysis System (HEC-RAS), calculations can be made easy and systematic. These programs were developed by the Hydrologic Engineering Center of U.S. Army Corps of Engineers and can be downloaded through internet. Supplementary manuals to HEC-HMS and HEC-RAS are prepared separately. PMO-FCSEC provides technical assistance regarding this flood control manual particularly on HEC-HMS and HEC-RAS, maps and guidelines.

Topographic maps are indispensable tools to formulate flood control plan, thus, it is necessary for the ROs and DEOs to have maps within their jurisdiction.

The procedure in flood control planning is enumerated below:

(1) Calculate the discharges corresponding to several flood frequency levels.

(2) Calculate the existing river flow capacities on several control points.

(3) Investigate the flood damages caused by past major floods and develop the relationship between flood discharge and flood damage.

(4) Consider the possibilities of river improvement.

(5) Determine the preliminary river improvement plan.

(6) Evaluate the cost to be incurred in the preliminary river improvement plan. If the preliminary river improvement planning is not realistic, repeat the procedure from step (3). In consideration of the small and medium scale flood control projects, economic evaluation is not discussed in this manual.

(7) Determine the most appropriate plan.

CHAPTER 2 FLOODING IN THE PHILIPPINES

2.1 TOPOGRAPHY

The Philippine archipelago lies in the southeast of the Asia continent between latitude 4º 23' to 21º 25'N and longitude 117º to 127º E. It is composed of more than 7,000 islands and islets as shown in **Figure 2.1**. The Philippine Archipelago is bounded by the South China Sea on the west, by the Pacific Ocean on the east, by Sulu and Celebes Seas on the south and by the Balintang Channel on the north.

The total land area of the country is approximately 300,000 km^2. The archipelago is composed of three geographical regions as follows: Luzon island in the north which is about 105,000 km^2 in area; the Mindanao island in the south with an area of about 95,000 km^2; and the Visayas region, 100,000 km^2 comprising of small islands between Luzon and Mindanao.

Most of the larger islands have narrow mountain ranges such as: the Sierra Madre Ranges that runs along the east coast of Luzon, the Ilocos Ranges along the western coast of northern Luzon, the Cordillera Ranges situated between the Ilocos and Sierra Madre Ranges. In the island of Mindanao, the mountain ranges are found along the east and west coasts and in the north and central part of the island while most of the islands of the Visayas region have mountain ranges through their interiors. In Luzon, the highest peaks rise to over 2,000 meters located in the Cordillera Mountains but the highest peak in the country is Mt. Apo in Mindanao with an elevation of over 2,900 m. In the Visayas, the highest peak is Mt. Canlaon in Negros Island with a height of about 2,400 m. Flat, level plains are found between mountain ranges in the floodplains of many rivers and along the coasts of the islands. The more notable plains are the Central Plain of Luzon, the Cagayan Valley in northeastern Luzon, the valley of Rio Grande in South western Mindanao and the Koronadal Valley in central Mindanao.

Most of the Philippine Islands are clustered in a predominantly north-south direction. In the southwest, two island groupings deviate from this predominant direction: the long, narrow island of Palawan and its offshore islands and, farther south, the approximately 900 small islands of the Sulu Archipelago. Both island groupings extend southwest toward Borneo with the Sulu Sea between them. The Sulu Archipelago includes many coral islands and reefs.

Figure 2.1 Topography

2.2 CLIMATE

2.2.1 Hydrologic Region

Due to its geographical location, the Philippines is influenced by weather-producing systems which occur at various space and time scales. Since the variability of rainfall is more pronounced compared with the variability in temperature, the climate is classified according to the rainfall distribution. Based on Corona's type of classification, the Philippines has two distinct seasons, the wet and dry which are further classified into four types (Figure 2.2) as follows:

Type I – Two pronounced seasons: dry from November to April and wet during the rest of the year.

Type II – No dry season with a very pronounced maximum rainfall from November to January.

Type III– Seasons not very pronounced: relatively dry from November to April and wet during the rest of the year.

Type IV – Rainfall more or less evenly distributed throughout the year.

Rainfall in the Philippines is brought about by different rainfall-causing weather patterns such as air streams, tropical cyclones, the Intertropical Convergence Zone (ITCZ), fronts, easterly waves, local thunderstorm, etc. About 47% of the average annual rainfall in the country is attributed to the occurrence of tropical cyclones, 14% to the monsoons while 39% are due to the effects of the other weather disturbances (Climate of the Philippines, R.L. Kintanar, 1984). The significance of each of these climatic influences varies with the time of the year.

Tropical cyclones are characterized by a low pressure center where winds of varying intensities blow around this center. The tropical cyclones are the most destructive weather disturbances because they are accompanied by strong winds aside from voluminous rains. They contribute largely to the rainfall from May to December and result to annual maximum quantities in many areas of the country especially in Luzon and the Visayas. Tropical cyclones are classified according to maximum winds near the center as follows:

Figure 2.2 Climate

Tropical Depression (TD)	-	winds from 45 to 63 KPH
Tropical Storm (TS)	-	winds more than 63 to 117 KPH
Typhoon (T)	-	winds of more than 117 KPH

The ITCZ is a series of cloudiness that oscillates from May to October. It typically appears in the southwestern portion of the archipelago in May and moves north reaching its northernmost position in July or August. It begins moving back southward in August moving south of the Philippines by November and its southernmost position in January and February. Several floods in Mindanao are caused by the ITCZ.

The monsoons are wind flows coming from the northeast or southwest. The Northeast and Southwest monsoons trigger the onset and recession of the rainy season in the Philippines. The Southwest monsoon may begin as early as mid April and end as late as early November depending on location while the Northeast monsoon may affect the country from November to March. The eastern coastal areas have a marked rainy season from October to March when the Northeast monsoon is dominant. During the period from May to October when the Southwest monsoon and tropical cyclone seasons are dominant, the western coastal areas receive heavy rainfall that may trigger flooding and landslides.

Topography also modifies the climate of a locality. In general, the windward side of a mountain receives more precipitation due to orographic effect compared with the leeward side. Maximum rainfall is also observed at intermediate elevations. When a rapid change in elevation occurs, a “splash effect” usually occurs and result to significant rainfall. The “splash effect” is a phenomenon that result when cold air moves down slope from precipitating clouds and causes the formation of another cloud at a lower elevation.

Another weather disturbance that affect the eastern sections of Philippines from November to late April or early May is the cold front. The cold front coupled with topography; produce rainfall along the eastern coasts and occasionally over the middle and western portions of the islands.

Other rainfall-producing systems are the easterly waves, which are frequent in summer and affect the mountainous and eastern coastal areas.

Thunderstorms are localized or small-scale disturbances that produce considerable amount of rainfall and occur over a relatively short period of time.

2.2.2 Annual rainfall distribution

Fig. 2.3 shows the average annual distribution in the Philippines. Annual rainfall amounts of more than 4,000 mm are experienced over eastern Samar, northeastern Mindanao, the Mountain Province and western Panay Island. The high values of annual rainfall are due to the influence of the exposure and topography of the area. Areas having annual averages of less than 2,000 mm are mostly situated in valley or plains or in areas that are shielded from the prevailing air streams by high mountains.

2.3 RIVERS IN THE PHILIPPINES

The National Water Resources Council (NWRC) identified the principal and major river basins nationwide in the "Principal River Basins of the Philippines" October, 1976. The principal river basins are defined as those having at least 40 km^2 of drainage area. The river basins with area of at least 1,400 km^2 among the principal river basins are classified as major river basins. There are 421 principal river basins identified with catchment area ranging from 40 to 25,469 km^2 including 18 major river basins.

The 18 major river basins are tabulated in **Table 2.1**.

Table 2.1 18 Major River Basins

River Basin	Region	Catchment Area (km^2)	Code No.
Cagayan	Region II	25,469	02001
Mindanao	Region XI and XII	23,169	12342
Agusan	Region XIII	10,921	10315
Pampanga	Region III	9,759	03059
Agno	Region III	5,952	03070
Abra	Region I	5,125	01036
Pasig-Laguna Bay	NCR and Region IV-A	4,678	04076
Bicol	Region V	3,771	05114
Abulug	Region II	3,372	02028
Tagum-Lubuganon	Region XI	3,064	11303
Ilog-Hilabangan	Region VI and VII	1,945	06235
Panay	Region VI	1,843	06197
Tagoloan	Region X	1,704	10331
Agus	Region XII and ARMM	1,645	12336
Davao	Region XI	1,623	11307
Cagayan	Region X	1,521	10332
Jalaur	Region VI	1,503	06205
Buayan-Malungun	Region XI	1,434	11364

Figure 2.3 Annual Rainfall

2.4 FLOOD DAMAGE

The geographical location of the Philippines makes it one of the most disaster-prone countries in the world. On the average, the country is affected by about twenty (20) tropical cyclones a year, seven or eight of which cross land and inflict considerable damages to lives and properties. Other rainfall producing phenomena include the monsoons, and other wind system that wreak havoc and disrupt life. These weather disturbances are accompanied by wind forces that cause storm surges and heavy rainfall resulting in inundation of river basins and low-lying areas including erosion and slope failures. **Table 2.2** lists the annual statistics of population affected, casualties and damaged houses including the estimated value of damages by typhoons and floods.

For the past twenty one years from 1980 to 2001, a total of 23,942 human lives were claimed or an annual average of 1,088 persons because of typhoon and flood.

Table 2.2 Summary of Typhoon and Flood Damage

Year	Population Affected		Causalities			House Damaged		Damage
	Families	Persons	Dead	Missing	Injured	Totally	Partially	Value *
1980	248164	1666498	36	4	55	16510	51101	1472
1981	250,325	1,472,417	484	264	1,922	44,994	159,251	1,273
1982	266,476	1,569,017	337	223	347	84,027	97,485	1,754
1983	140,604	747,155	126	168	28	29,892	85,072	523
1984	741,510	4,048,805	1,979	4,426	732	310,646	313,391	416
1985	318,106	1,643,142	211	300	17	8,204	211,151	3
1986	287,240	1,524,301	171	43	155	3,162	14,595	1,838
1987	464,162	2,591,914	1,020	213	1,455	180,550	344,416	8,763
1988	1,173,994	6,081,572	429	195	468	134,344	585,732	8,675
1989	501,682	2,582,822	382	89	1,088	56,473	184,584	4,494
1990	1,265,652	6,661,474	676	262	1,392	223,535	636,742	11,713
1991	150,894	759,335	5,101	1,256	292	15,458	83,664	74
1992	418,964	2,097,693	145	95	51	3,472	8,342	7,359
1993	1,523,250	8,202,118	814	214	1,637	166,004	456,773	25,038
1994	670,078	3,306,783	266	54	260	58,869	226,291	3,401
1995	1,710,619	8,567,666	1,255	669	3,027	294,654	720,502	57,781
1996	260,581	1,254,989	124	49	97	2,690	17,557	10,109
1997	777,997	3,954,175	199	28	66	13,225	53,980	4,842
1998	1,590,905	7,197,953	498	116	873	137,020	406,438	17,823
1999	270,424	1,281,194	56	3	25	144	687	1,555
2000	1,426,965	6,852,826	338	59	370	24,573	195,536	7,217
2001	756,938	3,629,295	431	134	418	14,899	54,422	6,924
Total	15,215,530	77,693,144	15,078	8,864	14,775	1,823,345	4,907,712	183,047
Average	691,615	3,531,506	685	403	671	82879	223,078	8,320

* Unit Million Pesos   Source : Office of the Civil Defense

Table 2.3 summarizes the tropical cyclones that crossed the country and leaving at least 100 persons dead and missing.

Table 2.3 Destructive Tropical Disturbance and Corresponding Casualties

Tropical Disturbance	Date of Occurrence	Casualties		
		Dead	Missing	Injured
T Ruping	Nov 10-14, 1990	508	246	
TS Uring	Nov 2-6, 1991	5,101	1,256	292
TD Ditang	July 17-21, 1992	36	77	
T Kadiang	Sep 30-Oct 7, 1993	126	26	37
T Monang	Dec 3-4, 1993	273	90	607
T Puring	Dec 24-29, 1993	187	52	280
TS Mameng	Sep 27 - Oct 1, 1995	116	126	49
TS Pepang	Oct 26 -30, 1995	265	67	323
T Rosing	Oct 31 - Nov 3, 1995	936	316	4,152
T Emang & TS Gading	Sept 16-21, 1998	108	20	
T Loleng	Oct 15-23, 1998	303	29	751
T Reming	Oct 26-Nov 1, 2000	114	10	
T Feria	July 2-6, 2001	188	44	241
T Nanang	Nov. 6-10, 2001	236	88	169
Total		8,497	2,447	6,901

In terms of the affected river basins/ watersheds, **Table 2.4** shows a breakdown of casualties for ten (10) river basins, in which most casualties are recorded during the occurrence of destructive tropical disturbances. The worst hit watershed for the past ten (10) years is the Anilao-Malbasag in Ormoc City, which lost more than six thousand (6,000), lives with the passage of TS Uring in 1991.

Table 2.4 List of River Basins with Records of Most Casualties

Tropical Disturbance	Date of Occurrence	Anilao-Malbasag	Bicol	Ilog-Hilabangan	Camiguin	Pampanga	Panay	Agno	Cagayan	Agusan	Tagoloan
T Ruping	Nov 10-14, 1990	26		222			58			11	47
TS Uring	Nov 2-6, 1991	6,161		148							
TD Ditang	July 17-21, 1992					15		18			
T Kadiang	Sep 30 - Oct 7, 1993					41		23	66		
T Monang	Dec 3-4, 1993		202						1		
T Puring	Dec 24-29, 1993			9			31			44	
TS Mameng	Sep 27 - Oct 1, 1995					103	101	17	1		
TS Pepang	Oct 26 -30, 1995			136							
T Rosing	Oct 31 - Nov 3, 1995		377			10			2		
T Emang & TS Gadin	Sept 16-21, 1998					8		101	4		
T Loleng	Oct 15-23, 1998		115			35	9	1	8		
T Reming	Oct 26-Nov 1, 2000		8								
T Feria	July 2-6, 2001					6	2	31	9		
T Nanang	Nov. 6-10, 2001			32	220		7				
Total		6187	702	547	220	218	208	191	91	55	47

Source : Office of Civil Defense

CHAPTER 3 FLOOD CONTROL PLANNING

3.1 NECESSITY OF FLOOD CONTROL PLAN

Whenever the catchment area and/or flood prone area to be considered is big or very important, and when the flood safety level is not balanced between upstream and downstream portions of the river, it should have a flood control plan. A flood control plan should be formulated from the basin-wide view point and should require proper coordination with the other plans such as:

- Irrigation development plan
- Road network/bridge plan
- Sabo plan
- Environmental management plan

3.2 DESIGN FLOOD FREQUENCY

Magnitude of flood events has theoretically no upper limit. The flood control work may be implemented targeting flood event with high magnitude say, safety level with higher return period, but in this case, construction cost is too expensive and will be difficult to implement because of limited funds, hence the whole areas with flood problems cannot be totally protected. The return period should be determined based on the size of catchment area, the degree of importance of the proposed project area and the economic viability of the project. Thus, it is necessary to determine the design flood discharge corresponding to the design flood frequency of the river. It is also necessary to consider the funds needed for the implementation of the proposed improvement works and the expected benefits.

There are two (2) methods to determine the return period of the design discharge of the project. In the first method, the safety level against flood is determined first and necessary measures are studied from engineering, socio-economical and environmental viewpoints. In this method, the return period applied to projects to protect areas and /or basin with similar socio-economic importance is considered. The socio-economic importance is evaluated by indices such as population, degree of urbanization, assets, etc. This method is mostly applied for Master Plan.

The other method is to find realistic measures for project implementation plan. This method is to select the return period that maximizes the benefit and cost ratio or internal rate of return of the project.

Table 3.1 summarizes design flood frequency on the foreign assisted flood control or studies..

Table 3.1 Sample of Design Flood Frequency

River Name	CA (Km^2)	Total Pop. (1000)	Flooded Area (km^2)	Affected Pop. (1000)	Design Flood Frequency (yr return period)		Code
					MP/Long Term	Short Term	
Cagayan	27,280	2,136	1,860		25	25	02001
Mindanao	23,169				100	100	12342
Agusan	11,400	134*	79*	115*	100	100	10315
Pampanga	10,503	1,792*	1,448*		100	100	03059
Agno	5,397	2,324	2,465	1,447	25	25	03070
Pasig-Marikina	1,519	5,926*	110*	1,100*	100	100	04076
Bicol	3,132	821	438		100	100	05114
Ilog-Hilabangan	2,162	347	120	47	100	100	06235
Panay	2,181	448	338	121	25	25	06197
Tagoloan	1,778	135	12.5		50	50	10331
Jalaur	1,742	394	108		100	100	06205
Laoag	1,332	197	202	79	25	25	01034
Amnay/ Patric (Ibod)	586/ 407				50/ 50	50/ 50	04157/ 04410
Iloilo/ Jaro (Iloilo City)	93.1/ 412.1	310	41	149	50	50	06210
Anilao/ Malbasag	25.2/ 11.1	129	-	7,922**	50	20	
Sacobia-Bamban/ Abacan/ Pasig-Potrero	1,296	736*	393*	205*	-	20	

Note Figures with asterisk * show those in target area.. Figures of Ormoc with ** are persons dead and missing.

3.3 PROCEDURES FOR THE FORMULATION OF FLOOD CONTROL PLAN

(1) Data and Information Collection

Following data and information should be collected.

(a) Topographic Information

(i) Topographic maps

Topographic maps with a scale of 1:50,000 should be used for basin delineation except for Metro Manila where maps with a scale of 1:10,000 are available.

(ii) Land Use Maps

Land use map is necessary to estimate runoff coefficient for the future land use condition and estimated flood damage. This is available from the municipalities covering the target river basin.

(iii) Other information such as aerial photographs, geological maps, etc.

(b) River information

(i) Previous surveying data (cross-section)

(ii) Quarry data

(c) Hydrological Information

(i) Specific Discharge Curve

Specific discharge curves are included in **Attachment 4.1** of the supplementary Specific Discharge, Rainfall Intensity Duration Curve and Isohyet of Probable 1-day Rainfall Manual.

(ii) Rainfall Data

The location maps of climatic and synoptic PAGASA stations, which can be selected for application for runoff calculation, are indicated in **Attachment 4.2** Based on the maps, the selected synoptic station to be selected should be located within or near to the target river basin.

(iii) Rainfall Intensity Duration Frequency (RIDF) Curve

The RIDF of the selected rainfall station is necessary to prepare the design hyetograph for runoff computation. The RIDF of the selected rainfall stations are indicated in **Attachment 4.3**

(iv) Probable Daily Rainfall and Specific Coefficient

When there is no PAGASA synoptic station in or near the target river basin, the design hyetograph can be established by reading the values of isohyets in the maps showing specific coefficient in **Attachment 4.4** and the daily rainfall with different flood frequency in **Attachment 4.5** for all the areas in the Philippines.

(d) Flooding Information

(i) Flood prone areas and causes of flooding

(ii) Flooding conditions such as maximum depth and duration of flooding

(iii) Flood losses and damages

(e) Socio-economic Information

Collect socio-economic information in the flood prone area for economic evaluation of the project.

(i) Population by barangay

(ii) Statistics of commercial and industrial data by barangay, city and/or municipality.

(iii) Statistics of agricultural data by barangay, city and/or municipality.

(iv) Future development plan by region, city and/or municipality

(2) Flood Control Plan

(a) Field Survey

(i) River cross-sectional survey

Table 3.2 River Cross-Sectional Survey Interval

Survey Item	**River Width**		
	Less than 50 m	50 – 200 m	200 m or more
Cross-section			
Scale			
Horizontal	1:200-1:500	1:500 or 1:1,000	1:1,1000
Vertical	1:100	1:100 or 1:200	1:200
Interval between sections			
Strait, uniform	50 m	100 m	200 m
Strait, irregular	50 m	50 m	200 m
Bend	25 m	50 m	100 m
Inland survey width	25 m	50 m	50 m

For the master plan, interval of cross-sectional survey can be extended as much as 500 m.

(ii) Longitudinal profile based on the cross-sectional survey

(iii) Bed material survey to identify river segment

(v) Inventory of the existing river facilities

(vi) History of flood control activities in the basin

(vii) Possible location of dam and retarding basin

(b) Analysis

(i) Existing River Features

- Average riverbed profile
- River segment
- Location of scouring
- Changes of river course and longitudinal profile, etc

(ii) Probable Discharge

- Delineation of catchment area

- Estimation of probable discharge by specific discharge or runoff analysis, which is made by rational formula or by unit hydrograph method

(iii) Flow Capacity of Existing River Channel:

- Identification of bank height considering freeboard
- Estimation of flow capacity by uniform flow or non-uniform flow computation using the probable discharge obtained above

(c) Design Discharge

In consideration of the importance of the basin of the target river system, the design flood frequency is determined and the design discharge hydrograph is established at the reference points.

(d) Alternative Study

The design discharge is allotted to the improved river channel, the possible dam/reservoir, the possible retarding basins and other flood control measures such as flood way. Alternatives are composed of several combinations of different size/capacity of the flood control facilities, all of which can accommodate the design discharge.

(e) Evaluation of Alternative Plans

The cost of respective alternative plans is estimated based on the preliminary design. The benefit of the project is estimated from flood damage analysis. The project is evaluated to determine the economic viability by comparing the required investment against the benefits that can be derived from the project based on the preliminary implementation plan and expected life of the project. The social and the environmental impacts are also assessed.

(f) Formulation of Optimum Plan

Based on the evaluation, the optimum plan is selected and implementation plan is formulated.

CHAPTER 4 DESIGN DISCHARGE

4.1 PROCEDURE IN THE DETERMINATION OF THE DESIGN DISCHARGE

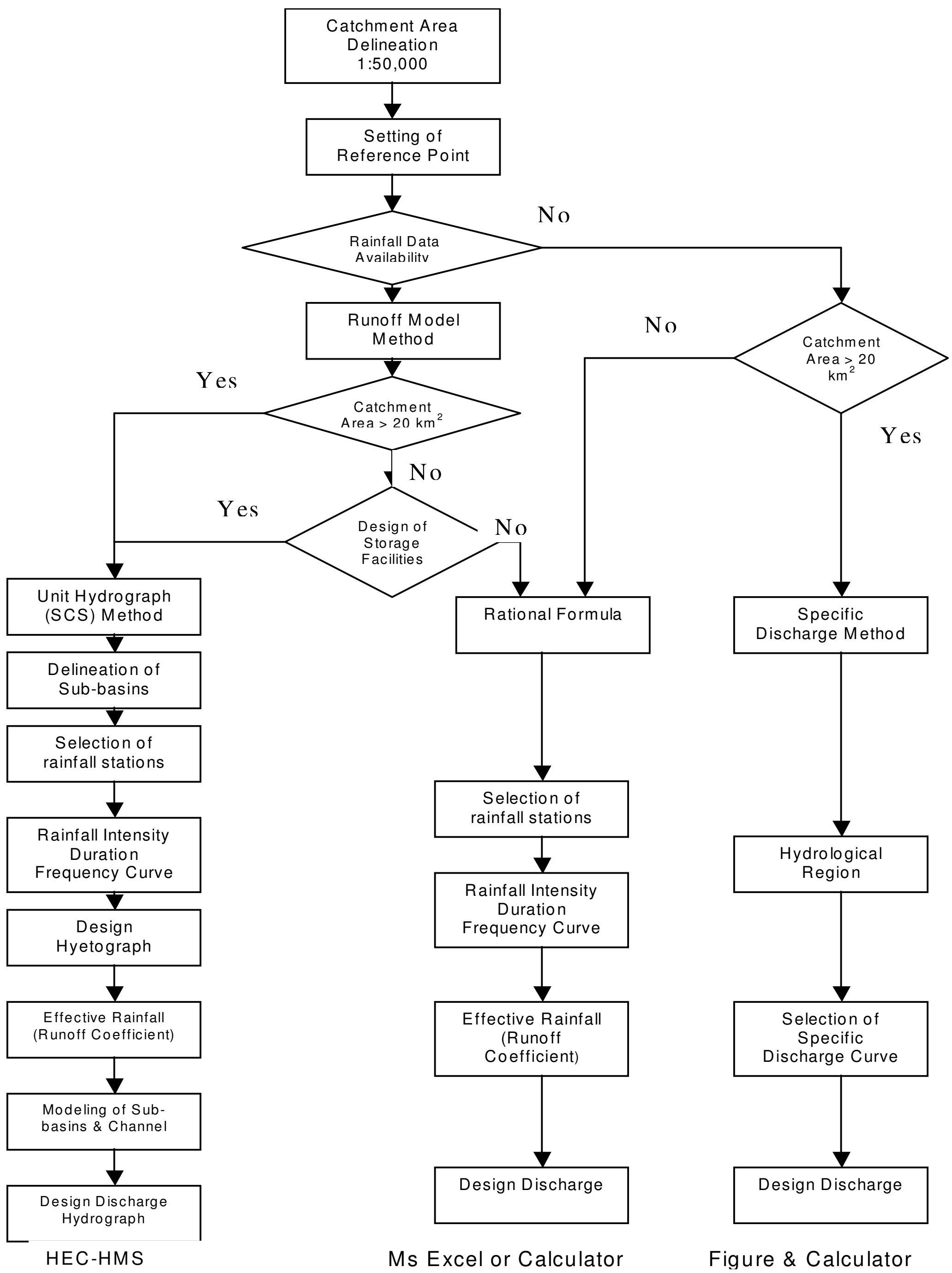


Figure 4.1 Flowchart of Design Discharge Estimation

The design discharge can be estimated by 1) specific discharge method or 2) runoff model using rainfall data through the procedure described in **Figure 4.1**.

The specific discharge curve explains the relationship between the flood peak discharge per unit catchment area ($m^3/s/km^2$), otherwise called as the unit discharge (ordinate), and size of catchment (abscissa). Using this curve, design discharge is roughly determined without any runoff analysis. Also, the reliability of the design discharge, which is estimated by runoff model, can be easily assessed by comparing it with other design discharges

Design discharge is an important input in deciding the appropriate types of countermeasures to be adopted in flood control planning and for the structural design of such countermeasures.

In the runoff model method, the design discharge is determined from the probable rainfall. A lot of runoff models have been proposed and applied. In the Philippines, storage function model, quasi-linear model, tank model and unit hydrograph method, to name a few, have been applied for middle to large river basins, while the rational formula is used for small river basins and drainage system. In consideration of the practice in the Philippines and easy usage, this manual recommends to apply the following runoff models

(1) The Rational formula: Catchment area of the target river basin is equal to or less than 20 km^2 (It is allowed for area less than 100 km^2 as reference).

(2) The Unit Hydrograph method: Catchment area for the target river basin is more than 20 km^2. When storage facility such as retarding basin is required to be planned, the unit hydrograph method is applied even for smaller river basin.

4.2 CATCHMENT AREA

The catchment area or basin boundary is delineated as a perpendicular curve (polygon) to the contour lines, using the latest edition of topographic map with a scale of 1:50,000 prepared by the National Mapping and Resource Information Administration (NAMRIA). Afterwards, the catchment area is computed using the following methods:

(1) by the use of planimeter

(2) by triangulation

(3) by cross-section mm paper

(4) by AutoCAD / GIS software

4.3 REFERENCE POINT

At least one reference point should be established in a river system. At the reference point, the safety level for the river system is determined and therefore, the reference point should be located just upstream or neighboring of a city/ a town which is the most important to be protected from flooding. At this point, water level and discharge data should be collected to analyze flood discharge and probable discharge as the basis for the flood control planning.

4.4 SPECIFIC DISCHARGE

Attachment 4.1 indicates specific discharge curves (Creager type) for 2-yr, 5-yr, 10-yr, 25-yr, 50-yr and 100-yr return period, for which the Philippines is divided into three (3) regions, namely Luzon, Visayas and Mindanao. **Table 4.1** indicates constants of the Creager type specific curve for the following equation.

$$q = c \cdot A^{(A^{-0.048} - 1)}$$

Where, $\boldsymbol{q}$ = specific discharge ($m^3/s/km^2$)

$\boldsymbol{c}$ = constant (**Table 4.1**)

$\boldsymbol{A}$ = catchment area (km^2).

Table 4.1 Constants for Regional Specific Discharge Curve

Region	Return Period					
	2-year	5-year	10-year	25-year	50-year	100-year
Luzon	15.66	17.48	18.91	21.51	23.83	25.37
Visayas	6.12	7.77	9.36	11.81	14.52	17.47
Mindanao	8.02	9.15	10.06	11.60	12.80	14.00

Using the specific curve or equation, design discharge or probable discharge is obtained as follows.

(1) Obtain catchment area (*A*) for which the discharge is necessary.

(2) Determine the return period or safety level.

(3) Read specific discharge from the relevant curve of **Attachment 4.1** corresponding to region, return period and catchment area as shown in **Figure 4.2**, or compute specific discharge from the equation, using catchment (*A*) and constant (*c*) corresponding to region and return period.

(4) Obtain specific discharge (*q*) as the product of the specific discharge in (3) and catchment area (*A*).

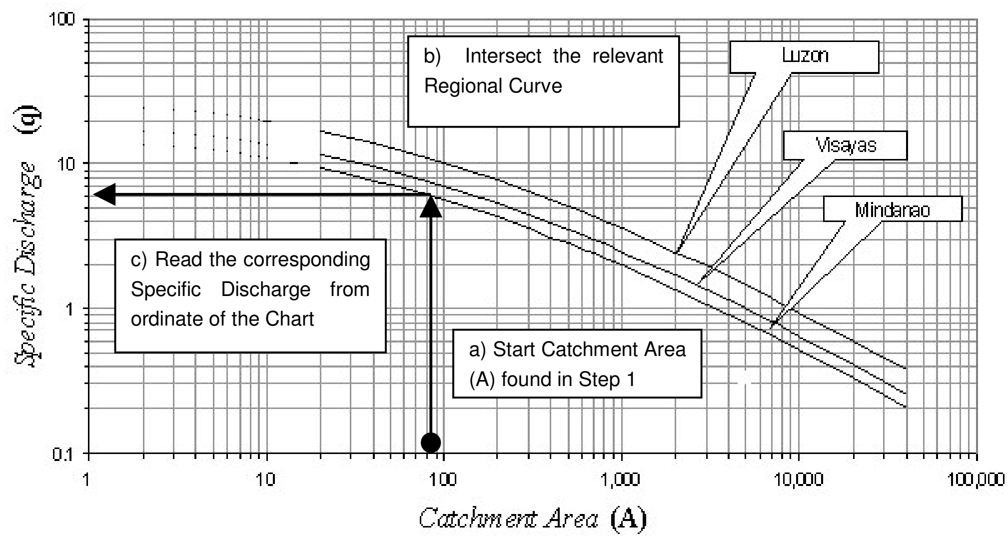


Figure 4.2 Illustration to Read Specific Discharge

Referring to the formula for finding q and Table 4.1 Constants for Regional Specific Discharge Curve, the values for q are obtained for Luzon, Visayas and Mindanao with equivalent values similar to Figure 4.2.

LUZON	c (constant)	Specific Discharge (q) ($m^3/s/km^2$)			
		20 km^2	100 km^2	1,000 km^2	10,000 km^2
2-year	15.66	10.484	6.283	2.229	0.583
5-year	17.48	11.703	7.013	2.488	0.651
10-year	18.91	12.660	7.587	2.692	0.704
25-year	21.51	14.401	8.630	3.062	0.801
50-year	23.83	15.954	9.560	3.392	0.887
100-year	25.37	16.985	10.178	3.612	0.944

VISAYAS	C (constant)	Specific Discharge (q) ($m^3/s/km^2$)			
		20 km^2	100 km^2	1,000 km^2	10,000 km^2
2-year	6.12	4.097	2.455	0.871	0.228
5-year	7.77	5.202	3.117	1.106	0.289
10-year	9.36	6.266	3.755	1.332	0.348
25-year	11.81	7.907	4.738	1.681	0.440
50-year	14.52	9.721	5.825	2.067	0.540
100-year	17.47	11.696	7.009	2.487	0.650

Minadanao	C (constant)	Specific Discharge (q) ($m^3/s/km^2$)			
		20 km^2	100 km^2	1,000 km^2	10,000 km^2
2-year	8.02	5.369	3.218	1.142	0.298
5-year	9.15	6.126	3.671	1.303	0.341
10-year	10.06	6.735	.036	1.432	0.374
25-year	11.60	7.766	4.654	1.651	0.432
50-year	12.80	8.570	5.135	1.822	0.476
100-year	14.00	9.373	5.617	1.993	0.521

Note

Specific Discharge Curves are formulated based on the studies of the major river basins nationwide. Therefore, for small or medium basins there are no sufficient data. Specific Discharge Method is applicable only to catchment areas with more than 20 km^2, otherwise Rational Formula is recommended.

4.5 RUNOFF MODEL

4.5.1 Rational Formula

The Rational Formula is applicable to catchment areas smaller than 20 km^2.

(1) Basic Equation

The idea behind the Rational Formula Method is that if a rainfall of intensity (*i)* begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration (t_c), when all of the watershed is contributing to the flow at the outlet point or point under consideration.

$$Q_p = \frac{ciA}{3.6}$$

Where:	Q_p	=	maximum flood discharge (m^3/s)
	c	=	dimensionless runoff coefficient
	i	=	rainfall intensity within time t_c
	A	=	catchment area (km^2)

The assumptions associated with the Rational Formula Method are:

· The computed peak rate of runoff at the outlet point is a function of the average rainfall rate during the time of concentration, i.e., the peak discharge does not result from a more intense storm of shorter duration, during which only a portion of the watershed is contributing to the runoff at the outlet.

· The time of concentration employed is the time for the runoff to become established and flow from the most remote part of the drainage area to the outlet point.

· Rainfall intensity is constant throughout the rainfall duration.

The required data to apply the Rational Formula is obtained as follows.

(2) Runoff Coefficient (c)

The runoff coefficient *(c)* is the least precise variable of the Rational Formula. Its use in the formula implies a fixed ratio of peak runoff rate to rainfall rate for the catchment area, which in reality is not the case. Proper selection of the runoff coefficient requires judgment and experience on the part of the hydrologist/engineer. The proportion of the total rainfall that will reach the river and/or storm drains depends on the percent imperviousness, slope and ponding characteristics of the surface. Impervious surface, such as asphalt pavements and roofs of buildings, will produce nearly 100% runoff after the surface has become thoroughly wet, regardless of the slope.

Field inspection and aerial photographs as well as the present land use maps are useful in estimating the nature of the surface within the target basin.

Runoff coefficient will increase with urbanization in the target basin due to increase of impervious surface and installation of drainage system. Therefore, in case where a large-scale development is planned, runoff coefficient after development should be used to determine the design discharge so that the expected safety level will be kept after development. The future land use plan can be obtained from the LGUs.

Tables 4.2 and **4.3** tabulate runoff coefficients described in the Design Guidelines Criteria and Standards, Vol. 1 MPWH 1987 (Red Book) and applied for the Metro Manila.

Table 4.2 Runoff Coefficient Used in the Philippines

Surface Characteristics	Runoff Coefficient
Lawn, gardens, meadows and cultivated lands	0.05 – 0.25
Parks, open spaces including unpaved surfaces and vacant lots	0.20 – 0.30
Suburban districts with few building	0.25 – 0.35
Residential districts not densely built	0.30 – 0.55
Residential districts densely built	0.50 – 0.75
Watershed having steep gullies and not heavily timbered	0.55 – 0.70
Watershed having moderate slope, cultivated and heavily timbered	0.45 – 0.55
Suburban areas	0.34 – 0.45
Agricultural areas	0.15 – 0.25

Source: Design Guidelines Criteria and Standards, Volume 1, MPWH, 1987

Table 4.3 Runoff Coefficient Applied for Metro Manila

Surface Characteristics	Runoff Coefficient
Low (Urban Area)	0.5
Middle (Urban Area)	0.65
High (Urban Area)	0.8
Factory	0.5
Open Space	0.35
Paddy	0.1
Farm land	0.3
Mountain	0.8

Study on Flood Control and Drainage Project in Metro Manila, JICA 1990

After the existing or the future land use figure or data is obtained, the land use is classified according to the categories to be applied and each area is measured to obtain the percentage to the total catchment area. Using the percentage of each area, the average runoff coefficient is obtained as the weighted average.

Table 4.4 Example of Percentage of Land Use Category

Total area of the Sub-Basin 22.8 km^2	Urban area			Factory Area	Open space	Total
	Low density	Middle density	High density			
Area by land use (km^2)	17.35	3.85	0.62	0.21	0.80	22.83
Percentage (%)	76.0 %	16.9 %	2.7 %	0.9 %	3.5 %	100 %

Average c =(76.0x 0.5 + 16.9 x 0.65 + 2.7 x 0.8 + 0.9 x 0.5 + 3.5 x 0.35)/100
= 0.53

[Example] Computation of the average runoff coefficient (c).

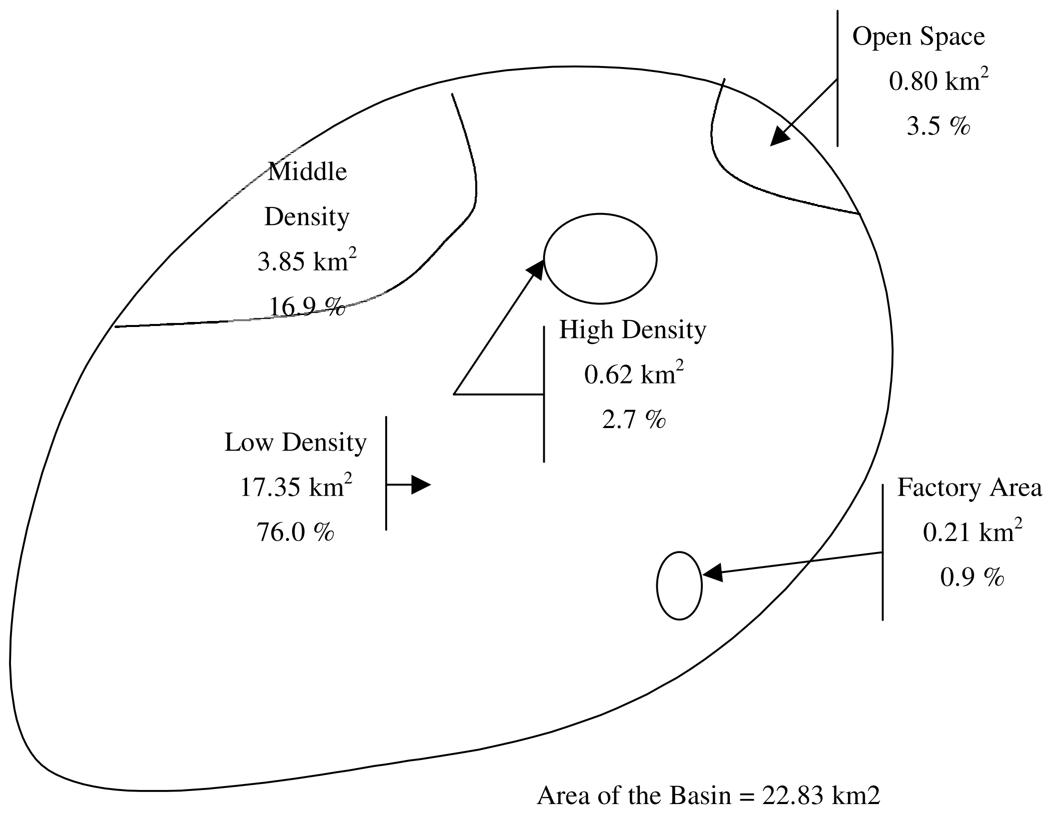


Figure 4.3 Example of Land Use Category

Average c = [17.35 km^2 x 0.50 + 3.85 km^2 x 0.65+ 0.62 km^2 x 0.80 + 0.21 km^2 x 0.50 + 0.80 km^2] / 22.83 km^2

= 0.53

(3) Time of concentration

The time of concentration (t_c) for the catchment area under consideration is obtained as follows.

$$t_c = t_i + t_f$$

Where: t_i = inlet time

= time it takes for flow from the remotest point to the inlet point or farthest point of river channel

t_f = flow time

= time it takes from the inlet point or farthest point of the river channel to the outlet point or point under consideration

= L / V

L = Length of river channel from its outlet point to its farthest point (m)

V = flow velocity (m/s)

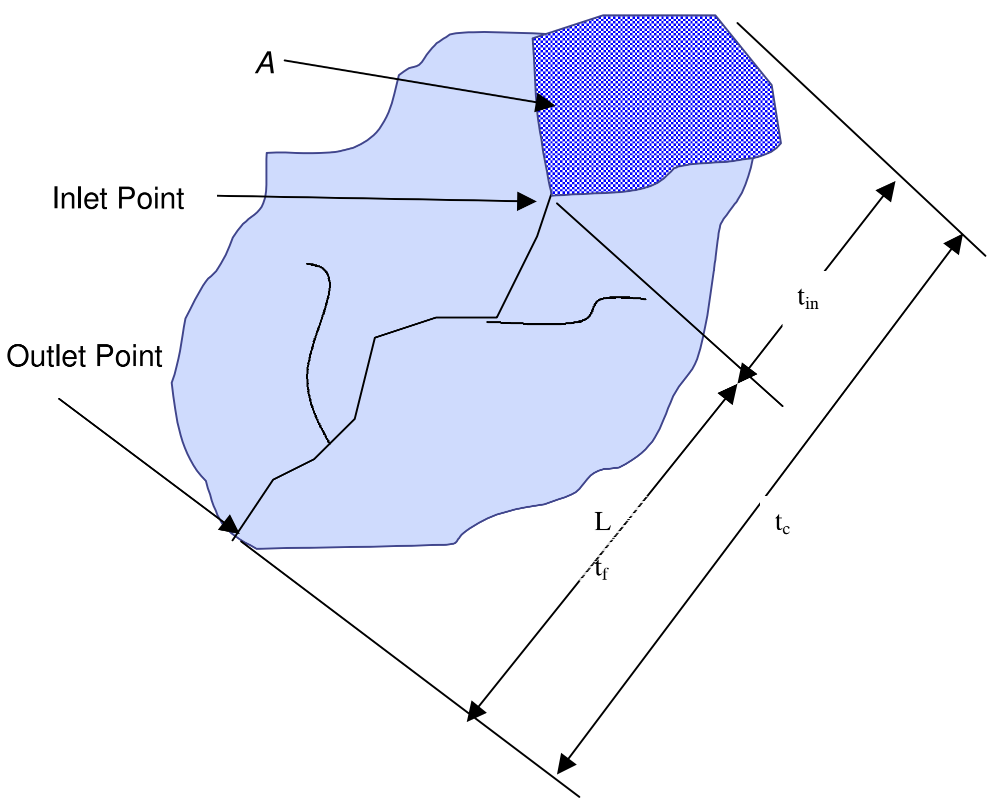


Figure 4.4 Inlet and Outlet Points of Rational Formula

(a) Inlet Time

Inlet time is computed as follows.

(i) Find the inlet point, the catchment area of which is 2 km^2 (**Figure 4.4**)

(ii) Set the inlet time as follows.

$t_i = 30$ min.

(iii) When the catchment area (A) of the farthest point of the channel is clearly judged to be less than 2 km^2, compute the inlet time (min.) from A (km^2) as follows

$$t_i = \frac{30\sqrt{A}}{\sqrt{2}}$$

(b) Flow Time

Flow time is computed from Kraven's Formula (**Table 4.5**), which gives relations between slope of water course and flow velocity as shown below.

Table 4.5 Kraven's Formula

Riverbed gradient (I_b)	I_b > 1/100 (steep slope)	1/100 > I_b > 1/200	I_b < 1/200 (mild slope)
Flow velocity (m/s)	3.5	3.0	2.1

[Example] Computation of Flow Time

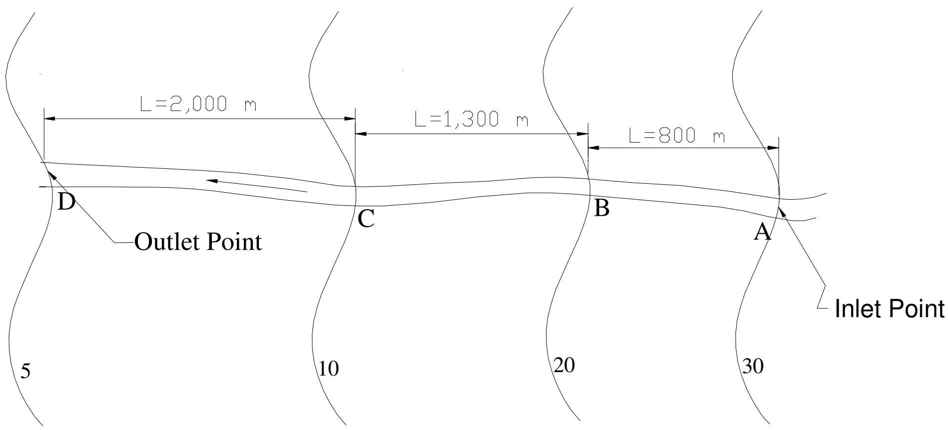


Figure 4.5 Sample River with Contour Lines

1. Find the intersection of river and contour line from the map (As A, B, C, D in **Figure 4.5)**

2. Measure the distance from A to B, B to C and C to D.

3. Compute the gradient using the following formula

$$Gradient = \frac{(ElevationA) - (ElevationB)}{(Dis\tan cefromAtoB)}$$

4. Compute for the flow time

   - Flow velocity

$$I_{A-B} = Gradient_{A-B} = \frac{30-20}{800} = \frac{10}{800} = \frac{1}{80}$$

$$I_{B-C} = Gradient_{B-C} = \frac{20-10}{1300} = \frac{10}{1300} = \frac{1}{130}$$

$$I_{C-D} = Gradient_{B-C} = \frac{10-5}{2000} = \frac{5}{2000} = \frac{1}{400}$$

$$I_{A-B} > \frac{1}{100}$$ Flow velocity is 3.5 m/s

$$\frac{1}{100} > I_{B-C} > \frac{1}{200}$$ Flow velocity is 3.0 m/s

$$I_{C-D} < \frac{1}{200}$$ Flow velocity is 2.1 m/s

- Flow time

$$t_f = \frac{800m}{3.5m/s} + \frac{1300m}{3.0m/s} + \frac{2000m}{2.1m/s} = 1{,}614\sec$$

= 26 min 54 sec

(4) Rainfall Intensity

The rainfall intensity *i* is the average rainfall rate in mm/hr with the decided safety level indicated in the form of return period for the considered catchment area during the concentration time.

The rainfall intensity is obtained from the rainfall intensity duration frequency (RIDF) curve. Two (2) types of RIDF curve are available for the runoff analysis.

(a) RIDF of PAGASA Synoptic Rainfall Station

PAGASA operates/maintains 52 Synoptic stations equipped with automatic rainfall gauge. **Attachment 4.2** indicates location of the selected **39** out of 52 PAGASA Synoptic stations, which are recommended to be used for runoff analysis. When one of the stations is located inside or near the target river basin, the RIDF of this station is used to obtain rainfall intensity during the concentration time. The RIDFs of the selected Synoptic PAGASA stations are shown in **Attachment 4.3**.

RIDF can be expressed as follows

Type 1: $R = \frac{A}{(C + T^{b})}$ Short duration (10 min – 1 hr)

Type 2: $R = \frac{A}{(C + T)^{b}}$ Long duration (1 hr – a day)

and best-fit equations for the selected station are indicated in **Attachment 4.3** with return period of 2-yr., 3-yr., 5-yr., 10-yr., 25 yr., 50 yr. and 100 yr.

RIDF is separated into 1) RIDF short duration (10 min to 1 hour) and 2) long duration (1 hour to 1 day) and RIDF corresponding to the concentration time should be selected.

[Example] The above RIDF is expressed for Vigan.

For 2 yr return period using 6 hr. $R = \frac{3321.85}{(35.42 + 360^{0.81})}$ = 21.70 mm

For 10 yr return period using 1 hr: $R = \frac{5948.97}{(19.57 + 60^{0.95})}$ = 86.89 mm

For 100 yr return period using 12 hr : $R = \frac{17210.75}{(155.32 + 720)^{0.92}}$ = 33.81 mm

(b) Specific Coefficient

When no Synoptic station is located near the catchment area, the RIDF for long duration (1 hr. to 1day) can be computed from isohyetal maps of specific coefficient β (**Attachment 4.4**) and 1-day probable rainfall (**Attachment 4.5**). The procedures to obtain RIDF are as follows.

(i) Read representative specific coefficient β of the target river basin from **Attachment 4.4.** The representative point can be the centroid of the river basin.

(ii) Read probable 1-day rainfall corresponding to the n-yr return period from **Attachment 4.5.**

(iii) Compute b from the following equation.

$$b = \frac{\log \beta}{\log 24 - \log 1} = \frac{\log \beta}{1.3802}$$

(iv) Obtain RIDF from the following equation.

$$I_t = \left(\frac{24}{t}\right)^b I_{24}$$

Where, I_t = rainfall intensity for duration t (mm/hr)

I_{24} = 1 day rainfall intensity (mm/hr)

t = hr.

(v) Compute rainfall intensity during the time of concentration from the above equation.

[Example] Application of Specific Coefficient ß

Step 1	**Location**		**Capas Tarlac**			
	Coordinates		1,6952941.14 E		232,920.94 N	
Step 2	**Read R (I_{24}=R/24)**					
	Return Period				**R, mm**	
	2				180	
	5				210	
	10				240	
	25				280	
	50				300	
	100				325	
Step 3	**Read ß**				**7.50**	
Step 4	**Compute *b***				**0.634**	
Step 5	**Compute RIDF, (mm/hr)**					
Return Period	Duration t in hr					
	1 hr	2 hr	3 hr	6 hr	12 hr	24 hr
2-year	56.25	36.25	28.03	18.06	11.64	7.50
5-year	65.63	42.29	32.70	21.07	13.58	8.75
10-year	75.00	48.33	37.37	24.08	15.52	10.00
25-year	87.50	56.38	43.6	28.10	18.11	11.67
50-year	93.75	60.41	46.72	30.10	19.40	12.50
100-year	101.56	65.45	50.61	32.61	21.01	13.54

Step 1 Get the location L of the centriod of the basin of interest.

Step 2 Using the 1-day isohyetal Maps, Attachment 4.5, for various return period at Location L, read the value of rainfall I_{24} in mm for 2 yr, 5 yr, 10 yr and 100 yr return period.

Step 3 Using the Maps of Specific Coefficient ß, Attachment 4.4, read the values of ß at the same location.

Step 4 For every return period, compute b using the equation b = log (ß)/1.3802.

Step 5 Compute RIDF in mm/hr for every duration t using the equation $I_t = \{24/t\}^b x(R/24)$

4.5.2 Unit Hydrograph Method

(1) Methodology

The Unit Hydrograph Method uses the following assumptions:

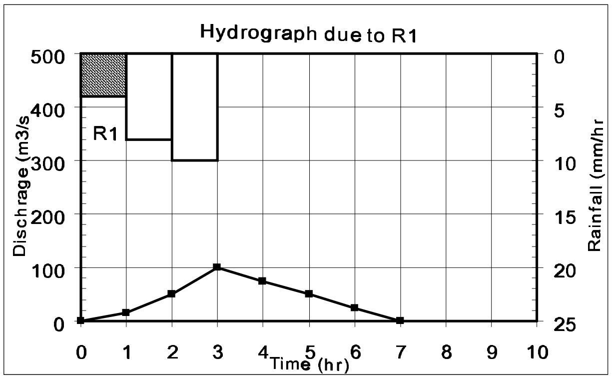
(a) Duration of direct runoff is in direct proportion to the intensity of rainfall with equal constant duration, irrespective of the intensity of that rainfall. In other words, the base length is constant.

(b) Volume of direct runoff is in direct proportion to the intensity of rainfall.

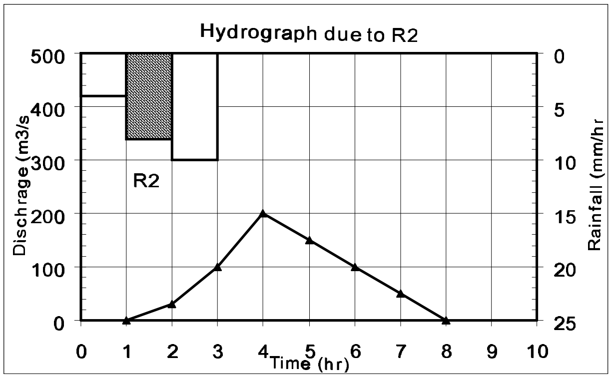
(c) Volume of runoff is to be determined by adding together the run-off components of each rainfall.

This means that the discharge curve at a certain point of a river by the unit effective rainfall which falls in a unit time has always the same form. The discharge curve obtained at that time is called the Unit Hydrograph.

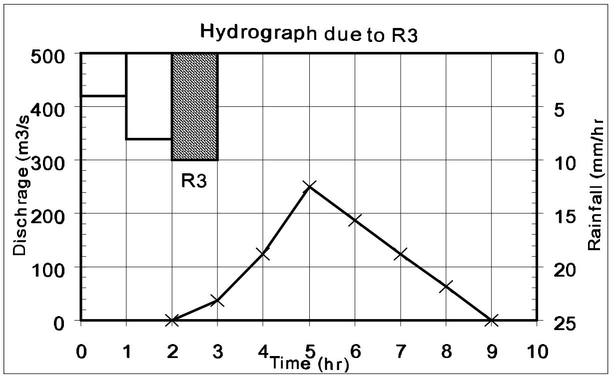
The assumptions above can be best explained by the illustration below (**Figure 4.6**).



The hydrograph is produced by the rainfall R1 with a certain base width.



The hydrograph is caused by R2 with fixed base width and time duration same as the previous one. The volume of direct runoff is proportional to the rainfall. Time delay is observed as R2 occurred after R1.



Similarly, the hydrograph has the same base width and time duration, and the rise follows the occurrence of R3 with proportional volume of runoff.

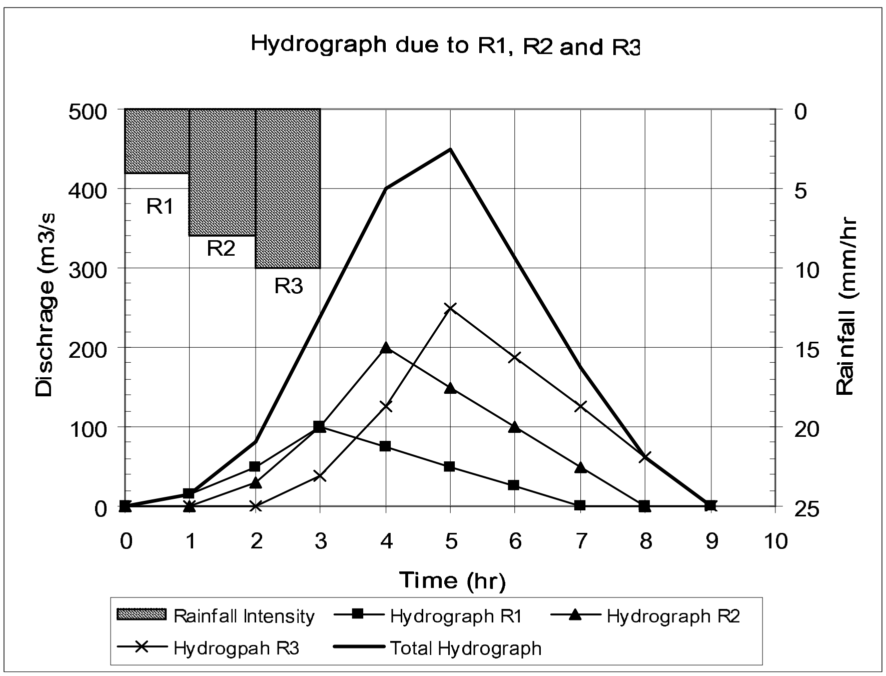


Figure 4.6 Computation of Unit Hydrograph

Collectively, the discharges produced by R1, R2 and R3 are added from the discharges produced by the individual rainfall as shown above.

The general equation for the unit hydrograph in discrete form is as follows:

$$Q_n = \sum_{i=1}^{n} R_i U_{n-i+1}, \quad \text{or}$$

$$Q_n = R_n U_1 + R_{n-1} U_n + R_{n-2} U_3 + \ldots + R_1 U_n ,$$

where Q_n is the storm hydrograph ordinate, R_i, is effective rainfall, and U_j *(j=n-I+1)* is the unit hydrograph ordinate.

Using the above equation, the following computation is applied to obtain the hydrograph indicated above.

Table 4.6 Computation of Direct Runoff by Unit Hydrograph Method

Time (hr.)	Effective Rainfall (mm)	Unit hydrograph ordinate (m^3/s/mm)						Direct Runoff (m^3/s)
		1	2	3	4	5	6	
		3.75	12.5	25.0	18.75	12.5	6.25	
1	4	15						15
2	8	30	50					80
3	10	37.5	100	100				237.5
4			125	200	75			400
5				250	150	50		450
6					187.5	100	25	312.5
7						125	50	175
8							62.5	62.5

(2) SCS Unit Hyetograph

The unit hydrograph method has been applied to many river basins of many countries and based on this method, several synthetic unit hydrographs have been proposed. Synthetic unit hydrograph can be estimated for ungauged river basins by means of relationships between parameters of a unit hydrograph model and the physical characteristic of the river basin. Out of these synthetic unit hydrographs, SCS unit hydrograph, which is proposed by the Soil Conservation Service (SCS, present Natural Resources Conservation Service, NRCS) is recommended to be applied, considering that: 1) shape of the unit hydrograph is easily obtained and 2) this unit hydrograph is applied in many countries.

SCS Synthetic unit hydrograph can be applied to the rivers of the Philippines, but study to revise the parameters of synthetic unit hydrograph should be done based on the rainfall and discharge data of the Philippines.

(3) Delineation of Subbasins

The basin, for which flood control will be planned, is modeled by subbasins and channels for the application of the SCS unit hydrograph, considering 1) reference point, 2) important points in which flood control structures are planned to be constructed and 3) river points to which tributaries join.

Area of subbasins should not be too small, since time interval of computation becomes short. In general, the area of subbasins for unit hydrograph can be extended to the extent of 100 km^2 to 200 km^2.

If there is a drainage area, in which rainfall is drained by pumps, this area should be deducted from the basins/subbasins and planned pump discharge is added to the design discharge of the river.

Figure 4.7 shows a sample of the San Juan River Basin, the tributary of the Pasig Marikina River. In this case, the San Juan River Basin is subdivided into 10 subbasins considering the junction with its tributaries and 3 channels.

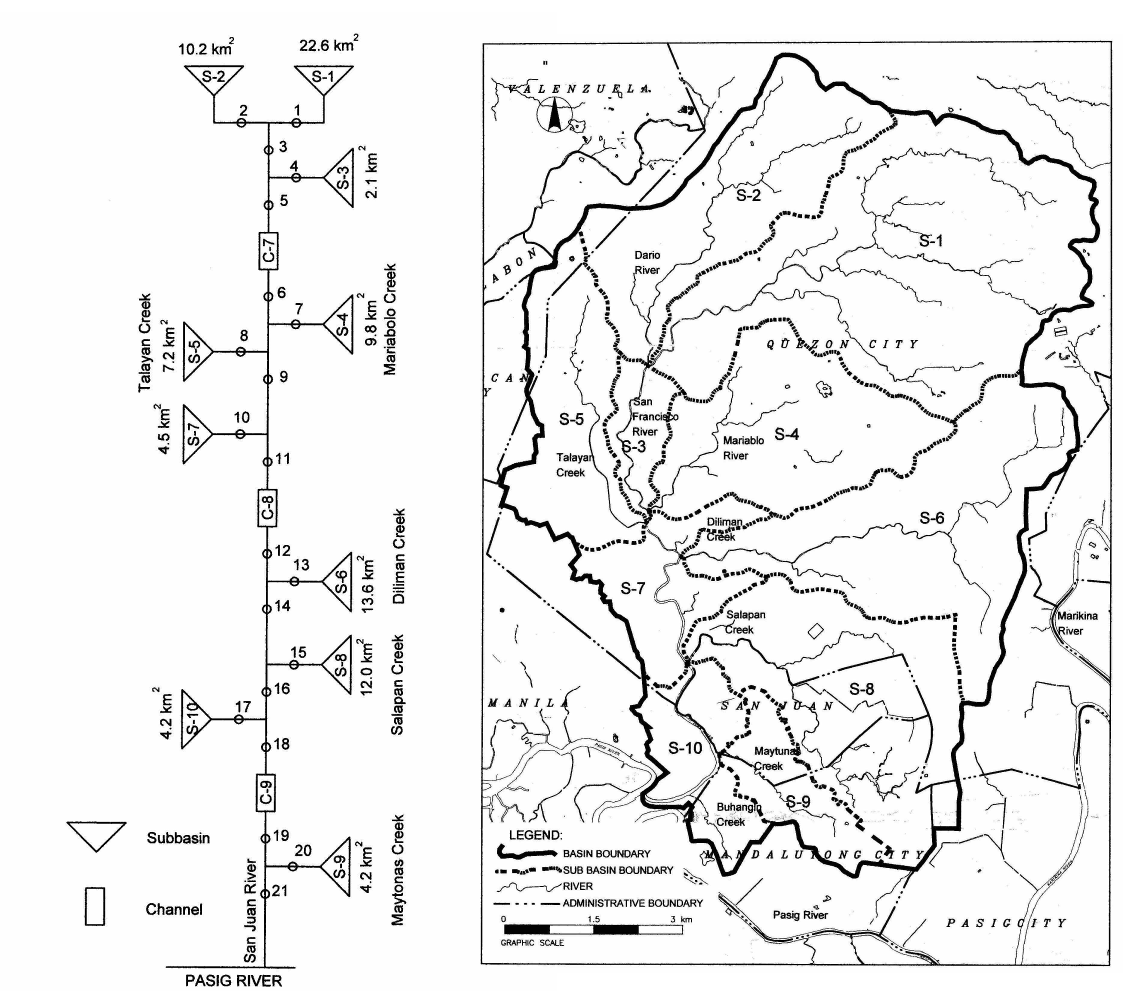


Figure 4.7 Runoff Model for San Juan River Basin

(4) Design Hyetograph

(a) Methods to Establish Design Hyetograph

The methods to prepare design hyetograph are classified into 1) one rainfall station method and 2) multiple rainfall station method.

The characteristics of rainfall is expressed by three (3) factors, namely 1) amount of rainfall, 2) temporal distribution of rainfall and 3) aerial distribution of rainfall. At present, design rainfall cannot be theoretically established from statistical or climate view points and therefore the two (2) method mentioned above are used.

In the first method, temporal distribution is considered by using so called "alternating block method" based on RIDF. Then the amount of rainfall is considered by using "area reduction factor". The same rainfall volume and the same time distribution are given to all the sub-basins.

In the second method, temporal and areal distribution can be considered at the same time by using hourly rainfall records of multiple rainfall stations as explained in Section 3.4 RAINFALL ANALYSIS. However, it is very rare that several automatic (PAGASA Synoptic) rainfall stations are located in the target river basin; thus, one rainfall station method may be applied with enough accuracy, except for large river basins such as the major rivers of the Philippines. The one rainfall station method is explained hereunder.

(b) Alternating Block Method (One Rainfall Station Method)

One design hyetograph for all the subbasins is prepared from the RIDF curve and area reduction curve as described below.

- Determine a reference point
- Determine storm duration

- Determine computation time interval
- Prepare RIDF curve
- Estimate area reduction factor corresponding to the catchment area of the reference point and storm duration
- Determine design hyetograph with one-hour interval or computation time interval

(i) Reference Point

A reference point is set at the upper end of the target stretch. The area reduction factor is determined from the catchment area of the reference point.

(ii) Storm Duration

Storm duration is set at 24 hours (1 day). However storm duration can be as short as 2 times of the lag time, when the lag time, which is the time difference from the rainfall peak to the discharge peak, is obtained from the rainfall and water level observation results or previous computation results.

(iii) Rainfall Data Time Duration

Design hyetograph is rainfall intensity series with time duration equal to the concentration time to the reference point of the target river. When the concentration time is longer than one (1) hour, hourly rainfall, which is rainfall intensity series with time duration of one hour, can be used as time interval for design hyetograph.

(iv) Rainfall Intensity Duration Frequency Curve

The RIDF curve is necessary to prepare the design hyetograph by alternating block method. As explained in **4.5.1 (4)** rainfall intensity, Rational Formula, two (2) methods are available to establish the RIDF curve of the target basin.

When the selected PAGASA Synoptic rainfall station is located in or near the basin, this RIDF should be used. Location of the selected

PAGASA synoptic rainfall station is shown in **Attachment 4.2** and the RIDF curves are shown in **Attachment 4.3**.

When no selected stations are located, the RIDF can be estimated from the specific coefficient shown in **Attachment 4.4** and the probable daily rainfall value shown in **Attachment 4.5.**

(v) Area Reduction Factor

Intense rainfall is unlikely to be distributed uniformly over a large river basin. The basin mean rainfall for specified frequency and duration is less than point rainfall. To account for this, Horton's formula may be applied to convert point rainfall to basin mean rainfall.

HORTON'S FORMULA:

$$r = r_o \cdot e^{[-0.1(0.386A)^{0.31}]}$$

Where, r = basin mean rainfall (mm)

r_o = point rainfall (mm)

A = catchment area (km^2)

$fa = r/r_o$, area reduction factor

(vi) Hyetograph Preparation

The rainfall intensity is computed from the RIDF curve for each of the durations Δt, 2Δt, 3Δt, …… and multiplied by the area reduction factor. These increments, or blocks, are recorded into a time sequence with the maximum intensity occurring at the center of the required duration Td and the remaining blocks arranged in descending order alternately to the right and left of the central block to form the design hyetograph.

Table 4.7 Design Hyetograph by Alternating Block Method

Duration (hr.)	Intensity (mm) (1)	Cumulative (2) =(1)/60*T (mm)	Incremental depth (m)	Design Hyetograph (mm)
1	93.79	93.79	93.79	8.40
2	68.68	137.37	43.58	9.86
3	55.63	166.89	29.52	12.02
4	47.36	189.46	22.56	15.56
5	41.57	207.83	18.37	22.56
6	37.23	223.39	15.56	43.58
7	33.85	236.94	13.54	93.79
8	31.12	248.95	12.02	29.52
9	28.86	259.77	10.82	18.37
10	26.96	269.63	9.86	13.54
11	25.34	278.70	9.07	10.82
12	23.93	287.10	8.40	9.07
13	22.69	294.94	7.84	7.84

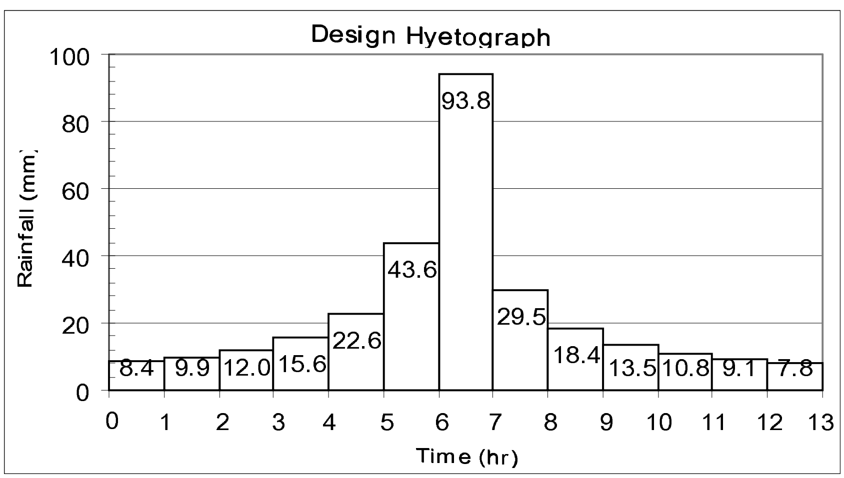


Figure 4.8 Design Hyetograph by Alternating Block Method

(5) Effective Rainfall

The next step after determination of design hyetograph is to estimate the effective rainfall. The effective rainfall or excess rainfall is neither retained on the land surface nor infiltrated into the soil but becomes direct runoff to the outlet of the river basin. A lot of methods have been proposed to estimate effective rainfall, one of them, is the runoff coefficient method explained in **4.5.1 (2) Runoff Coefficient, Rational Formula.** This can be applied to estimate the effective rainfall for the Unit Hydrograph.

(6) Unit Hydrograph

Emphasized in this section is the SCS Unit Hydrograph (UH). The SCS dimensionless hydrograph is a synthetic UH in which the discharge is expressed by the ratio of discharge U_r to peak discharge U_p and the time by the ratio of time t to the time of peak of UH, T_p. Based on study of gauged rainfall and runoff for a large number of small rural watersheds, U_p and T_p can be determined from time of concentration of the basin (or subbasin) and from U_p and T_p, the unit hydrograph for the basin (or the subbasin) can be obtained.

(a) Basic Concept and Equations

The SCS UH is a dimensionless, single-peaked UH as shown in **Figure 4.8**, which is expressed by the ratio of discharge U_t to peak discharge U_p and the time by the ratio of time t to the time of peak of UH, T_p

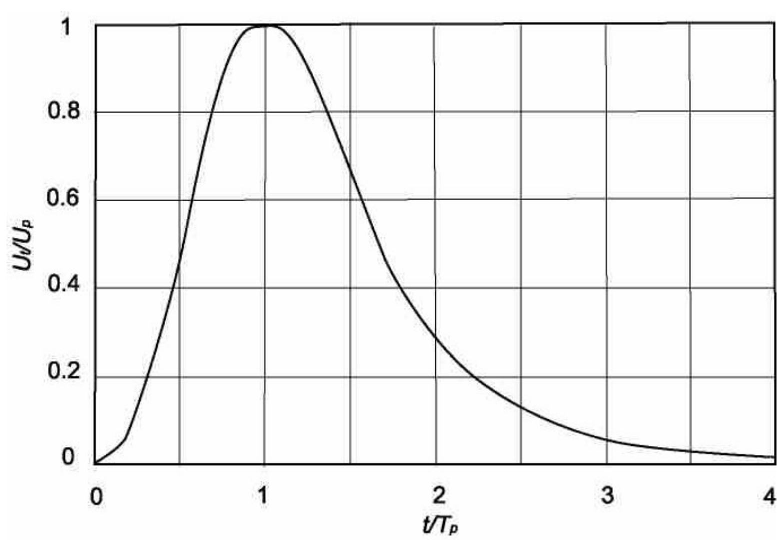


Figure 4.9 SCS Unit Hydrograph
(Source: HEC-HMS Technical Reference Manual)

Research by the SCS suggests that the time of recession may be approximated as 1.67 T_p. The peak direct runoff U_p may be computed as follows from the direct runoff volume, which is equal to the effective unit rainfall (1 cm) over area A (km^2), using a triangular unit hydrograph as shown in **Figure 4.9.**

$$U_p = 2.08 \frac{A}{T_p}$$

The time of peak is related to the duration of the unit of effective rainfall as:

$$T_p = \frac{\Delta t}{2} + t_{lag}$$

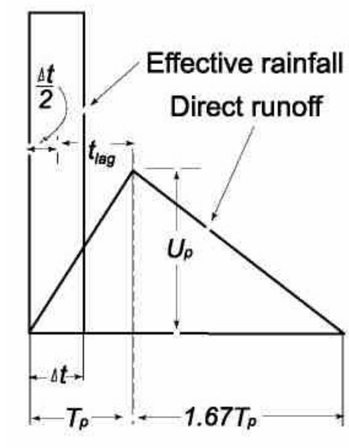


Figure 4.10 A Triangular Unit Hydrograph

Where,

Δt = the effective rainfall duration;

t_{lag} = the basin lag, defined as the time difference between the center of mass of rainfall excess and the peak of the UH. (Note that for adequate definition of the ordinates on the rising limb of the SCS UH, a computational interval, Δt, that is less than 29 % of t_{lag} must be used.)

(b) Estimating the SCS UH Model Parameter

The SCS UH lag can be estimated via calibration, using procedures described in Hydrologic Modeling System, HEC-HMS Technical Reference Manual, US Army Corps of Engineers, for gauged headwater sub-watersheds.

For ungauged watersheds, the SCS suggested that the UH lag time may be related to time of concentration, t_c, as:

$$t_{lag} = 0.6\ t_c$$

Time of concentration can be estimated by the method described in **4.5.1 (3) Time of Concentration, Rational Formula.** For the subbasin (residual subbasin), like S-10 of the San Juan River, which the channel cuts across, the time of concentration is computed as follows.

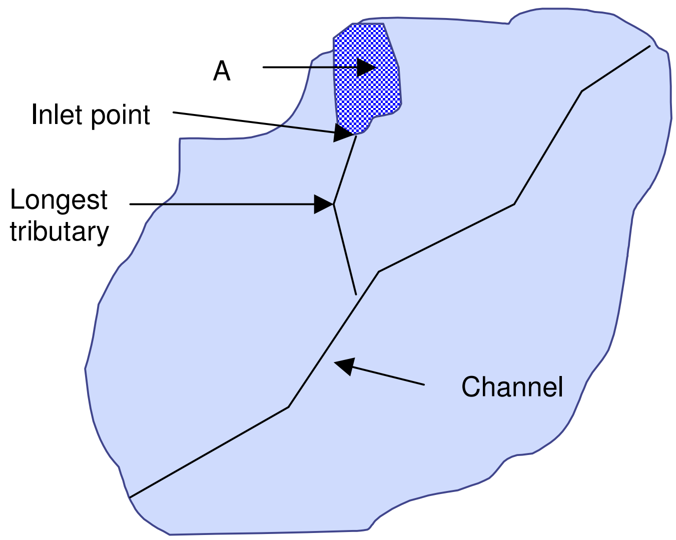


Figure 4.11 Concentration Time for Residual Subbasin

(i) Find the longest tributary flowing in the channel

(ii) Compute the flow time from Kraven's Formula

(iii) Find inlet point and set the inlet time t_i =30 min

(iv) When the catchment area (A) of the farthest point of the tributary is less than 2 km^2, compute t_i (min) from A (km) as follows.

$$t_i = \frac{30\sqrt{A}}{\sqrt{2}}$$

(v) If no tributaries are indicated in the topographical maps, assume the concentration time of the subbasin to be 30 min.

(7) Baseflow

Base flow is sustained runoff of prior rainfall that was stored temporarily in the river basin. The baseflow can be assumed to be constant during the flood. When a stream flow gauging station is located in or near the target river basin, the mean daily discharge of one day before the floods is used as the base flow. When no data is available, 0.05 m^3/s/km^2 can be used for the base flow.

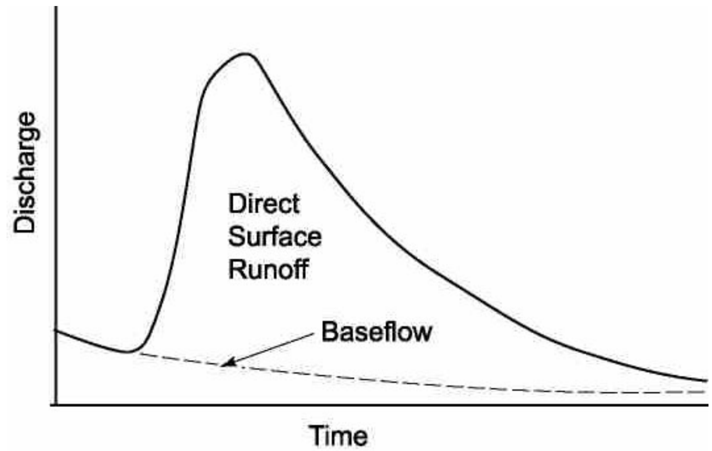


Figure 4.12 Baseflow

(8) Channel Routing Model

Distributed flow routing models are used to describe the transformation of storm rainfall into runoff over a watershed to produce a flow hydrograph for the watershed outlet. This hydrograph becomes input at the upstream end of a river system and route it to the downstream.

Several channel routing models have been proposed: These are (a) Storage function model, (b) Muskingum, (c) Kinematic wave and (d) Muskingum-Cunge standard and so on.

Out of these methods, the Muskingum-Cunge standard method is recommended due to easiness to use. The Muskingum-Cunge standard is based on the continuity equation and the diffusion form of the momentum equation to solve the unsteady flood flow (Refer to Type of Flow in **6.4.4 (1) (a))**.

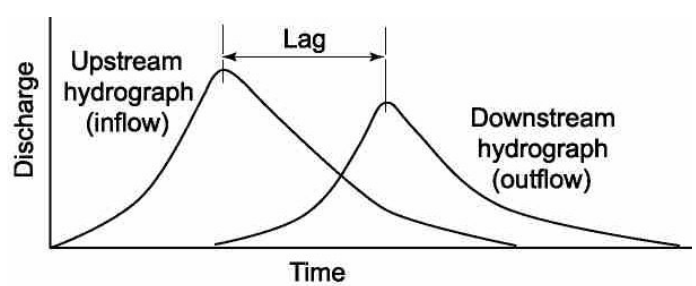


Figure 4.13 Lag and Attenuation Due to Channel

Outflow at time t (Q_t) is computed from inflow at time t (I_t) and t-1(I_{t-1}) and outflow at time t-1 (O_{t-1}) as follows.

$$O_t = C_1 I_{t-1} + C_2 I_t + C_3 O_{t-1} + C_4 (q_L \Delta x)$$

Where, q_L is lateral inflow. C_1 to C_4 are coefficients and function of wave celerity (C), discharge (Q).

(9) Computer Software

To compute the probable discharge with the SCS Unit Hydrograph, a computer and software for runoff computation is necessary. Out of a lot of software available, this manual recommends HEC-HMS for computation of SCS unit hydrograph. HEC-HMS is a computer program developed by the US Army Corps of Engineers and used by hydrologists all over the world.

CHAPTER 5 FLOOD CONTROL ALTERNATIVES AND DESIGN DISCHARGE DISTRIBUTION

5.1 DESIGN DISCHARGE ALLOCATION

The design discharge or the probable discharge is generally computed at the reference point and other important points of the target river such as the junctions of tributaries and sites of proposed flood control facilities. The design discharge/probable discharge along the river reaches is then allocated for the various flood control measures and their attendant costs determined and evaluated for the most optimum plan.

5.1.1 FLOOD CONTROL MEASURES

There are various river engineering works that are used, either singly or in combination, to provide flood protection and reduce flood damages along river reaches. These are summarized below.

Table 5.1 Flood Control Measures

No	Category	Facility/Measure
1	Increase of river flow capacity	- Dike/Levee - Widening of waterway/river - Dredging/Excavation - Combination of above
2	Reduction/control of the peak discharge of flood	- Dam - Retarding basin - Floodway
3	Prevention of bank collapse	- Revetment - Spur dike - Change of waterway/cut-off channel
4	Prevention of riverbed degradation	- Groundsill
5	Prevention of obstruction against river flow and/or maintain/conserve the good condition of the river in order to keep the flow uninterrupted	- Sabo works (for sediment control) - Regular maintenance (channel excavation/dredging)

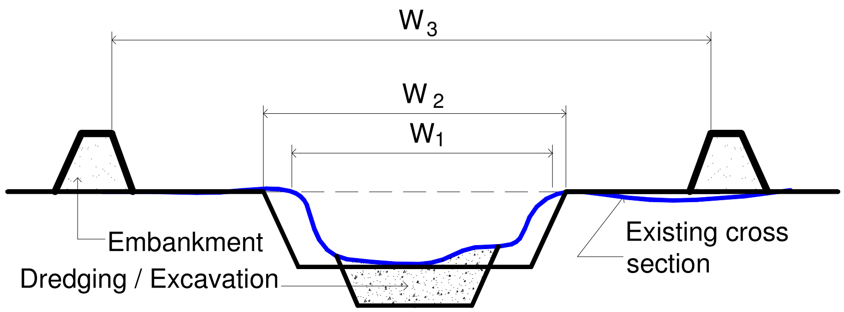
From the above table, Items No.1 and No.2 are measures directly related to the quantitative degree of design safety level against flood or the design flood frequency. Items No.3 to 5 are facilities/measures to accommodate and maintain the determined design discharge.

Therefore in flood control planning, the design discharge or the probable discharge is allocated to the facilities/measures of items No.1 and No.2 while the allocated design discharge is considered in the design to estimate the initial, operation and maintenance costs for items No.3 to No.5.

The following describes the flood control measures/facilities classified under Items 1 and 2.

(1) River Improvement

River improvement is a measure to increase the flow capacity of the existing river channel and includes widening, dredging/excavation, and dike construction as illustrated below.



W_1 = Existing river width
W_2 = Improved river width by widening
W_3 = Improved river width by diking

Figure 5.1 River Improvement

To attain a large flow capacity, widening of the channel is one of the appropriate measures. However, in urbanized area, implementation may be difficult due to land acquisition problem. Measures to reduce peak discharge at the upper reaches of the urbanized area are therefore considered to be necessary.

(2) Dam

A dam is a hydraulic structure constructed across a river to control and/or conserve water in a mountainous area. The flood peak discharge is reduced and stored in the reservoir and later released so as to reduce the peak discharge in the downstream. The dam should be situated in a place where large quantity of water will be possibly stored. However, such area is constrained by the topographical and geological conditions of the area.

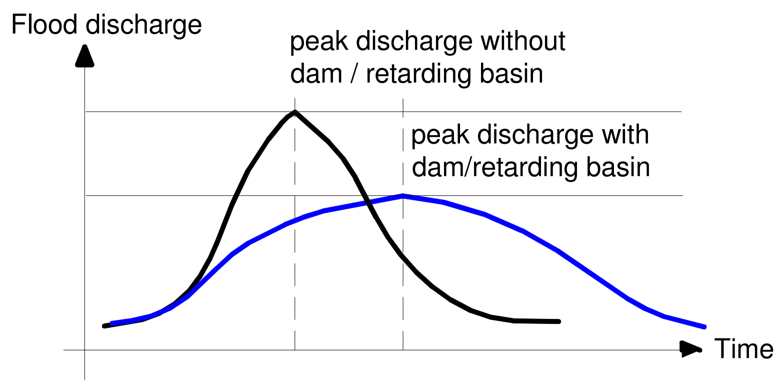


Figure 5.2 Reduction of Flood Peak Discharge by Dam/Retarding Basin

(3) Retarding Basin

A retarding basin has the same function as a dam (**Figure 5.2**). In general, a retarding basin is generally constructed in the middle reach to reduce flood discharge at the downstream reaches.

(4) Floodway

Floodway is aimed at diverting floodwater to the sea, lake or another main river from the existing river by excavating a new manmade waterway, in order to avoid the drastic widening of the existing river or to shorten the extension of improvement

5.1.2 Allocation Procedure in Flood Control Planning

In the formulation of a flood control plan, all possible flood control measures/facilities should be explored and evaluated from the engineering, socio-economic and environmental viewpoints so as to select the optimum plan. The design discharge allocated to the optimum flood control facilities/measures is termed as the design discharge distribution.

The procedures to evaluate flood control alternatives are described below.

(1) Allocate the design discharge to the river channel, which will be materialized by the river improvement, in consideration of the existing flow capacity and the land use of riverine area

(2) Allocate the remaining discharge to possible flood control facilities of item 2 of

table 5.1 Flood Control Measures.

(3) Find the size of flood control facilities to accommodate the allocated discharge

(4) Estimate the project cost based on the design of the flood control facilities

(5) Estimate the benefit to be accrued after the project implementation

These steps are iterated for the different alternative cases and their costs-benefits are compared to select an optimum plan.

The following illustrates an example, which compares four (4) alternative cases, all of which pertain to river improvement and dam construction.

Table 5.2 Allocation of Target Discharge (Example)

Alternative Case	Existing flow Capacity (m^3/s)	River Improvement (By Widening) (m^3/s)	Dam Cut (m^3/s)	Target Discharge (m^3/s)
1	2,000	3,000	0	5,000
2	2,000	1,500	1,000	5,000
3	2,000	1,000	1,500	5,000
4	2,000	0	3000	5,000

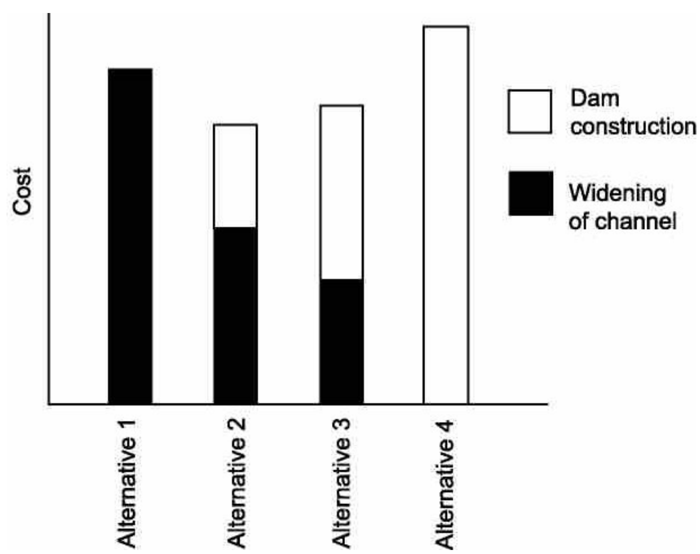


Figure 5.3 Comparative Cost Analyses of Alternatives (Example)

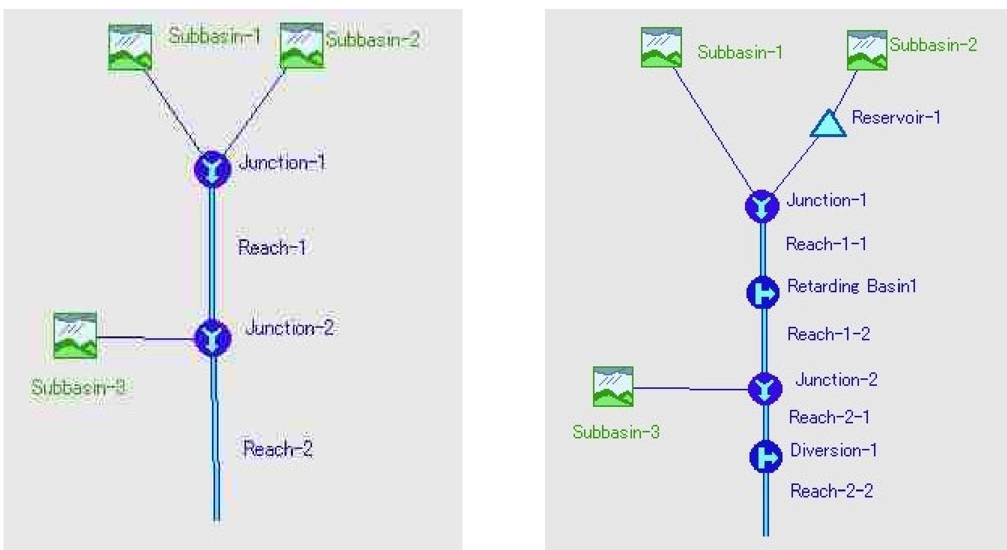
From the comparative study of alternatives' cost, alternative 2, which is of least cost, is deemed to be the optimum plan.

5.2 MODELING OF DISCHARGE ALLOCATION TO FLOOD CONTROL FACILITIES/MEASURES

5.2.1 Runoff Model for Alternatives

The same runoff model used to obtain the design discharge/probable discharge is also used to evaluate the effect of flood control or the reduction in discharge due to flood control measures/facilities, such as dam, retarding basin and floodway.

Figure 5.4 shows a sample river basin, in which the flood control plan is under study. **Figure 5.4 (a)** is a runoff basin model for SCS unit hydrograph in HEC-HMS window to obtain the design discharge, while **Figure 5.4 (b)** shows a model for the allocation of the design discharge to a dam, a retarding basin and a floodway. Using basin model (b), the characteristics (size) of these flood control facilities, which are necessary to accommodate the allocated design discharge, are determined.



(a) Model for Design Discharge

(b) Model for Allocation of Design Discharge

Figure 5.4 Runoff Basin Model

5.2.2 Modeling of Effect by Flood Control Facilities

Each runoff model has its different methods to compute the effects of flood control facilities to reduce the discharge at the downstream reaches. This manual recommends HEC-HMS software for runoff computation. Its modeling principles are explained hereunder.

(1) Dam

The outflow hydrograph reduced from the inflow hydrograph by storing floodwater in the reservoir is shown as follows.

There are several types of discharging floodwater stored in the reservoir. **Figure 5.5** illustrates typical hydrographs or attenuation type of peak flood discharges.

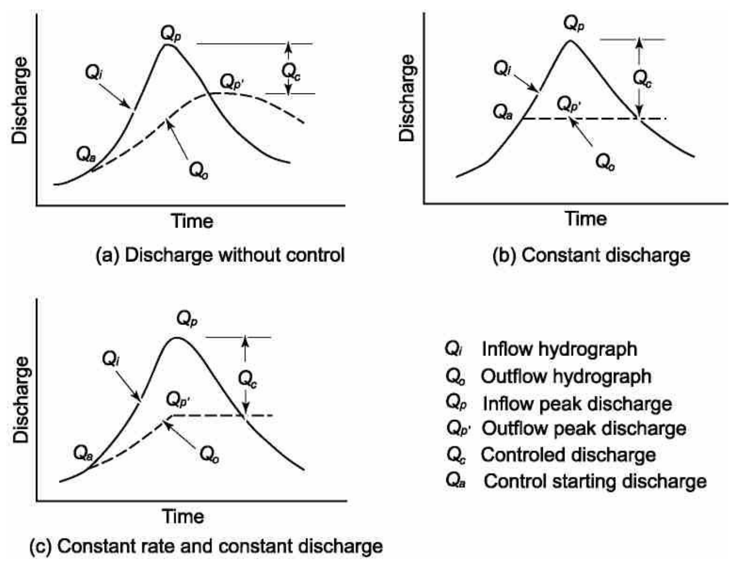


Figure 5.5 Typical Discharging by Dam

From the above, the discharge without control is generally used to evaluate the effectiveness of dam for the attenuation of flood in the downstream reaches. The outlet of this kind of dam may consist of outlet pipe (culvert) and the emergency spillway (**Figure 5.6**).

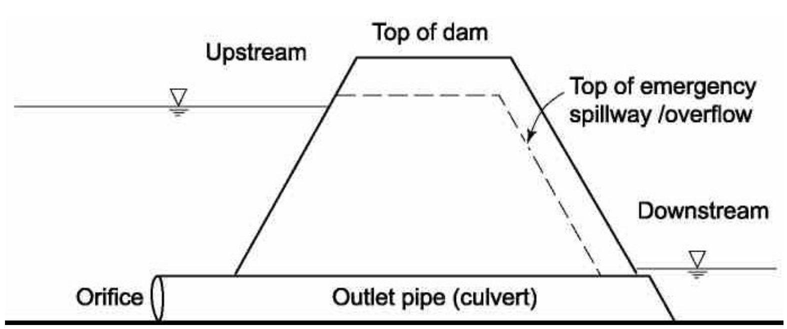


Figure 5.6 Simple Outflow Structure of Dam
(Source: Hydrologic Modeling System HEC-HMS Technical Reference Manual)

The outflow discharge from the outlet and the spillway can be computed using the orifice equation and the weir equation as described below.

Fully Submerged Orifice Equation

$$O = KA\sqrt{2gH}$$

Where, O = outflow discharge (m^3/s), K= discharge coefficient that depends upon the configuration of the opening to the culvert, A=cross-sectional area of the culvert, normal to the direction of flow, H = total energy head on outlet (m), which is the difference in the downstream water-surface elevation and the reservoir water-surface elevation.

Weir Equation

$$O = CLH^{1.5}$$

Where, O = outflow discharge (m^3/s), C = discharge coefficient that depend upon the configuration of the weir, L = effective weir width (m), H = total energy head on crest (m).

As can be seen from the orifice and the weir equations, the surface water level of the reservoir and that of the downstream channel are necessary for computation. The water level in the reservoir is computed from the following one-dimensional continuity equation;

$$I_{ave} - O_{ave} = \frac{\Delta S}{\Delta t}$$

Where, I_{ave} = average inflow during time interval Δt, O_{ave} = average outflow during time interval, ΔS = storage change during time interval. With a finite difference approximation, this can be written as:

$$\frac{I_t + I_{t+1}}{2} - \frac{O_t + O_{t+1}}{2} = \frac{S_{t+1} - S_t}{\Delta t}$$

Where, t = index of time interval, I_t and I_{t+1} = inflow discharge (m^3/s) at the beginning and the end of the t^{th} time interval, respectively, O_t and O_{t+1} = the corresponding outflow discharge (m^3/s), and S_t and S_{t+1} = corresponding storage volume (m^3).

This equation can be rearranged as follows:

$$\left(\frac{2S_{t+1}}{\Delta t}+O_{t+1}\right)=\left(I_t+I_{t+1}\right)+\left(\frac{2S_t}{\Delta t}-O_t\right)$$

All terms of the right-hand side are known. The values of I_t and I_{t+1} are the known inflow hydrograph. The values of O_t and S_t are known at the t^{th} time interval. At t = 0, these are the initial conditions and at each subsequent interval, they are known from calculation in the previous interval. Thus, the left-hand side ($2S_{t+1}/\Delta t+O_{t+1}$) can be computed from the above equation.

As explained above, the outflow discharge O_{t+1} is a function of the reservoir water level and storage volume S_{t+1} is also a function of the reservoir water level and thus, the corresponding values of O_{t+1} and S_{t+1} can be obtained.

Figure 5.7 illustrates elevation-outflow relationship consisting of the orifice and the weir.

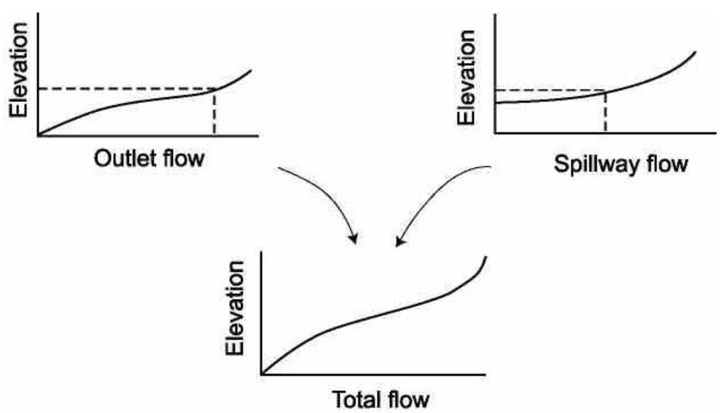


Figure 5.7 Composition of Outlet Flow and Spillway Flow
(Source: Hydrologic Modeling System HEC-HMS Technical Reference Manual)

The relationship of elevation and storage volume can be obtained by measuring the respective areas with respect to the elevation based on the topographical maps.

(2) Retarding Basin

The reduction of design discharge at the downstream and the necessary storage volume are simulated using HEC-HMS. The discharge to the retarding basin can be computed from a user-specified monotonically increasing inflow-diversion relationship. **Figure 5.8** shows a sample, in which outflow to main channel is equal to inflow until the inflow reaches storage starting discharge and then outflow is kept constant. The volume of the retarding basin is computed as the sum of the diverted discharge.

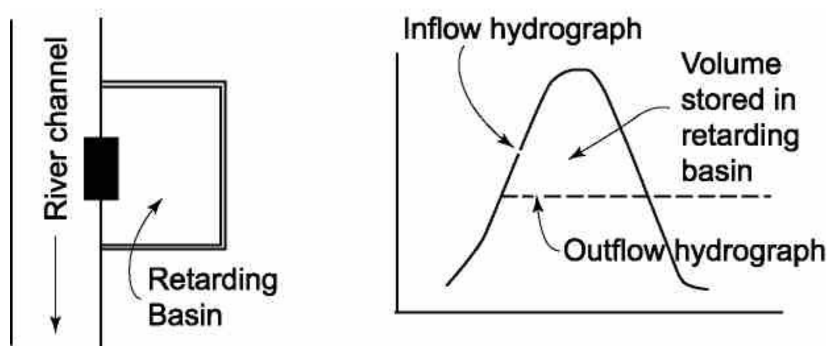


Figure 5.8 Flood Control by Retarding Basin

(3) Floodway

The diverted discharge and outflow discharge can be computed similar to the above.

5.3 DESIGN DISCHARGE DISTRIBUTION

Optimum plan is determined based on the design discharge diagram established at the reference points, the proposed location of flood control facilities and the major tributaries in consideration of the results of the project evaluation from the engineering, socio-economic and environmental view points. **Figure 5.9** indicates design discharge distribution for the Pasig Marikina River.

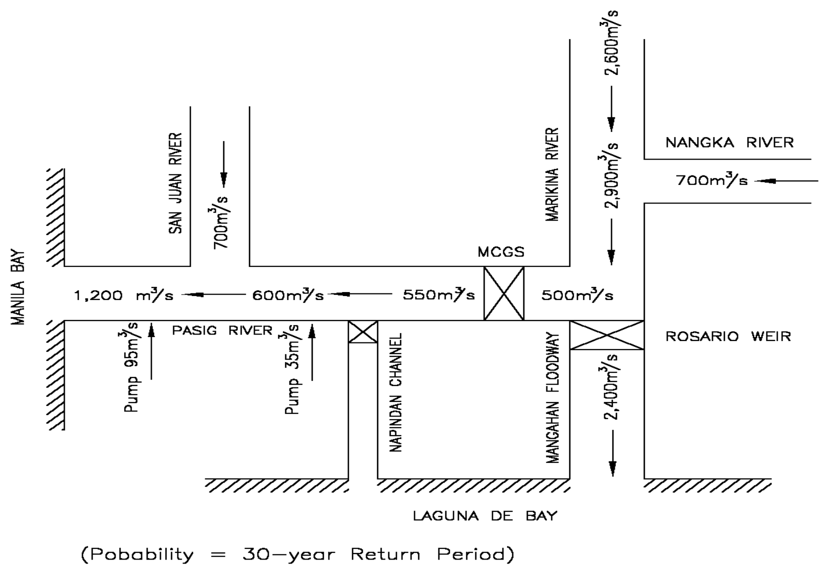


Figure 5.9 Design Discharge Distribution for the Pasig-Marikina River
(Source; Detailed Engineering Design of Pasig-Marikina River Channel Improvement Project, Hydrology, Volume VI, March 2002)

CHAPTER 6 RIVER CHANNEL IMPROVEMENT

6.1 GOAL OF RIVER CHANNEL IMPROVEMENT

Rivers generally take off from the mountains, then flow along the plain terrains and finally join the oceans. They form more or less defined channels; drain away the land water obtained by rainfall; and discharge the unutilized water back into the sea. The rivers not only carry water but also sediments washed down from the catchment area and eroded from the beds and the banks of rivers.

Channels are formed by the interactions of water and sediments. During large floods, floodwaters not only overflow and bring about inundation to riverine areas, but also cause serious sediment related damages. These include: 1) bank erosion/collapse including dike and revetment and 2) river bed degradation.

In the Philippines, a lot of river structures are damaged during floods and inundations in the riverine areas while substantial budgets are spent to reconstruct/repair these structures. According to Damage Profile of Flood Control Structures published under Project ENCA, DPWH in 2002, damages of structures occur, in some instances, due to the lack of analysis of the river characteristics related to sedimentation and nonexistence of design discharge and/or design flood level used for planning and design.

Considering the various functions of rivers, river channel improvement should be planned and designed based on the adequate analysis of river characteristics to attain the following:

(1) To carry design flood water, which is allocated to the target river channel

(2) To protect the target river channel from scouring and/or erosion for flood discharges equal to and lower than the design flood discharge

6.2 RIVER AND SEGMENT

6.2.1 Classification of River Segment

The shape of the river is formed through the recurring effects of scouring, meandering and sedimentation as a result of perennial floods. The shape/configuration of a natural river generally depends on the parameters, such as riverbed gradient, riverbed material and the annual maximum flood. The riverbed materials can be roughly assessed through the riverbed gradient. The riverbed gradient information can roughly provide the phenomenon of the stream and river characteristics. Therefore, during river improvement planning, the first step before river structure could be designed is to undertake river survey and determine the actual river (riverbed) gradient. Due to the difficulties in the actual cross sectional survey and the riverbed gradient determination from the result of the said survey, the importance of understanding the river characteristics according to long-range section is introduced in this guideline.

"Classification of River Segment" is introduced to assess the river characteristics. Each river segment is classified by the riverbed gradient, which has its distinct characteristics, which pertain to the riverbed material, tractive force of flow during flood, river width and water depth during ordinary flood, etc. Similar segments have almost the same roughness and/or sand bar conditions as the velocity of flow and phenomena of scouring are almost at the same range in the same segment. The identification of the segment of the target stretch for improvement is significant in river planning and designing of structure. Past plan and design of structure in the same segment may be useful references. **Table 6.1** shows the classification of river system into several segments.

Longitudinal profile of the river gradually becomes gentle from the upstream towards the downstream. It has been thought that the frictional action of the riverbed materials makes them smaller. However, the longitudinal profile and the size of the riverbed materials change in a certain point rather than gradually changing. The riverbed materials, such as gravel disappear while rough sand appears in a certain area. There is lesser tractive force to move the gravel in the downstream where the riverbed gradient is gentle. Gravel usually accumulates in the upstream point. Moreover, fine sediments from the mountain area flow downstream and do not commonly remain in the upstream area.

The safety of the river structure against scouring phenomena depends on the river characteristics by segment. One of the external forces that destroy the dikes and banks is flow velocity, which is relative to the river alignment, longitudinal and cross sectional profiles and types of riverbed materials. The countermeasure required to overcome these external forces is by changing/adjusting the riverbed gradient. Thus, primarily when the river improvement plan is discussed, the classification of each river segment should be recognized and cannot be ignored.

Table 6.1 Classification of River Segment and its Characteristics

CLASSIFICATION	SEGMENT M	SEGMENT 1	SEGMENT 2		SEGMENT 3
			2-1	2-2	
Geography	Mountain	Alluvial	Narrow Plane	Natural Levee	Delta
Diameter of typical riverbed Materials	Various materials	More than 2 cm.	3-1 cm.	1- 0.3 mm	Less than 0.3 mm.
Riverbank Material	Many types of soil and rocks appear on the banks as well as on riverbed.	Riverbank material is composed of thin layer of sand and silt which is same as the riverbed.	Lower layer of the riverbank material is the same with the riverbed.	Mixture of fine sand, clay and silt. Same material with riverbed	Silt and Clay
Gradient	Various. Generally steep gradient.	1:60 – 1:400	1:400 – 1:5,000		1:5,000 – Level
Meandering	Various	Few bend/meander	Heavy meandering		Large and small meandering
Bank Scouring	Heavy	Heavy	Medium. Mainstream course changes where bigger riverbed materials exist.		Weak. Location/course of stream is almost fixed.
Water Depth of Annually Maximum Flood	Various	0.5 - 3m	2.0 – 8.0 m		3.0 – 8.0 m

Source: Technical Standards and Guidelines for Planning and Design, March 2002

6.2.2 River Characteristics Corresponding to River Segment

The river channel is a result of the movement of river materials forming the bed and the banks through the interaction of tractive force of water and the bed materials during floods. Large floods cause the river channel to change by the movement of a large amount of sediments, resulting in deposition and scouring. Furthermore, the introduction of river improvement works that aims to increase the flow capacity and consequently effect changes in river conditions, may trigger a new channel movement with serious scouring/degradation or might return to the original river channel with sediment deposition.

Therefore, the river characteristics of the target stretches for river improvement should be roughly assessed by classifying according to the particular river segment to analyze the phenomena related to the sediment movement and to select the optimum planning/design for the river improvement and consequently minimize initial/maintenance cost. Hereunder, the river characteristics or phenomena of the bed materials are explained in connection with the river segment.

(1) Longitudinal Profile

During floods, sediments in the upstream areas are transported and deposited in the downstream areas relative to the size of sediments and the tractive force acting on the sediments.

Imbalance of inflow and outflow of sediment volume in a river stretch causes change of the riverbed elevation. When the inflow volume becomes larger than the outflow, the riverbed elevation of the stretch rises. Gradually, the outflow of the stretch increases due to the rise of riverbed, resulting in propagation of riverbed rise to downstream. These phenomena tend to occur in the following river segment.

(a) Segment 1 and Segment 2-1 (Gravel bed river channel)

(i) Degradation in the upper reaches of the river segment in the river in which sediment supply is decreasing. But due to armoring, the lowering of the riverbed is constricted

(ii) Sediment deposition in lower reaches of the river segment in which gradient becomes milder.

(b) Segment 2-2 (Sand bed river channel)

Degradation occurs in upper reaches of the river where sediment supply is decreasing due to dam construction. The sand is easily transported compared with the gravel; thus, lowering is fast.

(c) Segment 3

In this segment, especially at the transition from segment 2-2, heavy sediment deposition occurs and reduces flow capacity.

Measures to maintain the river structures, such as revetments, bridges, function of junction and retarding basins, should be considered in river stretches of degradation. While dredging should be regularly undertaken in rivers stretches of aggradation to keep the design flow capacity.

(2) Sand Waves (Bed Forms)

For rivers flowing in alluvium, sand waves (bed forms) are formed by sedimentation. The sand wave (bed form) is broadly classified into (1) small scale sand wave and (2) medium scale sand wave (Refer to **Figure 6.1**). Both small and medium scale sand waves can exist under the same hydraulic conditions.

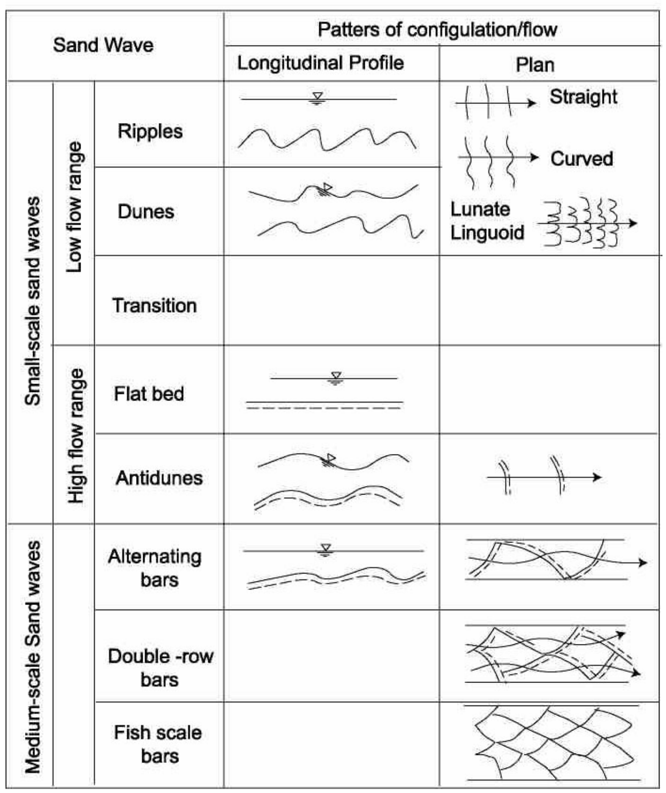


Figure 6.1 Type of Sand waves

Small-scale sand wave has close relation with channel flow resistance and with sediment discharge, while medium scale sand wave has close relation with river meandering and with riverbank erosion.

(a) Type of Sand Waves (bed forms)

(i) Small Scale Sand Waves

Depth - particle size ratio (*H/d*) is a dominant parameter of the small-scale sand waves. Where *H* is the depth of the water and *d* is the particle size.

Ripples: The wavelength and height are closely related with particle size (wave length = 500*d* to 1500*d*). They move toward downstream with less velocity than mean water flow.

Dunes: The wavelength and the height are closely related with water depth (wavelength = 4*H* to 10*H*). They move toward downstream with lesser velocity than mean water flow. Peak of water surface takes place at the drop of sand wave.

Transition: Undeveloped ripples and dunes exist on flat bed

Flat bed / plain bed: Flat bed with high sediment transport

Small-scale sand waves (bed form) change from ripples ($Fr \ll 1$), dunes ($Fr<1$), transition ($Fr<1$) and flat bed ($Fr<1$ and $d \leq 0.4$ mm) corresponding to water depth and velocity during a flood. Further, it changes to antidune ($Fr \gg 1$) when velocity increases.

Fr = Froude Number

$$F_r = \frac{V}{\sqrt{gd}}$$

(ii) Medium-scale Sand Waves (Sand Bars)

Width depth ratio (*B/H*) is a dominant parameter of sand bars (**Figure 6.2**). Scale of sand bars is shown in **Figures 6.3 and 6.4**

Alternate bars: $B < 100$ (70 to 140) H (Length of bars: L_s)
L_s =5 to 15B

Multiple bars: $B > 100$ (70 to 140) H (Width of bars: B_s)

L_s= 2 to 6B, B_s =100(70 to 140)H

L_s = 1.5 to 4B_s, B_s =100(70 to140)H

Point bars: Standing bars that are often seen in meandering channels

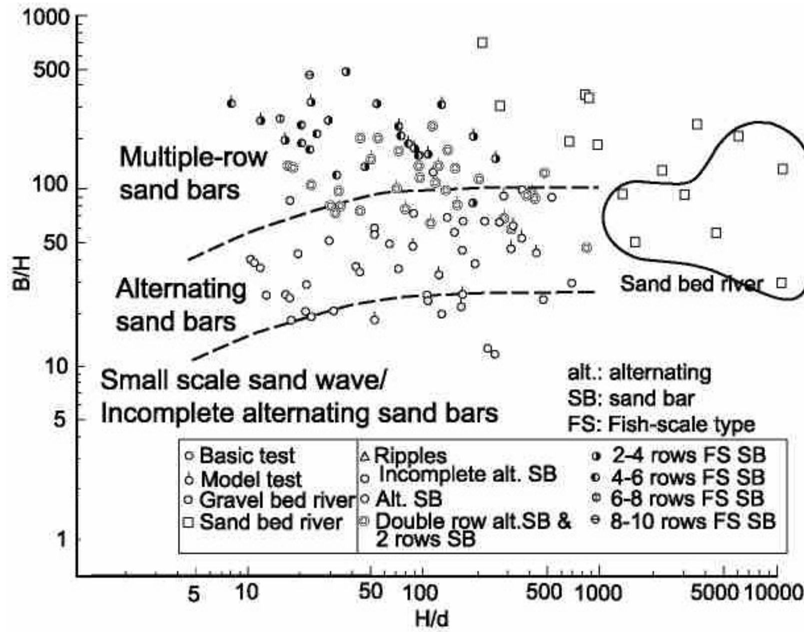


Figure 6.2 Sand Bar Occurrence

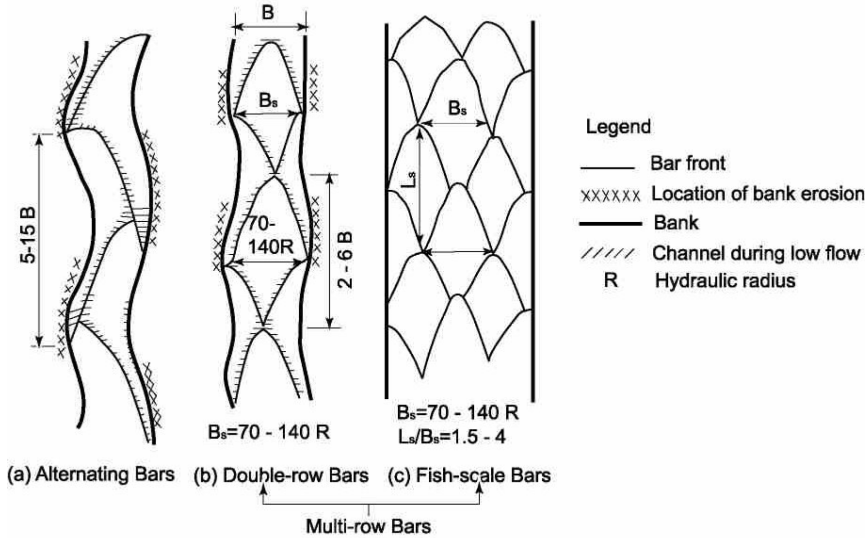


Figure 6.3 Scale of Typical Sand Bars

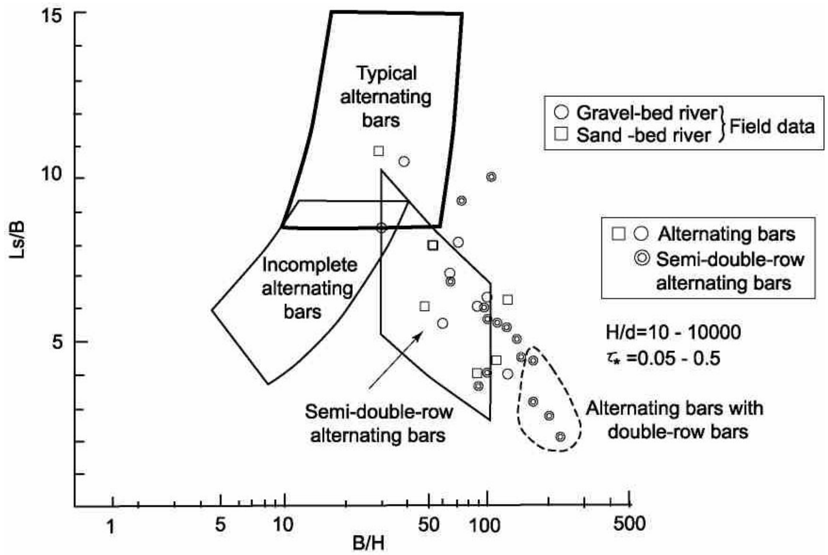


Figure 6.4 Scale of Sand Bars and River Channel

(b) Characteristic of Sand Bars

When the ratio of water width (B) to water depth (H_m) of mean annual maximum discharge exceeds 10 ($B/H_m \geq 10$), sand bars are generally formed (**Figure 6.2**).

(i) Segment 1

In segment 1, multi-row sand bars are produced in shallow and widespread water flow. Flows are separated towards both banks on the multiple sand bars and thus tend to reduce meandering phenomena. The sand bars move towards downstream while the attack point of the flow moves together, thus restraining meandering. Consequently, straight river channel is generally formed. The ratio of river width to depth (B/H_m) is greater than 100 (Refer to **Figure 6.2**).

(ii) Segment 2

In segment 2, sand bars are produced and make flood flow drift towards banks and cause scouring at the attack points located at the front edges of sand bars, especially in segment 2-1.

Alternating sand bars develop with bank erosion, thus initiating and accelerating the formation of meander. Even in a straight channel with a constant width, bank is easily eroded and/or scoured due to movement of sand bars towards downstream. In wide river channels, in which multiple sand bars and islands are formed, meandering is not strong.

In river stretches where alternating sand bars are formed, there is a high possibility of bank erosion through formation of meandering channel. Meandering is controlled by construction of revetment and/or spur dike.

When bank erosion and/or meandering is controlled by river structures, scouring becomes deeper at flow attack zones and thus stronger foot protection will be necessary.

(iii) Segment 3

No sand bars or weak sand bars are formed in river channels of segment 3 and drift of flow due to sand bar need not be considered.

(3) River Channel Feature and Mean Velocity

The river channel features for mean annual maximum discharge Q_m (bankfull discharge), such as river width (B), cross-sectional area (A), mean water depth (H_m) and mean flow velocity (V_m), can be estimated from three (3) parameters; namely, Qm, bed slope (I_b) and representative grain diameter (d_R).

Figure 6.5 shows the relationships between water depth (H_m) of the low-water channel at the time of mean annual maximum discharge, the representative grain diameter (d_R) and different bed slope (I_b), based on the river data in Japan.

Figure 6.6 indicates the mean flow velocity (V_m) in a low-water channel corresponding to the mean annual maximum discharge and 100-year return period flood.

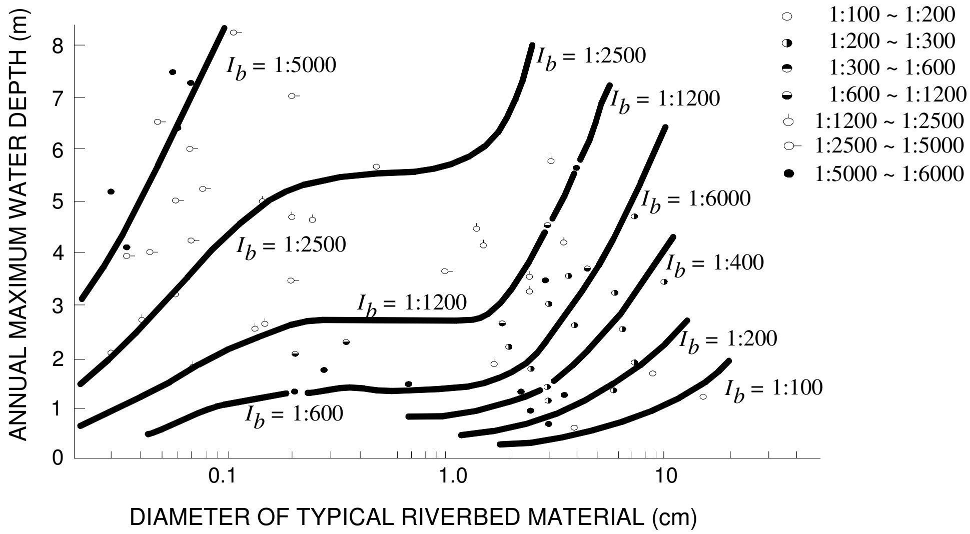


Figure 6.5 Relationship with the Diameter of Riverbed Material and Annual Maximum Water Depth

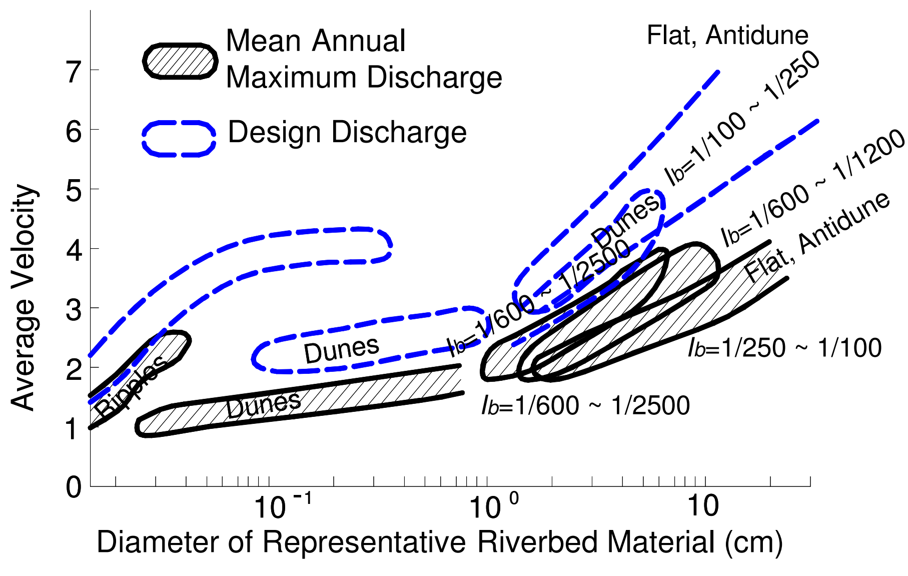


Figure 6.6 Relationship with the Diameter of Riverbed Material and Mean Velocity Based on the Diked Rivers of Japan

(4) Erosion

(a) Types of Bank Erosion

Bank erosion can be classified into three types, namely; 1) simple erosion, 2) collapse type erosion and 3) slip type erosion. The respective characteristics of the three types of erosions can be summarized as follows.

Table 6.2 Type and Cause of Bank Erosion

Type of bank erosion	River Segment	Causes	Size of Erosion	Typical Bank Material	Remarks
Simple erosion	Segment 1 Segment 2-1	River bed erosion and water attack due to development of sand bars	Erosion width reaches 100 m. Length is related to scale of sand bars	Gravel (same as river bed material)	Erosion zone moves with sand bars
Collapse type erosion	Segment 2 Segment 3	Scouring of bank slope or flowing out of fine sand layer, resulting in collapse of high water channel	In most cases, erosion width is between 1 to 3 times of bank height.	Sand Silt	In mild slope river, eroded materials remain in the channel, thus reducing erosion
Slip type erosion	Segment 2 Segment 3	Shear force reduction due to increase of water content of bank material during floods	Erosion width is wider than collapse type erosion	Sand Silt	This erosion is rare

(b) Causes of River Bed Erosion

Simple erosion and collapse type erosion, which occur in most cases, are induced by riverbed erosion. The causes of riverbed erosions can be classified as follows.

(i) Change in River Bed Elevation

Any excavation of riverbed, such as river improvement and gravel extraction, may cause riverbed degradation. Also, a reduction in sediment transport from upstream, such as dam construction, destroys the sediment balance resulting in bed elevation degradation.

(ii) River Channel Alignment

Scouring occurs in two (2) portions of the river channels related to river channel alignment; 1) narrow sections (change in river width) where the flow velocity increases and 2) curved or meandering channel.

Change in River Width

Assuming $q_s = k \cdot u_*^p$

Where, q_s: sediment discharge per unit width ($m^3/s/m$), u_*: shear velocity (m/s) , k: constant, and p varies from 3 to 10.

$$u_* = \sqrt{gRS}$$

Where, g: gravity acceleration (9.8 m/s^2), R: hydraulic radius and S: energy slope.

Mean depth H_m, surface width B, water area A and channel slope I of the narrow channel is estimated from H_o, B_o, A_o and I_o in upper section.

For Gravel bed: during flood → p = around 5 to 7

$H_m/H_o = (B/B_o)^{-4/5 \text{ to } -6/7}$ $A/A_0 = (B/B_0)^{1/5 \text{ to } 1/7}$ $I/I_0 = (B/B_0)^{2/5 \text{ to } 4/7}$

For Coarse sand bed: Dune bed during flood → p = around 3

$H_m/H_o = (B/B_o)^{-2/3}$ $A/A_o = (B/B_o)^{1/3}$ $I/I_o = (B/B_o)^0$

For Medium/fine sand bed: Flat bed during flood → p = around 4

$H_m/H_o = (B/B_o)^{-3/4}$ $A/A_o = (B/B_o)^{1/4}$ $I/I_o = (B/B_o)^{1/4}$

Figure 6.7 indicates scouring at narrow section for p =3 and p = 4.

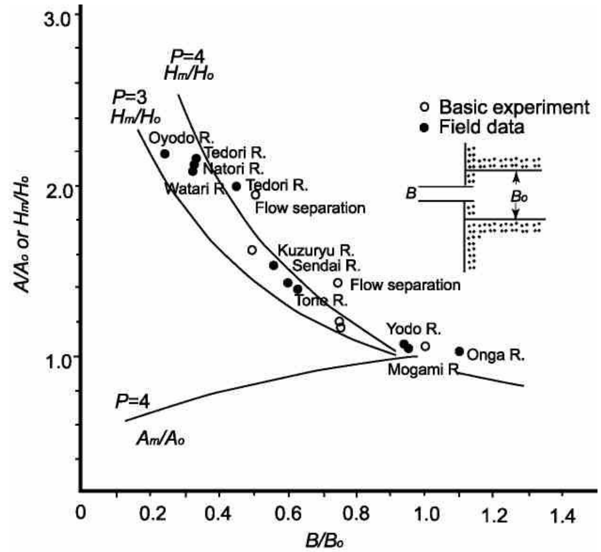


Figure 6.7 Scouring at Narrow Section

Curved Channel

The ratio of maximum water depth of the curved channel (H_{max}) to average water depth of the straight channel (H_m) is shown in **Figure 6.8**.

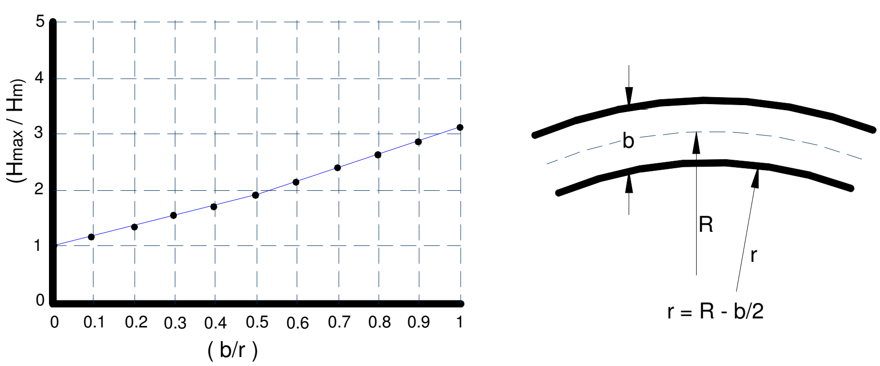


Figure 6.8 Scouring at Curved Channel

(iii) Sand Bars (Medium-scale Sand Waves)

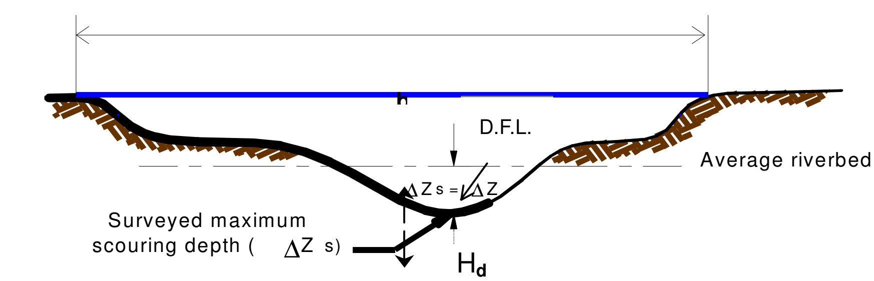
Even though a channel is straight, sand bars make flood flow drift towards banks and cause scouring at flow attack zones.

The depth of scouring is greatly affected by the bar height of sand waves. The bar height (H_s) of the gravel-bed river is roughly estimated from 1) width of waterway (b) and 2) mean water depth (H_m) of mean annual maximum discharge and 3) representative particle size (d_R).

<u>For Straight Line Waterway</u>

Please refer to **Figure 6.9.** Basically, surveyed maximum scouring depth (ΔZs) is the same as maximum scouring depth (ΔZ), when maximum scouring depth (ΔZ) is influenced by the <u>height of sand bar</u>, the Maximum scouring depth (ΔZ) is the larger value between the surveyed maximum scouring depth (ΔZs) or the calculated maximum scouring depth (ΔZc).

When sand bar is not generally generated in the river where minute sand (0.2mmΦ or less) piles up and the ratio of river width (b) and average water depth (Hd) is 10 or less (b/Hd ≤ 10), the surveyed maximum scouring depth (ΔZs) is the maximum scouring depth (ΔZ).



where :
b : river width
H_d :average water depth

Figure 6.9 Maximum Scouring Depth

In case b/Hd is ≤ 10 and dr is ≤ 0.2mm

When the ratio b/Hd exceeds 10 (b/Hd ≧ 10), <u>sand bar is generally formed</u>. In this case, the calculated maximum scouring depth (ΔZc) should be determined. Then, compare with the surveyed maximum scouring depth (ΔZs). The Maximum scouring depth (ΔZ) has greater depth.

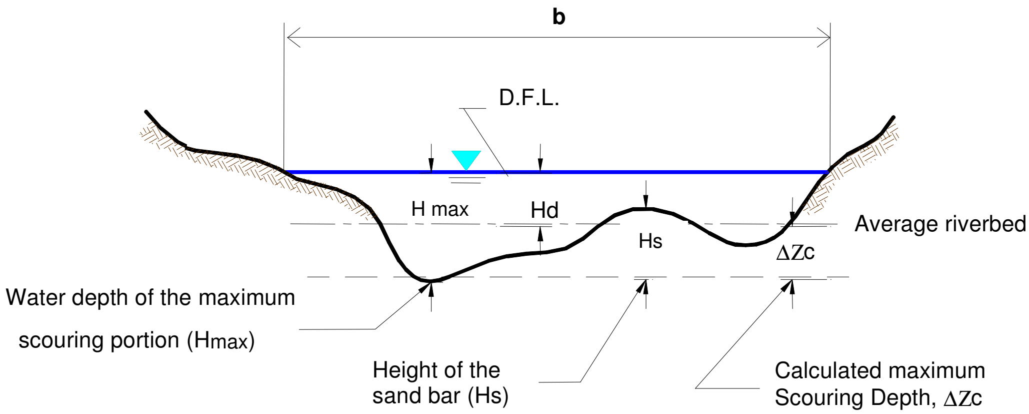


Figure 6.10 Maximum Scouring Depth Influenced by the Height of Sand Bar.

where :

b : river width

H_d : average water depth

H_{max}: maximum scouring portion

H_s : height of sandbar

H_d = DFL – (A / b)

where :

A : cross sectional area

b : width of waterway

<u>For rivers with gravel diameter, dr ≥ 2 cm</u>

- The ratio of height of the sand bar (H_s) and average water depth (H_d) is decided as follows.

Calculate the ratio of width of the waterway and average water depth.

b / H_d I

Calculate the ratio of average water depth and diameter of typical riverbed material.

H_d / d_r **II**

The ratio of H_s / H_d is decided from figure based on I and II. Using **Figure.6.11** below, value of Hs/Hd is obtained

H_s / H_d **III**

Water depth with the maximum scouring portion (H_{max}) is calculated by this formula and based on III.

H_{max} = 1+0.8(III) (H_d) **IV**

The calculated maximum scouring depth (ΔZc) is determined by using the following formula.

ΔZ_c = (IV) – H_d **V**

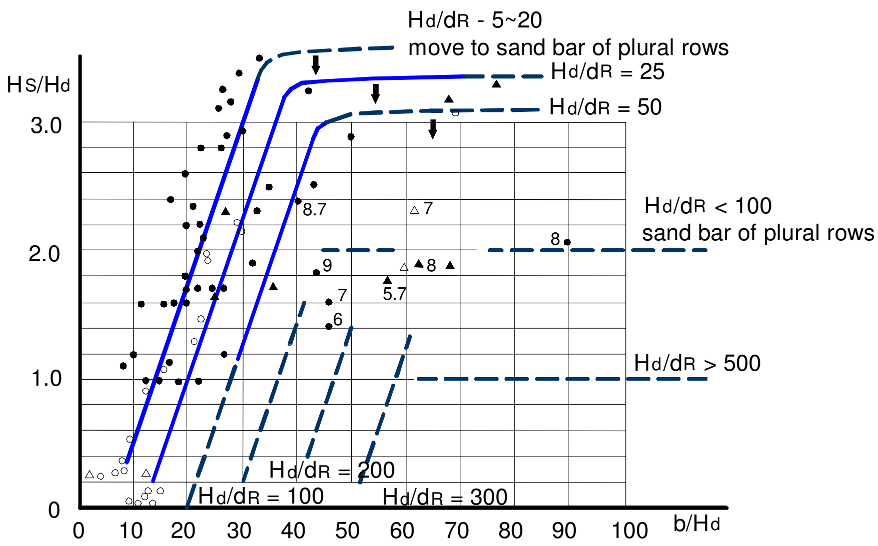


Figure 6.11 Relationship of Hs/Hd~b/hd

Source : Guidelines for Disaster Restoration Works, Utsukushii, Mamoru, Sagai, Honshin, Japan Oct. 1998.

The surveyed maximum scouring depth (ΔZs) and the calculated maximum scouring depth (ΔZc) are compared, and the larger value is the Maximum scouring depth (ΔZ).

Rough sand and middle level sand (in case of 0.2mm < d_r < 2cm)

- Sand bar of plural rows (sand bar which look like scales of fish)

  Considering the influence of sand bar of plural rows, the calculated maximum scouring depth, ΔZc should be multiplied by **1.5**.

  $\Delta Zc = 1.5$ (V) **VI**

(iv) Structures

A structure located in the path of flowing water increases the velocity of flow around the structure and causes local scouring, such as flow around the bridge and the spur dike.

(4) Response to River Channel Improvement

(a) Change of Profiles

When dredging of channel is undertaken with steeper bed slope than the existing one, the bed slope tends to return to the original slope, since the slope forms and reaches a state of equilibrium through sedimentation for certain duration. With this, when local dredging is made, deposition occurs in the dredged stretches and the bed returns to the original condition.

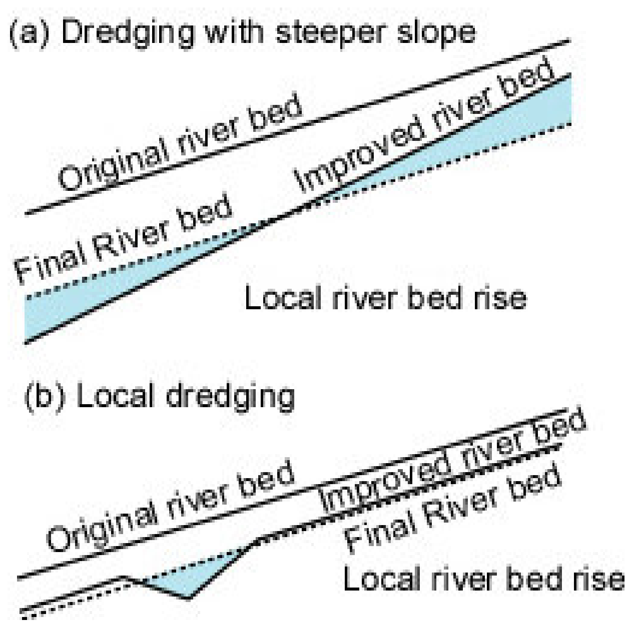


Figure 6.12 River Bed Change after Dredging

The same phenomenon occurs when the cut-off channel is constructed to correct/straighten the conspicuous meandering. The cut-off channel installed with steep slope in the milder existing channel break the stability, causing river bed degradation because of increased velocity in the upper reaches and correspondingly causing the rise of riverbed because of deposition in the lower reaches.

(b) Movement of Sand bars

River bed scouring occurs at the flow attack zones located at the edge of sand bars. However, the flow attack zones start to move causing modification of the sand bars due to the following river works.

(i) Improvement of ground sill (lowering of crest height)

(ii) Installation of impermeable spur dike and/or revetment

(iii) Dredge/excavation of river bed including gravel extraction

6.3 EXISTING RIVER CHANNEL

6.3.1 River Characteristics

The channel improvement should attain not only a river channel, which can confine the design flood discharge, but also a stable channel, which is of less susceptible to river course and bed changes for easy maintenance.

River improvement planning involves analysis of river characteristics on the existing river channel in order to determine the various factors that can make the existing river channel stable and unstable.

The following parameters are therefore taken into account in the planning procedures.

(1) Hydraulic quantities during flood: flow velocity (V) and attractive force (T_*) during flood for design of river structures

(2) Typical scale of channel: channel width (B), water depth (H), bed slope (I_{BM})

(3) Floodplain (high-water channel) characteristics: quality of high-water channel deposits and behavior of high-water channel during flood

(4) Channel alignment: types of meander, relationship between sand bars and a, location of bank erosion and rate of erosion, formation of islands

(5) Channel cross section: scour depth, changes in cross section due to flood

(6) Types of change in longitudinal profile of channel: rate of change, armoring

(7) Others such as small-scale sand wave pattern, sediment discharge, ecosystem, types of human-induced change in channel characteristics and river scope

It has been found that the river features mentioned above can be roughly classified and described in terms of (1) mean annual maximum discharge A_m, 2) bed slope I_{BM} and (3) representative grain diameter of bed material d_{ry},

(1) Mean Annual Maximum Discharge

In Japan, it is found that the bank full discharge, which formed the low-water channel of compound river channel approximately corresponds to the mean annual maximum flood, which is nearly equivalent to 2-year or 3-year return period flood.

The scale of low-water channel of an alluvial river reflects the force of flow water mediated by bed material. Bed material, therefore, is closely related to attractive force acting on the riverbed. For the rivers in Japan, the representative grain diameter (d_{ry}) has close relation with the square of shear velocity (u_*^2), which is equal to the product of gravitational acceleration (g), average water depth (H_{im}) and bed slope (I_{BM}) of mean annual maximum discharge

The bankfull discharge is estimated by changing discharge of the uniform flow or the non-uniform flow computation method as explained in **6.4.4** (**Water Level Computation).** The obtained bankfull discharge should be compared with the mean annual maximum discharge and/or the probable discharge mentioned above.

(2) Riverbed Slope

Riverbed slope is estimated from the average riverbed elevation.

(a) Average Riverbed Elevation

(i) Identifying the River Element

The right and left bank shoulders, shape, base, etc. of the river cross-section should be established first. Determine the base of the river by identifying the bank shoulders, which can accommodate the maximum capacity of the river, the possible bottom width and the side slopes through survey.

The plotted cross-section of the river illustrates the features of the river as shown below.

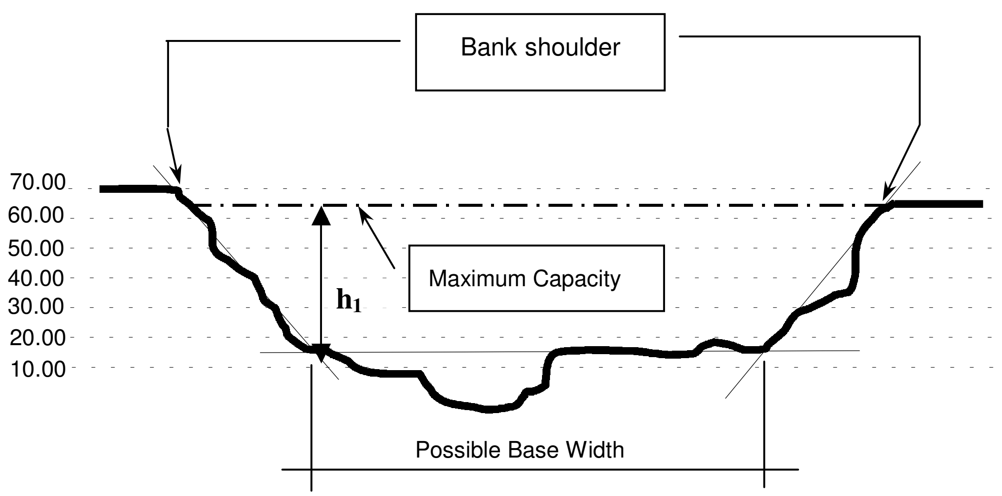


Figure 6.13 River Dimensions

From the plotted cross-section, the average riverbed can be determined. There are many methods used in determining the average riverbed of which the trapezoidal method is popularly used.

(ii) Area Method.

Identify the maximum discharge capacity of the river including the maximum flood level from the plotted cross-section by using planimeter, cross-section millimeter paper, triangulation or any applicable method. Identify the area covered by the maximum capacity of the river, the base (W_2) and the top width (W_1). The depth (h) can be calculated using the formula below: Please refer to **Figure 6.13**

$$Area = \frac{W_1 + W_2}{2}(h_2)$$

$$h_2 = \frac{2 \times Area}{W_1 + W_2}$$

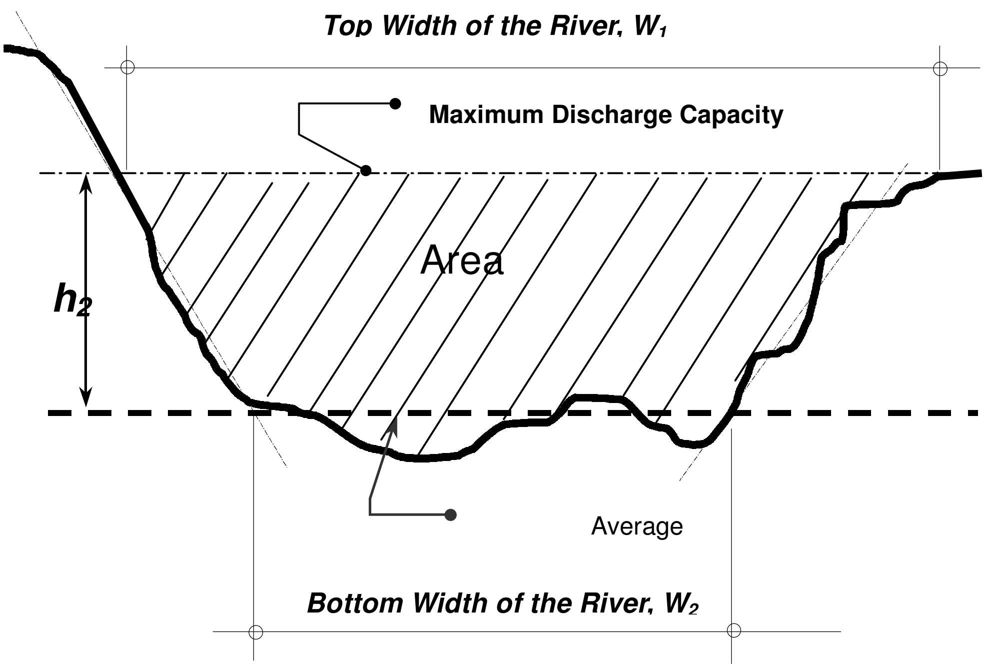


Figure 6.14 Average Riverbed Elevation

Compare h_1 and h_2. If h_1 and h_2 have different values, determination of the river elements should be repeated until the values become very close.

(b) Longitudinal Profile of Average Riverbed Elevation

Using the average riverbed elevation mentioned above, the longitudinal profile of average riverbed elevation is prepared as shown in **Figure 6.15**.

In this figure, the deepest riverbed should be indicated because this will be one of the important parameters in deciding the design foundation depth of revetment.

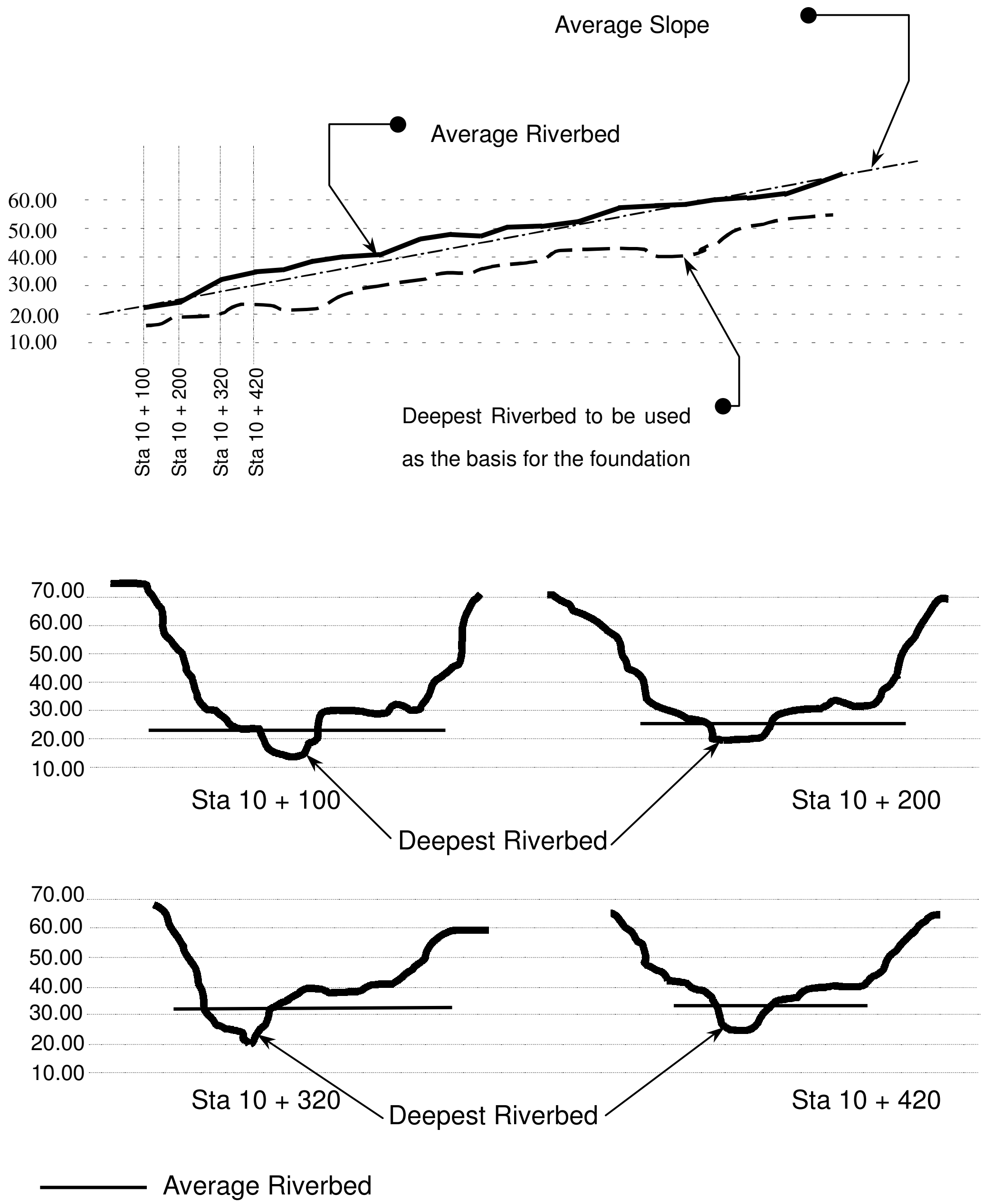


Figure 6.15 Profile of Riverbed Elevation and Average Slope (Example)

(c) Estimation of Riverbed Slope

Based on the longitudinal profile of average riverbed elevation, separate riverbed into several stretches, which falls on the same riverbed slope and then compute river slope of respective stretches.

(3) Representative Grain Diameter of Bed Material

Representative grain diameter of bed is an indicator to show mobility of bed material of river channel and is defined as d_{60}, when the grain size distribution is plotted on logarithmic scale as the abscissa axis against normal scale of the percent passing (or percent finer) as the ordinate axis as shown in **Figure 6.16**.

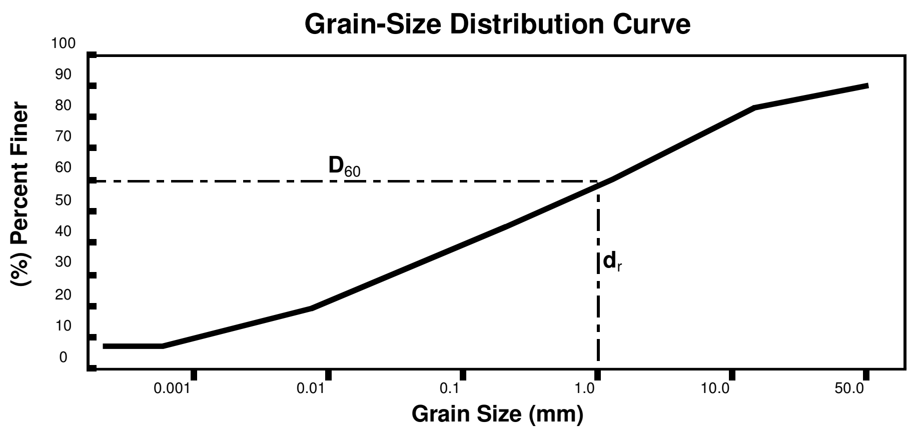


Figure 6.16 Representative Grain Diameter

(a) Longitudinal Interval of Sampling

The grain size survey may be undertaken in the target stretches with an interval of 1 km in the longitudinal directions. This survey site should consider locations where the bed material changes such as river segment change, confluences of tributaries, existing dams and other river structures as well as location where the river structures are planned to be constructed.

(b) Sampling and Grain Size Analysis

At the site, the best spot to get sample is selected from those where sample of riverbed material is exposed and corresponding maximum grain diameter is determined. Then, according to the maximum grain diameter, the following method can be applied

(i) Field Measurement Method (Line Sampling Method)

This method is used when maximum grain diameter of riverbed material is equal to or more than 20 cm. The procedure is as follows.

Using steel tape, determine 20 sampling points with interval of the biggest riverbed material. **Figure 6.17** shows the sampling points for biggest material of 50 cm.

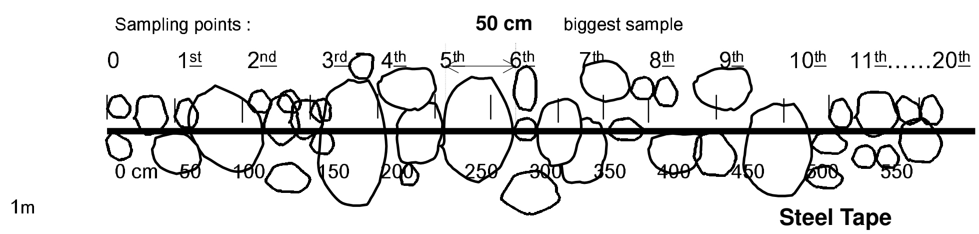


Figure 6.17 Sampling in Field Measurement

Pick the stones beneath the sampling points and compute the equivalent diameter (d) from the following formula.

$$d = \sqrt[3]{x_1 \cdot y_1 \cdot z_1}$$

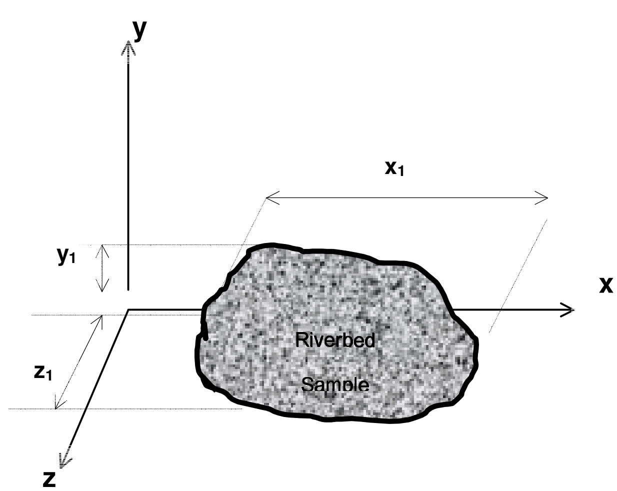


Figure 6.18 Measurement of x_1, y_1 and z_1

Using these diameters, percent finer ($P(d_i)$) corresponding to d_i (i th smallest diameter stone) can be obtained as follows.

$$P(d_i) = \frac{d_1^3 + d_2^3 + \cdots + d_i^3}{d_1^3 + d_2^3 + \cdots + d_{20}^3} \cdot 100$$

(ii) Laboratory Measurement Method

This method is applicable to riverbed materials having less than 20 cm maximum grain diameter.

Get riverbed sample from the best spot and perform grain-size analysis by sieve method. Sieve method can measure the diameter varying from 100 mm (maximum sieve opening d_{max}) to 0.75 mm (minimum sieve opening d_{min}) and for smaller size of d_{min}, the precipitation method is applied in general.

Without using the precipitation method, where an easier measurement of particle size distribution especially representative grain diameter of riverbed material, percent finer can be obtained as follows,.

For gravel containing those larger than the maximum sieve opening d_{max}.

$P(d)$, percent finer of sand gravel finer than d_{max}, is obtained as follows.

$$P(d) = p_o(d) \frac{W_s}{W_s + W_p}$$

$P'(d)$, percent finer of gravel larger than d_{max}, is obtained as follows.

$$P'(d) = \frac{W_s + W_l}{W_s + W_p}$$

where, $P_o(d)$: percent finer for the sieve analysis, W_s: total weight of sample finer than d_{max}, W_L: total weight of sample larger than d_{max} and finer than d, W_p: total weight of sample larger than d_{max}.

Diameter of sample larger than d_{max} is computed by measuring x_1, y_1 and z_1 as explained above.

Using the bankfull discharge Q_m, bed slope I_b and representative grain diameter of bed material d_R, the following important river characteristics can be analyzed/estimated.

(1) River Segment

Based on the river slope obtained above, the target river channel is divided into river segment, namely segment 1 (alluvial fan), segment 2 (valley bottom plain and natural levee) and segment 3 (delta) in accordance with **Table 6.1**. The river segment 2 is further divided, based on riverbed material, into segment 2-1 (gravel riverbed) and segment 2-2 (sand riverbed). Moreover, the respective segments are divided into small segment in consideration of the riverbed slope. The identified river segment is indicated with the longitudinal profile of river bed elevation.

(2) Typical Scale of River Channel

The water surface width (B) and mean depth (H_m) for bankfull discharge or 2 yr/3yr return period flood, are obtained, especially to evaluate the characteristics of movable bed and the sand bar occurrence together with the river segment classification.

(3) Hydraulic Quantities during Flood

The average velocity of the low-water channel of a compound section channel corresponding to the mean annual maximum flood and the assumed design flood (roughly assumed twice the depth of mean annual maximum discharge) can be estimated from the representative riverbed material (d_R) and the average riverbed slope (I_b) using **Figure 6.6**.

d_R = 1 cm or less; The velocity, V = 2m/sec or less at the annual maximum flood.

d_R = 0.5 mm or less; Roughly V = 3 m/sec or less at the design discharge.

d_R = 0.5 mm – 4mm; Roughly V = 4m/sec at the design discharge by making the riverbed plain.

d_R = 1 cm or more; V = 2m/sec or more at the annual maximum flood; V = 3m/sec or more at the design discharge.

d_R = 3 - 4 cm or more; When the riverbed gradient becomes steeper than 1:250, V = 5 m/sec or more at the design discharge.

(4) Meandering and Sand Bars

The type of sand bars and possibility of the meandering can be estimated from the river segment and ratio of water surface width (B) to mean depth (H_m) of the mean annual maximum flood (**6.2.2 (2) (b)** Characteristics of Sand Bar).

(5) Channel Cross Section

The possible location of scouring and some of the scouring depth can be estimated based on the explanation of **6.2.2 (4) Erosion**).

(6) Change of Longitudinal Profile

Location of sediment deposition and scouring can be quantitatively estimated as explained in **6.2.2 (1)** Longitudinal Profile.

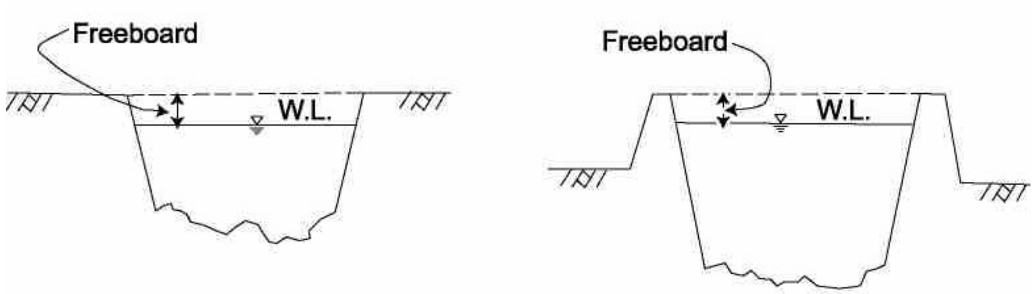
6.3.2 Flow Capacity

Flow capacity of the existing river channel should be estimated to identify the reaches lacking safety level and to consider the possible alternatives to accommodate the design discharge. Flow capacity of the existing river channel shall be basically estimated by non-uniform flow method (refer to **6.4.4**) except the channel with uniform shape and slope, for which uniform flow method can be applied.

The water level used to estimate flow capacity shall be as explained below.

(1) Reaches without dike : Ground elevation minus freeboard

(2) Reaches with dike : Dike elevation minus freeboard



(a) Non-diked River (b) Diked River

Figure 6.19 Water Level for Flow Capacity Evaluation

To get the flow capacity of respective cross-sections, first estimate water levels at these sections corresponding to the discharge for a several return periods, such as 2-yr, 5-yr, 10-yr, 25-yr, 50-yr and 100-yr return periods. Then, find two (2) *WL*s, which is higher (WL_{n+1}) and lower (WL_n) than the water level for flow capacity (WL_f) explained above. Using these *WL*s, flow capacity can be computed as follows.

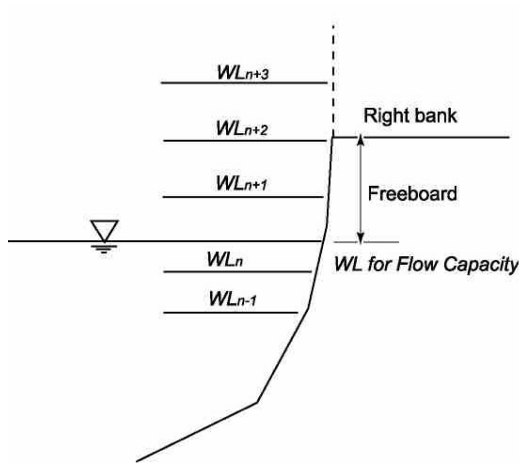


Figure 6.20 Flow Capacity Computation

$$FlowCapacity = Q_n + \left(\frac{WL_f - WL_n}{WL_{n+1} - WL_n} \right) \cdot \left(Q_{n+1} - Q_n \right)$$

Where, Q_n and Q_{n+1}: Discharges corresponding to WL_n and WL_{n+1}, respectively.

Estimated flow capacity shall be indicated in separate figures for left and right banks as follows with design flood or floods with several return periods.

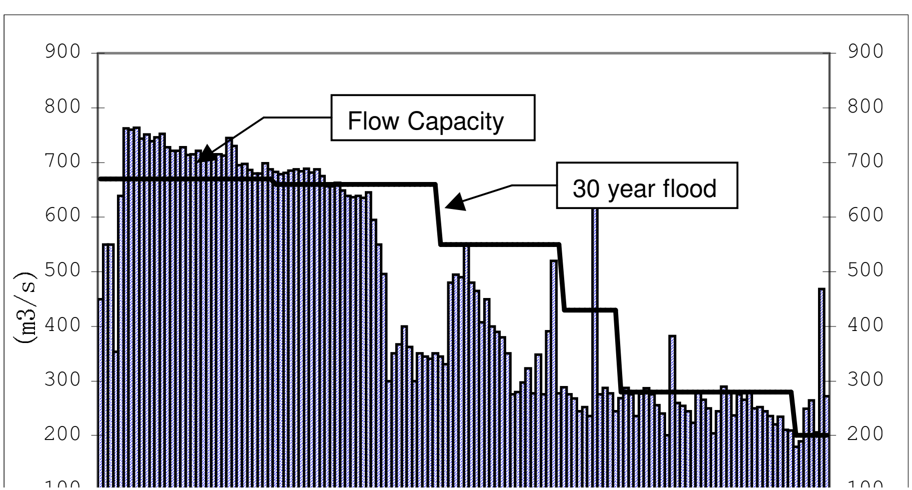


Figure 6.21 Flow Capacity Diagram

6.4 RIVER CHANNEL PLANNING

6.4.1 Planning Procedure

The procedure to formulate the river improvement plan covers the following steps.

(1) Setting the improvement stretch of the project

(2) Setting the river channel route

(3) Setting the alignment of river

(4) Setting the longitudinal profile consisting of design flood level and riverbed slope

(5) Setting the river's cross section

6.4.2 Improvement Stretch

The improved stretch should be determined based on the flood prone area to be protected and the flow capacity of the existing river channel. A continuous river improvement plan shall be formulated, since the discontinuity of the river improvement plan, in which the stretch with flow capacity less than the design discharge still causes inundation. Also the stretch for necessary improvement should be connected to another stretch with flow capacity equal to or more than the design discharge, like mountain connected dike so that flood water overflowing would not attack the flood prone area as shown in **Figure 6.22**.

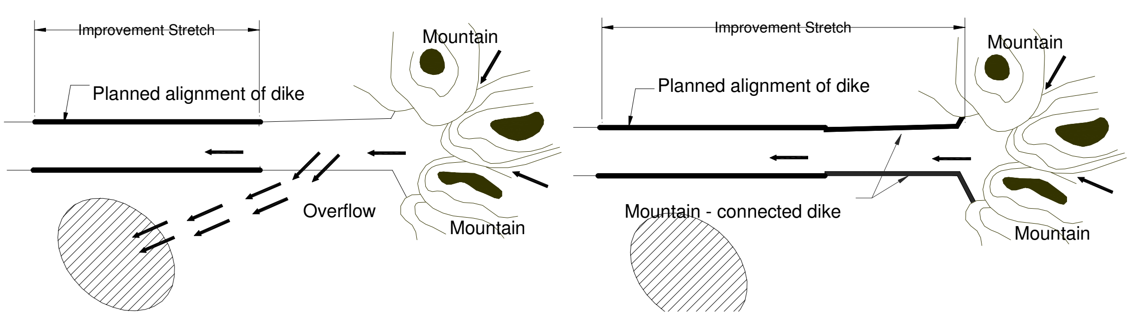


Figure 6.22 Determination of Improvement Stretch

6.4.3 River Channel Route

River channel route should basically follow the existing one. If there is a problem on the existing land use and flow disruption because of sharp meandering, then cut off channel shall be considered. Several routes should be set by combining the portions of existing river use and the portions of new river excavation. To select the best route the following conditions shall be taken into account:

(1) topographic and geologic reasonableness,

(2) considerations for the present and future land uses,

(3) administrative district,

(4) irrigation and drainage systems,

(5) influence to groundwater level,

(6) countermeasure against inner waters,

(7) influence to the upper and lower reaches of the planned section,

(8) project cost for improvement, maintenance after improvement .

For setting the improvement route, the following matters should be considered:

(1) Alignment must be set smoothly with minimal meanderings.

(2) As much as possible, the improved river channel route should be far away from a densely populated area.

(3) The embankment sections shall be a mountain-connected dike as practically as possible (**Figure 6.22**).

(4) The river with wide width and high velocity due to steep slope shall be planned to have open dikes: 1) to reduce discharge in the downstream stretch in case of flood exceeding design by storing flood water in the neighboring areas through the openings and 2) to accept easily the inland flood water as well as excess flood through the openings (**Figure 6.23**). .

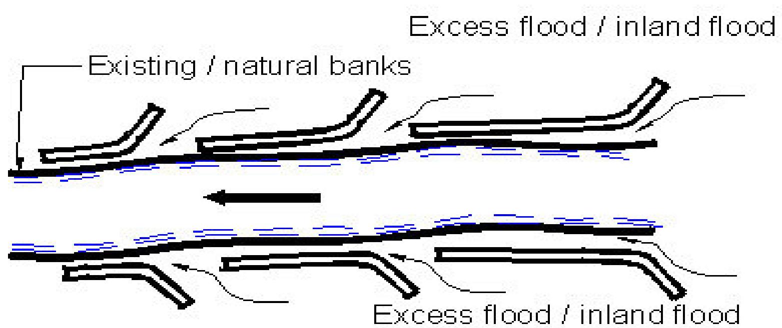


Figure 6.23 Open Dike

6.4.4 Water Level Computation

After the river channel route is determined, the alignment, the longitudinal profile and the cross section are jointly studied in formulating the optimum plan, applying the water level computation method, namely, the non-uniform flow method or the uniform flow method.

(1) Flow Classification

Water level of river channel is basically computed by non-uniform flow computation method.

(a) Type of Flow

Open-channel flow can be classified as follows according to the change in flow depth with respect to time under consideration and along channel.

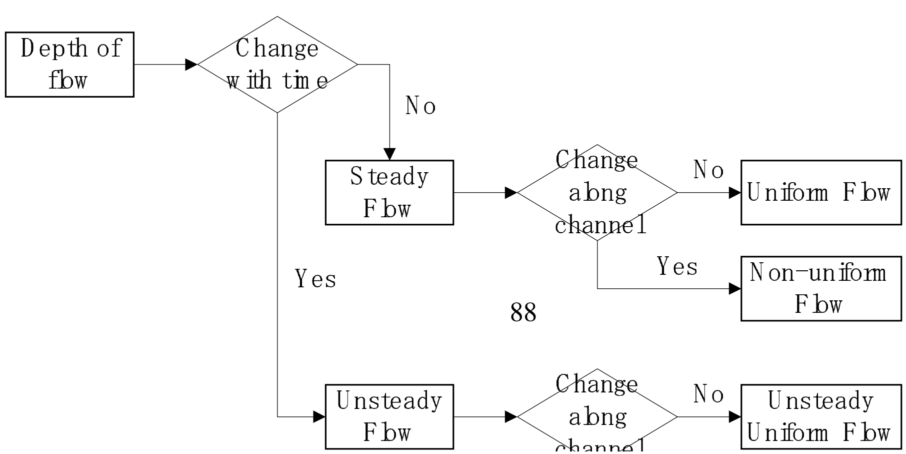


Figure 6.24 Type of Flow

In general, the shape and slope of river vary along the channel and water surface is influenced by that of downstream sections. Therefore, the non-uniform flow method shall be basically applied for determination of design flood water level as well as estimation of flow capacity of the existing river channel.

(b) Flow Profile and Control Section

Water surface profile of non-uniform flow is classified into thirteen types according to the channel bed slope and zones that is classified by the normal depth (depth of uniform flow) and the critical depth (**Figure 6.25**).

When discharge and channel shape are given, critical depth h_c is determined from the following equation.

$$F_r = \frac{V}{\sqrt{gD}} = 1; \quad D = h_c$$

Where, V = mean velocity (m/s) = Q/A, g = acceleration of velocity (m/s/s), D = A/T, Q = discharge (m^3/s), A = cross sectional area (m^2) and T= water surface width (m).

When discharge, channel conditions including channel bed slope are given, normal depth (h_o) is determined from Manning's equation as described below.

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

Where, n = Manning's roughness coefficient, R = hydraulic radius (m), i = channel bed slope.

When slope of channel is changed from the horizontal slope to adverse slope for the given discharge and channel shape, normal depth h_o is deeper than the critical depth h_c at first, but gradually h_o decreases and becomes equal to h_c in a certain slope, namely the critical slope, after which, h_o becomes shallower than h_c.

In this way, five (5) different bed slope can be defined as 1) horizontal slope, 2) mild slope which is milder than the critical slope but steeper than the horizontal slope, 3) critical slope, 4) steep slope, which is steeper than the critical slope and 5) adverse slope. And on the respective slope of channel, h_o and h_c lines divide the space in a channel into three (3) zones:

Zone 1: The space above the upper line
Zone 2: The space between the two lines
Zone 3: The space below the lower lines

Thus, thirteen flow profiles are defined according to the five (5) types of bed slope and three (3) zones in which the flow surface lines as indicated in **Figure 6.25**. The flow profile is designated by letters for slopes: H for horizontal, M for mild, C for critical, S for steep, and A for adverse; and the number represents the zones from 1 to 3.

As indicated in **Figure 6.25**, flow profile shows behavior of flow at several specific depth. Knowledge of the flow profile is so helpful to evaluate the computation results of non-uniform flow.

(i) At the normal depth h_o, flow surface is parallel to the riverbed

(ii) At the critical depth h_c, a hydraulic jump or drop occurs and flow profile is vertical

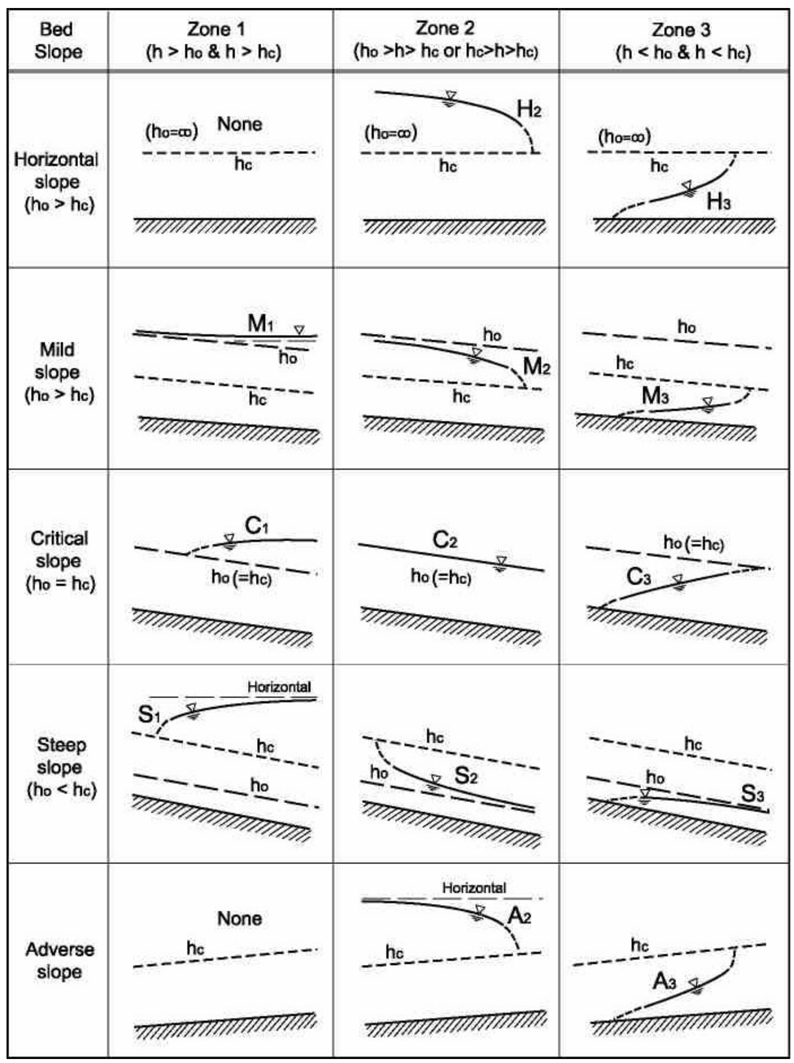


Figure 6.25 Flow Profile

(Source: Open Channel Hydraulics by Ven Te Chow)

Flow Profiles

M Profiles

M1 is commonly known as the backwater curve. It occurs at the downstream end of a long mild channel when submerged in a reservoir at a greater depth than the normal depth of the flow channel. Examples of M1 are the profiles behind the dam in a natural river and in a canal joining reservoirs.

M2 occurs when the bottom of the channel at the downstream end is submerged in a reservoir at a depth less than the normal depth. Examples are the profiles at the upstream side of a sudden enlargement of a canal segment and the profile in a canal leading to a reservoir, where the pool level is show both above and below the critical-depth line.

M3 occurs when a supercritical flow enters a mild channel. Examples of M3 are the profile in a stream below a sluice and after the change in bottom slope from steep to mild.

S1 begins with a jump at the upstream and becomes tangent to the horizontal pool level at the downstream end. Examples are the flows behind a dam in a steep channel and in a steep canal emptying into a pool of high velocity.

S2 is usually very short and like a transition between a hydraulic drop and uniform flow. Examples are profile on the downstream side of an enlargement of channel section and on the steep slope side as the channel slope changes from steep to steeper.

S3 is formed between an issuing supercritical flow and the normal depth line to which the profile is tangent. Examples are profile on the steep slope side as the channel slope changes from steep to milder steep and below a sluice with the depth of the entering flow less than the normal depth on a steep slope.

C profiles represent the transition conditions between M and S profiles.

H profiles are limiting cases of M profiles.

H2 profiles appear at the upstream side of the changing point of the slope gradient from horizontal to steep such as the upstream of the weir or the head type groundsill.

H3 profiles appear at the downstream side of the gate or the downstream side of the changing point of the slope gradient from steep to horizontal.

A2 and A3 profiles are similar to H2 and H3.

(c) Usage of Uniform Flow Computation

Uniform flow computation may be used to estimate water level of rivers under the following conditions.

(i) River of steep slope with supercritical flow in all reaches under consideration

(ii) River of mild slope with uniform shape and slope along channel

The bed slope where supercritical flow appears is roughly estimated from the following equation.

$$i > n^2 g R^{-1/3}$$

Where; i = Channel bed slope

n = Manning's roughness coefficient

g = gravitational acceleration (9.8 m/s^2)

R = hydraulic radius (m)

(2) Non-Uniform Flow Computation

(a) Energy Equation for Non Uniform Flow

Water level profiles are computed from one cross-section to the next by solving the Energy equation. The Energy equation is written as follows:

$$Y_2 + Z_2 + \frac{V_2^2}{2g} = Y_1 + Z_1 + \frac{V_1^2}{2g} + h_e$$

Where; Y_1, Y_2 = depth of water at cross section (m)

Z_1, Z_2 = elevation of the main channel bed (invert) (m)

H_1,H_2 = water level (=Y_1+Z_1 and Y_2+Z_2, respectively)(m)

V_1, V_2	= average velocity (m/s)
g	= gravitational acceleration (m/s^2)
h_e	= energy head loss (m)

A diagram showing terms of the energy equation is shown below.

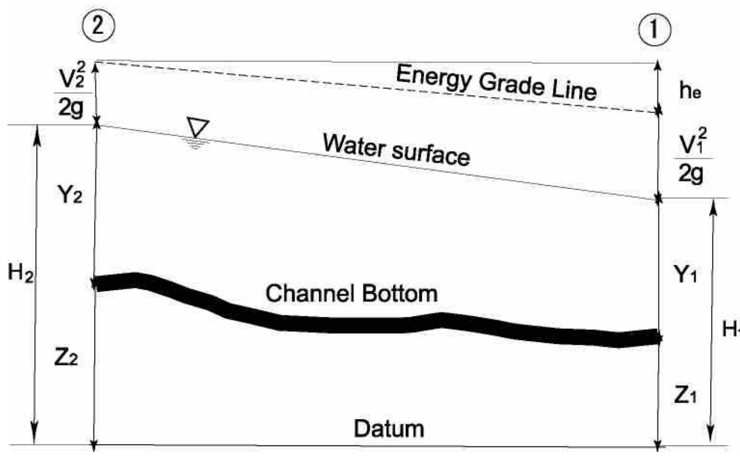


Figure 6.26 Terms in the Energy Equation

The energy head loss (h_e) between two cross sections is composed of friction losses.

$$h_e = L\overline{S_f}$$

Where, L = reach length

$\overline{S_f}$ = representative friction slope between two sections

The friction slope (slope of the energy grade line) at each cross section is computed from Manning's equation as follows:

$$S_f = \left(\frac{Q}{K}\right)^2$$

Where, Q = discharge (m^3/s)

K = conveyance $\left(= \frac{AR^{2/3}}{n}\right)$

There are several equations to compute representative reach friction slope, of which the following equations are generally applied.

(i) Average Friction Slope Equation

$$\overline{S_f} = \frac{S_{f1} + S_{f2}}{2}$$

(ii) Average Convergence Equation

$$\overline{S_f} = \left(\frac{Q_1 + Q_2}{K_1 + K_2}\right)^2$$

(b) Computation Procedure

In energy equation, cross sectional area (A) and hydraulic radius are functions of water level (H)

$$A = f(H)$$

$$R = f(H)$$

The average velocity (V) and friction slope ($\overline{S_f}$) are functions of water level (H) as shown below.

$$V = Q / A = f(A) = f(H)$$

$$S_f = f(K) = f(A,R) = f(H)$$

Therefore, if H_2 is given, an unknown value H_1 can be solved using the energy equation and vice versa.

Water level at the downstream influences the upper reaches in a subcritical flow and conversely, vice versa in a supercritical flow. Therefore, in a subcritical flow regime, boundary condition (starting water level, explained in **6.4.4 (5)** Starting Water Level is required at the downstream end of the river system. If a supercritical flow regime is to be calculated, the boundary condition is necessary at the upstream end of the river system.

In a river stretch, using the energy equation the unknown water level is determined by iteration solution, starting from a known starting water level. The following is the computational steps for a subcritical flow. For a supercritical flow, computation is made from the upstream cross section.

(i) Step 1

Compute the cross sectional area (A_1) and the hydraulic radius (R_1) for downstream cross section 1. (H_1) corresponds to the given water level, which is the starting water level or obtained water level in previous set of computation.

(ii) Step 2

Compute the velocity V_1 from a given discharge Q_1 and the obtained A_1 based on Continuity Equation (Q=AV)

(iii) Step 3

Assume a water level (H_{2A}) at the upstream cross section.

(iv) Step 4

Compute A_2, R_2, and V_2, corresponding to H_{2A} and Q_2 .

(v) Step 5

Compute friction loss head, corresponding H_1, Q_1, H_{2A} and Q_2.

(vi) Step 6

Compute water level H_{2C} using the Energy Equation with values obtained above.

(vii) Step 7

Compare the water level (H_{2A}) from step 3 and the water level (H_{2C}) from step 6. Repeat steps 1 through 6 until the difference of water levels agrees within the predetermined value.

Using the obtained water level H_2 as given water level H_1, computation is continued up to the upper stream end to obtain all the water level of a target stretch.

(3) Cross Sectional Data

(a) Preparation of Cross Section Data

In a non-uniform flow computation, area (A) and hydraulic radius (R) with water levels (H) for respective sections of a target stretch are necessary data as explained above. These are calculated from data set of (X, Y) prepared based on the cross sectional survey. The datum of the survey should be the same as the starting water level. Computer Program for non-uniform calculates area (A) and hydraulic radius (R) from the given data of (X,Y).

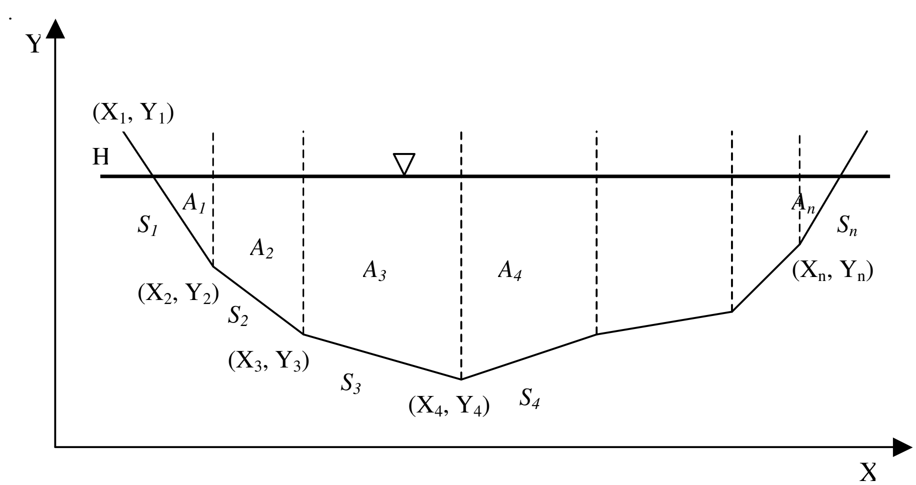


Figure 6.27 Cross Section Data Set

(b) Interval between Sections

Interval of cross-sectional survey for river improvement planning or design of river structures is recommended to be undertaken as follows and basically non-uniform flow computation is expected to be used with these intervals.

Table 6.3 Interval of Cross Sectional Survey

River Condition	River Width		
	Less than 50 m	50 – 200 m	200 m or more
Strait, uniform	50 m	100 m	200 m
Strait, irregular	50 m	50 m	200 m
Bend	25 m	50 m	100 m
Inland survey width	25 m	50 m	50 m

For the master plan, interval of cross-sectional survey can be extended as much as 500 m.

(c) Interpolation of Sections

When a control section appears, or when a large energy gradient occurs, cross-sections should be interpolated to obtain better computation results.

The following figure shows the difference of water surface by adding the interpolated sections with different interval (ΔX). In this case, the critical depth occurs at the point of X=0 where a groundsill is located in a river course with a mild slope of 1/10,000. As explained in **Flow Profile**, please see Figure 6.25, flow profile should be *M2*. But in cases for ΔX=250 m and 200 m, flow profile is similar to *M1* due to large energy gradient. When sections are interpolated in the cases of ΔX =50 m and 10 m, M2 flow profile appears, which is considered to be correct.

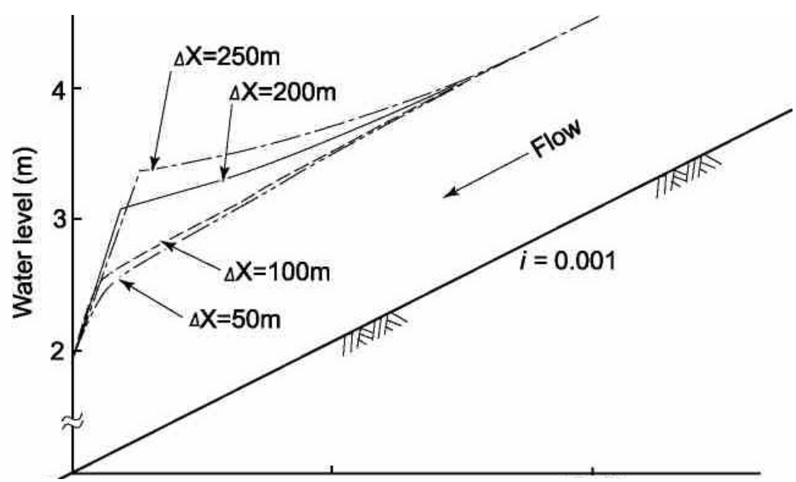


Figure 6.28 Change of Flow Profile due to Interpolated Sections

(d) Ineffective Flow Area (Dead Water Zone)

When the shape or the slope of a river channel abruptly changes, energy loss occurs due to the separation of flow streamlines in addition to the friction loss. This loss may be computed as the contraction or expansion loss, using “contraction or expansion loss coefficient”. However, this coefficient is difficult to estimate for the actual river channels and thus water level change is estimated considering reduction of cross sectional areas due to occurrence of the ineffective area.

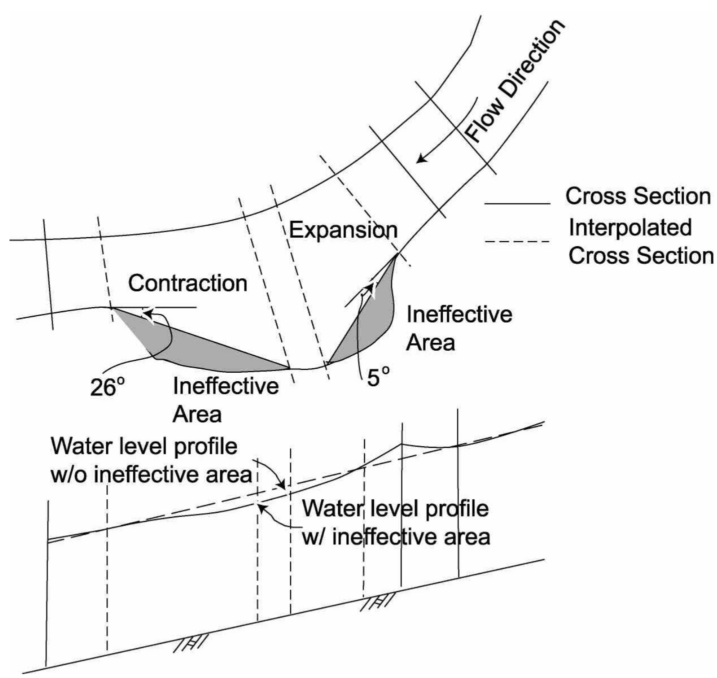


Figure 6. 29 Ineffective Area and Water Level Change

The ineffective area occurs in the expansion or contraction reaches as shown above and estimated as follows.

Flow starts to expand with angle of 5° from the streamlines of upper reaches and contracts with angle of 26°.

If no surveys are made at cross sections of the expansion or contraction reaches, non-uniform computation should be made to evaluate energy loss due to expansion and/or contraction by interpolating cross sections as indicated above.

(4) Roughness Coefficient

Manning's roughness coefficient shall be determined with emphasis on the analysis of flood marks of the past floods when the flow capacity of the existing river is estimated, provided the data of the past floods are few or not accurate. The following roughness coefficients are recommended. Also these values are applied to determine the Design Flood Level of a channel after improvement.

Table 6.4 Values of Manning's Roughness Coefficient "n"

Surface/Description	Range Min.	Max.
1) Natural stream channels (top flood width less than 30 m)		
(i) Fairly regular section:		
a. Some grass and weeds, little or no brush	0.030	0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035	0.050
c. Some weeds, light brush on banks	0.035	0.050
d. Some weeds, heavy brush on banks	0.050	0.070
e. Some weeds, dense trees	0.060	0.080
f. For trees within channel, with branches submerged at high flood increase all above values by	0.010	0.020
(ii) Irregular sections, with pools, slight channel meander; increase values given above about	0.010	0.020
(iii) Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high flood:		
a. Bottom of gravel, cobbles, and few boulders	0.040	0.050
b. Bottom of cobbles, with large boulders	0.050	0.070
2) Larger stream channels (top flood width greater than 30 m)		
Reduce smaller stream coefficient by 0.010		
3) Flood Plains (adjacent to stream beds)		
Pasture, short grass, no brush	0.030	0.035
Pasture, tall grass, no brush	0.035	0.050
Cultivated land - no crop	0.030	0.040
Cultivated land – nature field crops	0.045	0.055
Scrub and scattered bush	0.050	0.070
Wooded	0.120	0.160
4) Man-made channels and ditches		
Earth, straight and uniform	0.017	0.025
Grass covered	0.035	0.050
Dredged	0.025	0.033
Stone lined and rock outs, smooth & uniform	0.025	0.035
Stone lined & rock cuts, rough and irregular	0.035	0.045
Lined—metal corrugated	0.021	0.024
Lined—smooth concrete	0.012	0.018
Lined—grouted riprap	0.017	0.030
5) Pipes:		
Cast iron	0.011	0.015
Wrought iron	0.012	0.017
Corrugated steel	0.021	0.035
Concrete	0.010	0.017

Source : Design Guidelines, Criteria and Standards, MPWH 1987, P.746

For the channel with compound cross section with different n-values as shown below, the equivalent roughness coefficient N is obtained, dividing into parts of which wetted perimeter *P*, area *A* and n-values as follows.

$$N = \frac{A_t \cdot R^{2/3}}{\sum \left(\frac{A_t}{n} \cdot R^{2/3} \right)}$$

$$A_t = \sum A_i$$

$$R = \frac{A_t}{P_t} = \frac{\sum A_i}{\sum P_i}$$

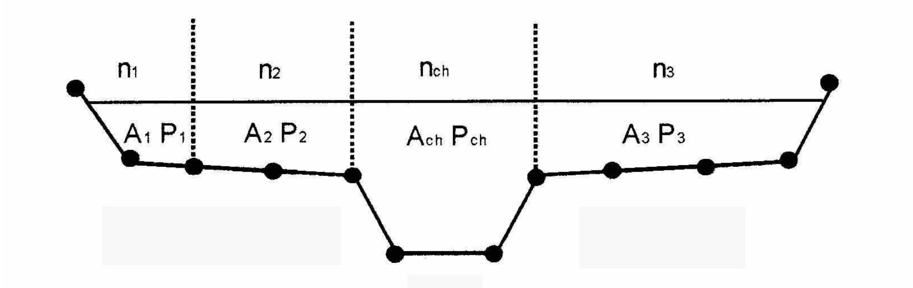


Figure 6.30 Composite Roughness Coefficient

The equation above is obtained assuming that each part of the area has different mean velocities but without shear stress acting on the boundary of neighboring water parts.

(5) Starting Water Level

Starting water level for non-uniform computation is selected from one of the following water levels; (a) water level at the river mouth, (b) water level at the junction with the main river channel and (c) other water levels such as uniform flow water level at the uniform stretch or the critical water level at the control section.

(a) Water Level at the River Mouth

The mean monthly highest water level (MHW) should be basically used for the starting water level at the river mouth. The MHW is the average of the monthly-highest water level, where the monthly-highest water level for a particular month is defined as the highest water level occurring in the period from 2 days before the day of the lunar syzygy (new moon and full moon) to 4 days after the day of the lunar syzygy.

The NAMRIA has 41 tide stations as tabulated in **Table 6.5** and shown in **Figure 6.31**. When one of the tide stations is located near the river mouth of the target river, get the MHW measured from the primary tidal benchmark listed in **Table 6.5** from the Oceanography Division, NAMRIA. After connecting the primary benchmark with cross sectional survey of the target river, use the MHW as the starting water level.

When no tide station is located near the target river mouth, to get the necessary starting water level, tide observation for short duration may be conducted. Then correlate the MHW of the target river mouth to that of the existing tide gauge. The obtained MHW is connected to the benchmark for the cross sectional survey to be used to get the stating water level. Regarding the tide observation, consult the Oceanography Division, NAMRIA.

Table 6.5 Location of Primary and Secondary Tidal Gauge

No	Name	Type	Latitude	Longitude	Locality	Primary Tidal BM
1	Manila	P	14.58333	120.96667	Pier 15, South Harbor, Manila	BM 4B
2	Cebu	P	10.30000	123.91667	Pier 1, Cebu City	BM 3A
3	Legaspi	P	13.15000	123.75000	Legaspi Port, Legaspi City	BM 22
4	Davao	P	7.08333	125.63333	Sasa Port, Davao City	BM 17
5	San Fernando	P	16.61667	120.30000	Port Poro, San Fernando, La Union	BM 8
6	Jolo	P	6.06667	121.00000	Jolo, Wharf, Jolo, Sulu	BM 1
7	Tacloban	P	10.25000	125.00000	Tacloban Port, Tacloban City	BM 4
8	San Vicente	P	18.51667	122.13333	San Vicente Port, Sta. Ana, Cagayan	BM 5
9	Surigao	P	9.78333	125.50000	Surigao Port, Surigao del Norte	BM 3
10	San Jose	P	10.33333	121.08333	San Jose, Occidental Mindoro	BM 3
11	Port Irene	P	16.38333	122.10000	Sta. Ana, Cagayan	BM 1
12	Puerto Princesa	P	17.75000	118.73333	Puerto Princesa. Palawan	BM 1
13	Balabac	S	8.00000	117.06667	Balabac Pier, Palawan	BM 3
14	Tagbilaran	S	9.65000	123.85000	Tagbilaran Pier, Bohol	506-D
15	Ozamiz City	S	8.13333	123.85000	Ozamiz Pier, Ozamiz City	BM 3A
16	Solvec	S	17.45000	120.45000	Solvec. Narvacan, Ilocos Sur	BM 2
17	San Narciso	S	11.58333	122.56667	San Narciso, Quezon	BP 50
18	North Harbor	S	14.60000	120.95000	Pier 4, North Harbor, Manila	GM 3B A
19	Catanauan	S	13.56667	122.31667	San Antonio. Catanauan, Quezon	BP 11
20	Navotas	S	14.68333	120.93333	Navotas, Metro Manila	JL 6

No	Name	Type	Latitude	Longitude	Locality	Primary Tidal BM
21	Real	S	14.66667	121.61667	Real, Quezon	BM 1
22	Baler	S	15.75000	121.58333	Baler, Aurora	BM 1BAL
23	Basco	S	20.45000	121.96667	Basco, Batanes	BM BAS3
24	Claveria	S	18.60000	121.06667	Claveria, Cagavan	CLAV BM1
25	Palanan	S	17.11667	122.03333	Palanan. Isabela	BM PAL1
26	Pag-asa Island	S	11.03333	114.28333	Pagasa Island, Kalayaan Group	BM 1
27	Batangas	S	13.76667	120.96667	Sta, Clara Pier, Batangas City	BM 1
28	Dumaguete	S	9.30000	123.30000	Dumaguete Port, Dumaguete City	BM 5
29	Cagayan de Oro	S	8.50000	124.66667	Macabalan Pier, Cagavan de Oro City	BM 3
30	Iligan	S	8.21667	124.21667	Iligan Bay, Iligan City	BM 4
31	Banago	S	10.70000	122.93333	Banago Pier, Bacolod City	BM IA
32	Batan	S	11.58333	122.50000	Port Batan, Akian	AK 12
33	Bislig	S	8.20000	126.36667	Bislig Bay, Bislig	BM 3
34	Basilan	S	6.70000	121.96667	Isabela, Basilan	BM 5
35	Virac	S	13.58333	124.23333	Virac Pier, Catanduanes	CA 4
36	Currimao	S	17.98333	120.48333	Gaang Bay, Currimao	BM 2
37	Penascosa	S	9.76667	118.51667	Penascosa Pier, Palawan	BM 2
38	Laoang	S	12.58333	125.00000	Laoang, Samar	BM 1
39	Iloilo	S	10.70000	122.58333	Iloilo Harbor, Iloilo City	BM 3
40	Ormoc	S	11.00000	124.60000	Ormoc Pier, Ormoc City	OC 3
41	Subic Bay	S	14.81667	120.28333	Subic Bay, Olongapo City	BM 2

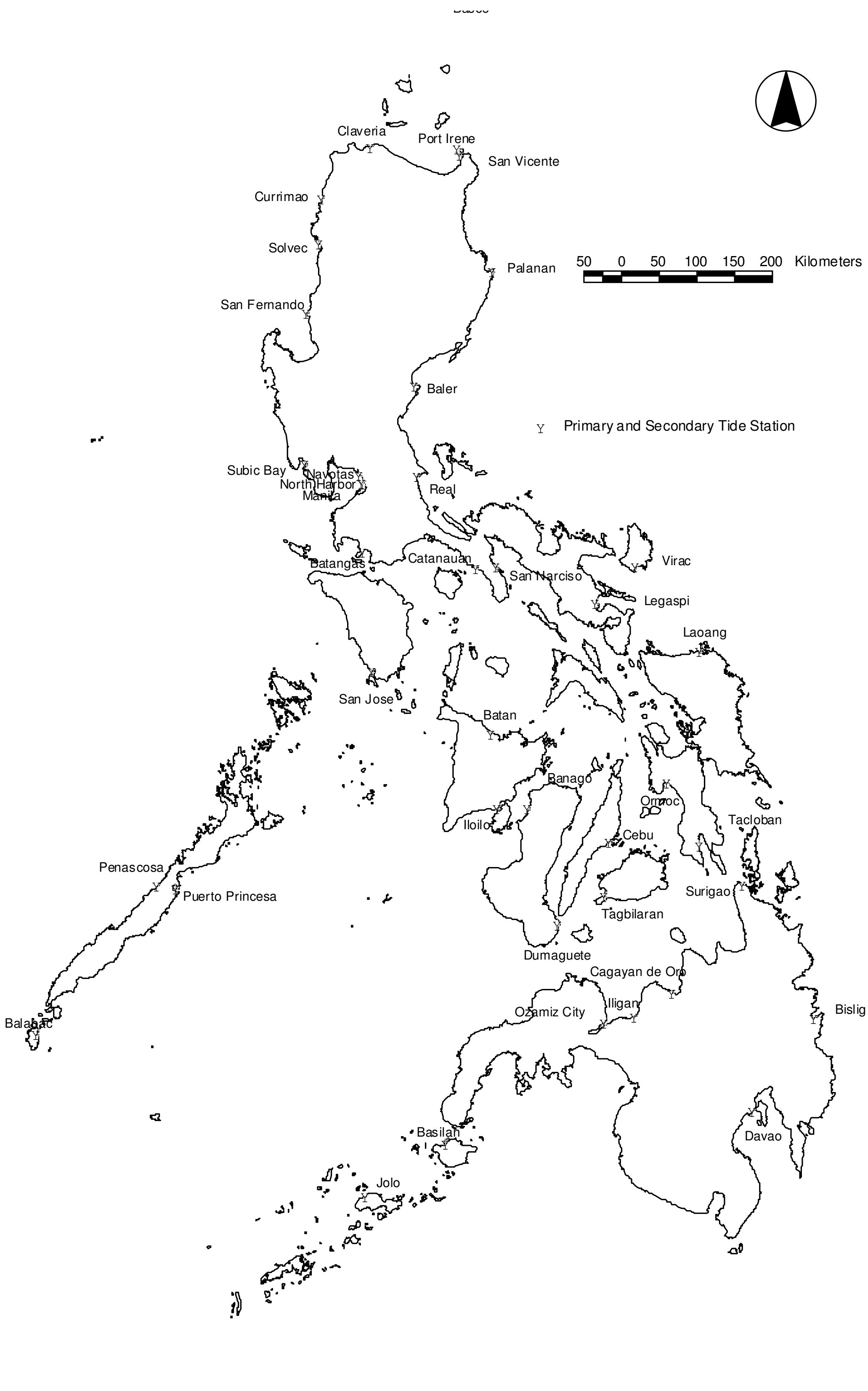


Figure 6.31 Location of Tidal Gauge

When training dikes exist or are planned to be constructed or when sand bars exist at the river mouth, the following consideration should be added to the MHW.

(i) Training Dike

When no dike is constructed, starting water level should be at Point A, but when a dike is constructed or planned to be constructed, the starting water level should be at point B, the end point of training dike. See **Figure 6.32**:Location of Starting Water Level.

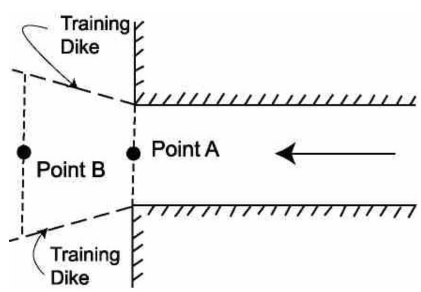


Figure 6.32 Location of Starting Water Level

(ii) Sand Bar

When sand bar is located at the river mouth or at the end point of training dikes, starting water level can be computed as follows.

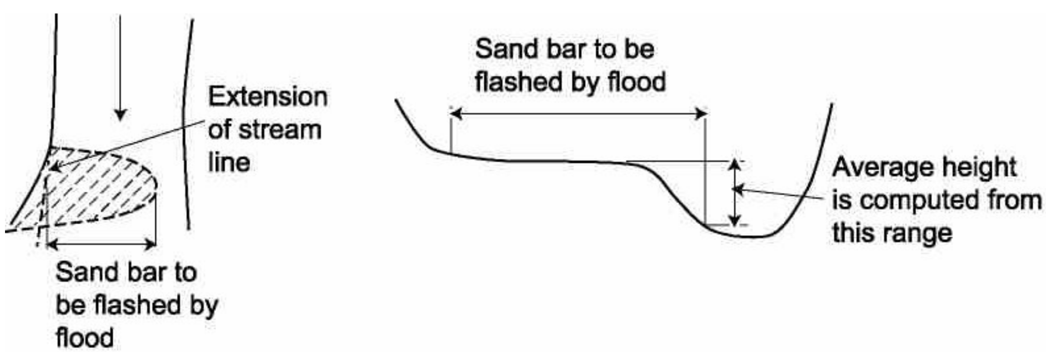


Figure 6.33 Starting Water Level for Sand Bar

- Estimate the range of sand bar to be flashed out during the flood by extending flow lines
- Estimate the average height of sand bar to be flashed out
- Compute the starting water as the average height of sand bar plus 0.5 m

(b) Water Level at the Junction with Main River

The starting water level of a tributary should be set in consideration of the water level of the main river, for which the Design Flood Level has been formulated.

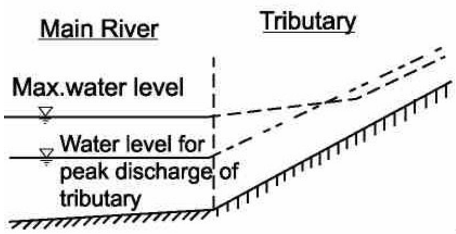
(i) Tributary without Flood Control Gate

Two (2) cases of non-uniform flow computation should be made to find the highest water level of the tributary, namely 1) the design flood level (DFL) of the main river and the tributary discharge just when the DFL of the main river occurred, and 2) peak discharge of the tributary and water level of the main river just when the peak discharge occurred.

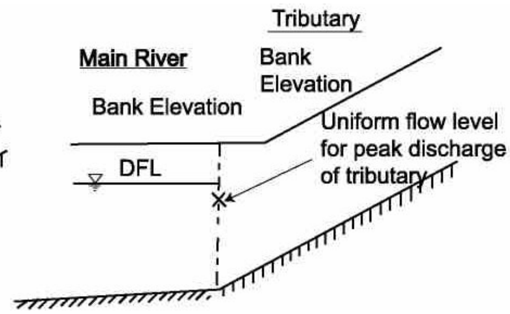
When the relationship of the discharge of the tributary and the water level of the main river mentioned above is unclear, the starting water level is selected as the higher water level between (1) the DFL of the main river and (2) uniform flow water level corresponding to the peak discharge of the tributary. Then non-uniform flow computation is undertaken with the peak discharge of the tributary and the starting water level.

(ii) Tributary with Flood Control Gate

A flood control gate is closed when water level of a main river is higher than that of a tributary. Therefore, water level of the main river is not considered for this type of tributaries and starting water level is set as a uniform flow water level corresponding to peak discharge of the tributary.



(a) Tributary without Gate



(b) Tributary with Gate

Figure 6.34 Starting Water Level of Tributary

(c) Other Water Levels

The methods discussed below deal with computation of water level in a certain stretch of the river where, more or less, the uniform flow occurs and the portion where the sea or the river mouth is far away.

(i) Uniform Flow Water Level

This water level is calculated on the stretch with mild slope and approximately uniform shape and side slopes along the channel with a length of almost at least 100 m.

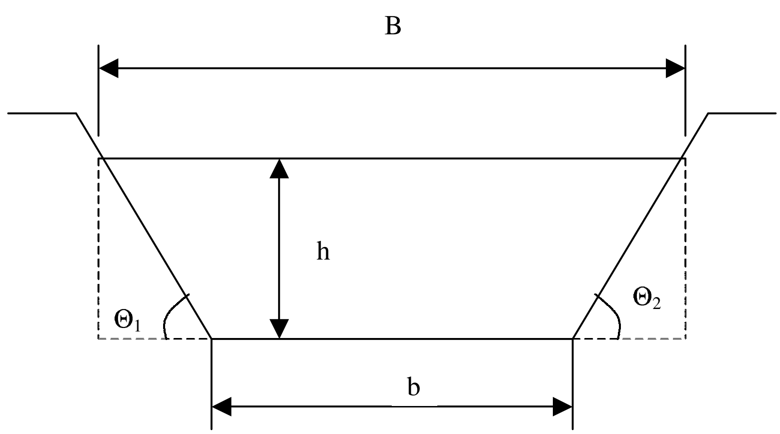


Figure 6. 35 Sample Cross-Section

Discharge Q is given

Where Q = AV

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} = \frac{1}{n} \left(\frac{A}{P} \right)^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$A = \frac{B+b}{2} h$$

$$B = b + \frac{h}{\tan\theta_1} + \frac{h}{\tan\theta_2}$$

$$A = \frac{1}{2}\left(b + \frac{h}{\tan\theta_1} + \frac{h}{\tan\theta_2} \right) h = \frac{h}{2}\left(2b + \frac{\tan\theta_1 + \tan\theta_2}{\tan\theta_1 \tan\theta_2} h \right)$$

$$P = b + \frac{h}{\sin\theta_1} + \frac{h}{\sin\theta_2} = b + \frac{\sin\theta_1 + \sin\theta_2}{\sin\theta_1 \sin\theta_2} h$$

$$Q = \frac{S^{\frac{1}{2}}}{n} \frac{1}{P^{\frac{2}{3}}} A^{\frac{5}{3}}$$

$$Q = \frac{S^{\frac{1}{2}}}{n} x \frac{1}{\left(b + \frac{\sin\theta_1 + \sin\theta_2}{\sin\theta_1 \sin\theta_2} \right)^{\frac{2}{3}}} x \left[\frac{h}{2} x \left(2b + \frac{\tan\theta_1 + \tan\theta_2}{\tan\theta_1 tain\theta_2} h \right) \right]^{\frac{5}{3}}$$

(ii) Critical Water Level

The critical water level appears at the overflow point of the groundsill and weir of overflow type. At this point, the flow changes from subciritcal to supercritical.

The critical water level is calculated as follows:

$$h_c = \sqrt[3]{\frac{Q^2}{gB^2}}$$

where h_c : critical depth (m)

g : gravitational acceleration (m/s^2)

Q : discharge (m^3/s)

B : width of the river

(6) Computer Program

A computer and a program are necessary to conduct faster non-uniform flow computation. There are several programs for non-uniform flow computations; one of with is the HEC-RAS, which is developed by the Hydrologic Engineering Center (HEC) of the US Army Corps of Engineers. HEC-RAS is recommended to be used by the DPWH engineers, since this program is not only easy to use but also covers all of the necessary methods to analyze the channel flow and design the river channel.

6.4.5 Alignment

For deciding the alignment of river, the following points must be comprehensively examined:

(1) The existing river width, though it exceeds the width necessary for accommodating the design discharge, should be maintained or preserved as wide as possible; in anticipation of the retarding effect.

(2) During the event of floods, the direction of the river flows and positions of flow attack zones along the river should be analysed carefully in order to devise the suitable alignment for the floodwater flow with a little resistance as much as possible. Generally in most cases, rapid rivers are almost linear. Medium to small rivers shall avoid sharp bend, rather the alignments should be generally smooth. In large rivers, flow attack zones can be fixed in order to omit the revetments on the other side. In this case, most designs are worked out with mild bends for large rivers with meandering course.

(3) The position of new flow attack zone shall be decided in consideration with the present river course, topographic and geologic features in the hinterland, and conditions of land use. House-congested areas and the closing places of old rivers, etc. shall be avoided as practically as possible.

(4) At the point of sharp bend, it is necessary to offset the bend as well as the river width into a mild course so that flow velocity towards the flow attack zone could be decelerated or slackened.

(5) The bank alignment of the low flow channel in a compound cross section should be normally parallel to the alignment of the dike whenever it is linear or slightly curved. But in case its alignment is not parallel to those of the banks, as it is decided generally in consideration of the channel maintenance, low flow channel uses, i.e., navigational, irrigation purposes, etc. It is necessary to arrange/set the banks as far as possible from the dikes.

6.4.6 Longitudinal Profile

To increase the discharge capacity, the cross sectional area shall be improved through widening, excavation/dredging and/or dike construction. The design Flood Level (DFL) shall be determined and the required longitudinal profile and the cross section of the river channel are then determined.

(1) Design Flood Level

Design Flood Level (DFL) means the high water level that corresponds to the design discharge. Basically, the DFL shall be set approximately at the level of the ground elevation along the river (Please see **Figure 6.36**). When the DFL is set above the ground height, It should not be set above the experienced maximum flood level because it will induce problems on overflow flooding, on tributary confluence, and etc.

As much as possible, river should be planned as non-diked. When there is sufficient afflux of drainage from the hinterland to the river and overflow flooding occurs, the damage potential is minimal. On the other hand, for diked rivers, if the floodwaters continue to rise, it induces a large pressure against the dike and the damage potential is great once the dike is broken.

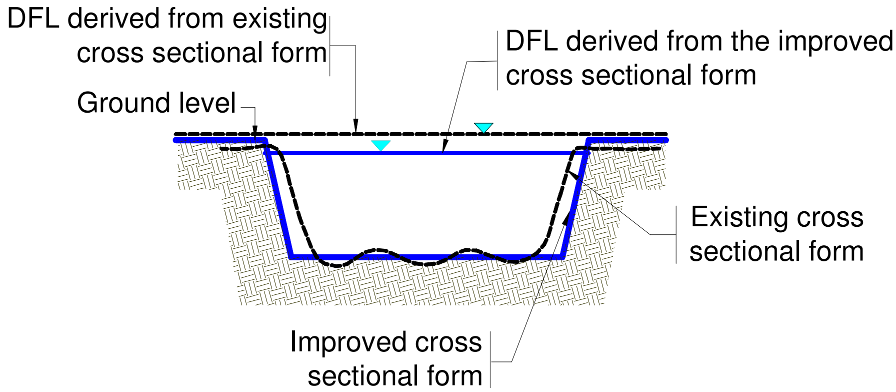


Figure 6.36 Design Flood Level (DFL)

(2) Design Flood Level of Tributary Affected by Backwater of Main River

The peak flood discharges of the main river and a tributary do not usually occur at the same time. In situations where the drainage basins are extremely different between the main river and the tributary, little relationship exists during peak flood occurrence; the backwater of the main river is surmised to be almost horizontal. In consideration to the relation between the catchments area of the main river and the tributary, if the two peak discharges seem to occur at same time, the backwater effect should be determined by uniform flow calculation. (**Refer 6.4.4 (5) Starting Water Level (b) Water Level at Junction with Main Channel**)

(2) Longitudinal Riverbed Profile

The longitudinal riverbed profile shall be determined without any revision/change of the existing longitudinal riverbed slope. The riverbed slope should be set according to the average elevation of the existing riverbed and not on its centerline (**Figures 6.37** and **6.38**).

This is the safest method in setting up the said river profile, because whatever riverbed modification has been introduced through dredging/deepening, it will return to its original profile as explained in the river segment. The deepest riverbed should be indicated in the longitudinal profile because this will be the one of the important parameters in deciding the design foundation depth of revetment.

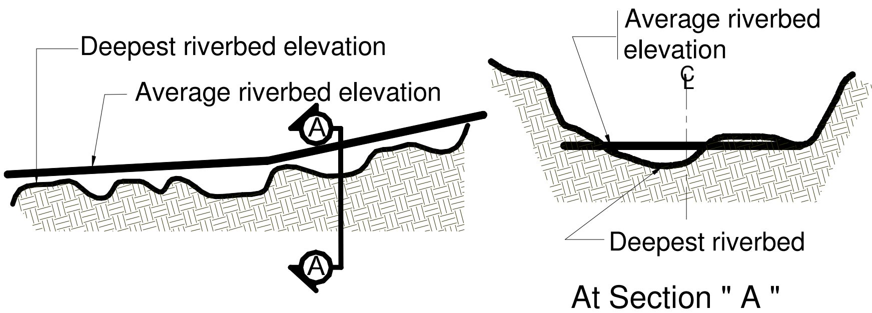


Figure 6.37 Longitudinal Profile

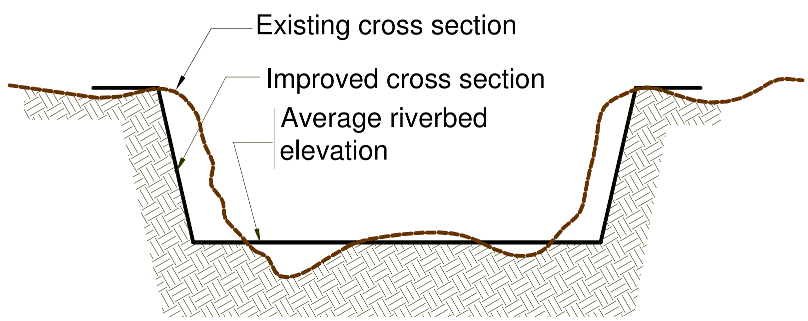


Figure 6.38 Cross Section Form

6.4.7 Cross-sectional Plan

In a large river, when the ratio of the design flood level to the ordinary water level is large, the design cross-section form (of a waterway) should be a compound cross section, if possible. However, it is costly to maintain the low water channel because it is normally planned with revetment to maintain the waterway. The purpose of setting a low water channel is to secure and/or fix the waterway stability to prevent meandering, to protect the bank and to maintain a navigable waterway.

In a small river, rapid flow usually occurs resulting to several changes in watercourse. Under such situation, it is often difficult to clearly set low water channel and to maintain it, thus, the single cross section is normally adopted.

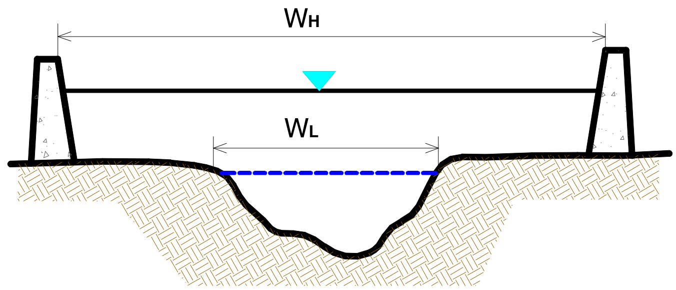


Figure 6.39 Compound cross section

Note: W_L – Width of low water channel
W_H – Width of high water channel

(1) River Width

The river width shall be decided according to the design flood discharge, considering the gradient, topographic features of the river, and the situations of land use from the upstream to the downstream of the river, etc. Even if the discharge is the same, the differences in water depth, gradient and bed roughness change the required and/or desired river width from the viewpoint of river capacity. Furthermore, it depends upon the conditions of housing congestion along the river, situations of land acquisition, and so on.

In making a river improvement plan, values in **Table 6.6**: is recommended.

Table 6.6 Recommended River Width

Design Flood Discharge (m3/s)	River Width (M)
300	40 – 60
500	60 – 80
1,000	90 – 120
2,000	160 – 220
5,000	350 – 450

(2) Low and High Water Channels

The height of a high water channel is to be discussed together with the width of a low water channel, as it is not preferable to have an excessively high velocity on the high water channel from the maintenance viewpoint, i.e., to secure the stability of high water channel on the occasion of a flood.

The width of a low water channel is generally decided with emphasis on the present situation. While the height of a high water channel is designed by calculating the flow capacity for the frequency of one to three floods on high water channel per year, depending on the demand for utilization of high-water channel.

(3) Freeboard

Freeboard is a margin of the height for the dike that does not allow overflow against the design flood level. In general, the dike is made of earth and sand and is very weak to overflow. Therefore, the dike is provided with adequate freeboard in preparation for the temporary rises of the water level caused by wind and waves on the occasion of a flood, swell and hydraulic jump, etc. The freeboard of a dike shall not be less than the value given in **Table 6.7**, according to the design flood discharge.

Table 6.7 Minimum Required Freeboard for Dike

Design flood discharge (m^3/s)	Freeboard (m)
Less than 200	0.6
200 and up to 500	0.8
500 and up to 2,000	1.0
2,000 and up to 5,000	1.2
5,000 and up to 10,000	1.5
10,000 and over	2.0

When the ground height in the inland adjacent to the channel is higher than the design flood level and the freeboard is designed as described in **Table 6.7**, there is a possibility that inland water is difficult to be drained (**Figure 6.39**) and further flood water exceeding the design discharge can flow downstream to the channel with dike, resulting in overflowing and causing serious flood damage (**Figure 6.40**). Therefore, the freeboard of the channel, the design flood level of which is lower than the ground height, can be set less than values listed in **Table 6.7** to avoid adverse effects mentioned above.

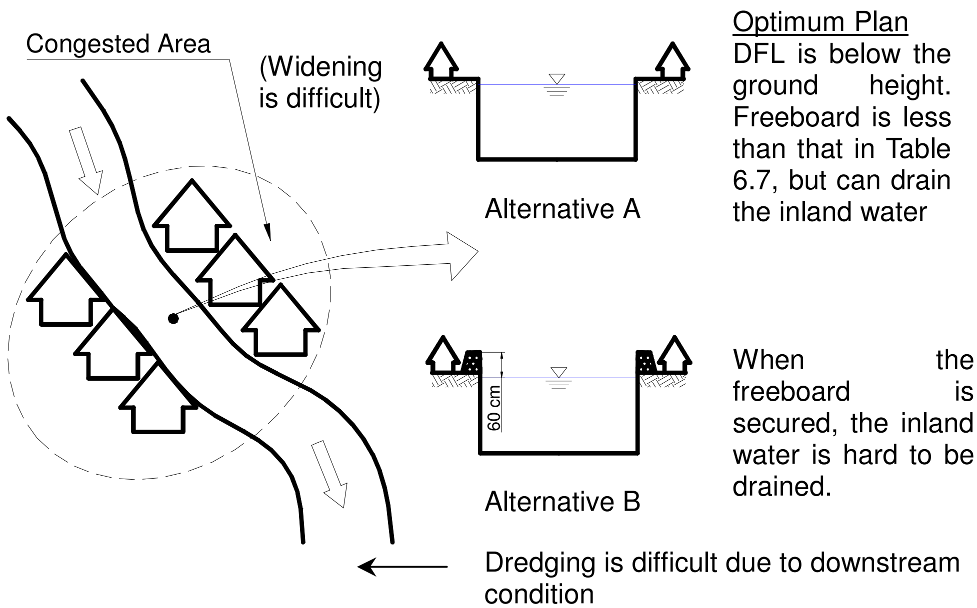


Figure 6.40 Freeboard of Non-diked Channel (1)

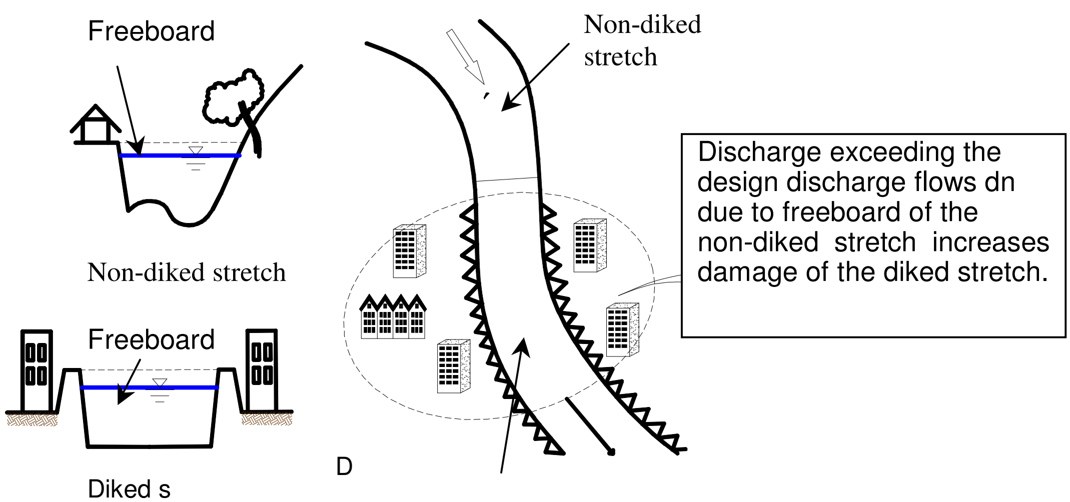


Figure 6.41 Freeboard of Non-diked Channel (2)

(4) Cross Section Form at Curve

At a curve of waterway, a drift current occurs during floods, and the water level at the concave side of the curve rises to cause high velocity locally, threatening to make the waterway unstable. Considering that dead water zone is caused inside the curve, and that the effective cross-section area of the river is decreased due to eddy, river width at said portion shall be designed about 10% to 20% wider. And at the outer bend side, if scouring and erosion occurred frequently, cut off channel should be considered.

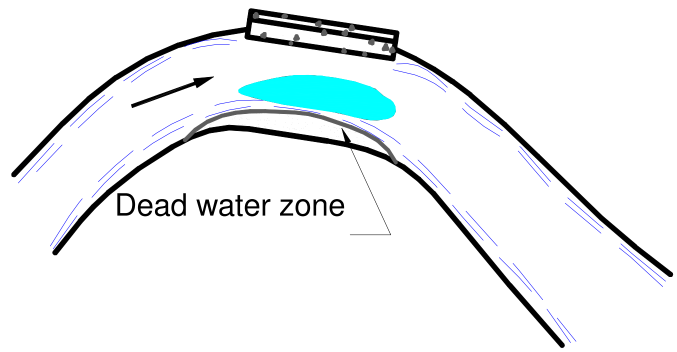


Figure 6.42 River Curve (Example)

GLOSSARY OF TERMS

AGGRADATION - Progressive rising of the riverbed elevation as a result of sediment accumulation. Rapid aggradation of the riverbed usually occurs during volcanic eruption (like the case of Mt. Pinatubo eruption) where the lahar and debris flow are deposited in the mountainside and are carried by water in abundant quantities during typhoon/floods.

ALLUVIAL - Soil or earth material which has been deposited by running water.

ALLUVIAL FAN - A fan shaped deposit formed where a stream emerges from an entrenched valley into a plain or flat (sometimes referred to as ' gravel wash'). Outspread mass of alluvium deposited by flowing water where it debouches from a steep, narrow canyon onto a plain or valley floor. The abrupt change of gradient eventually reduces the transport of sediment by the issuing stream.

AREA, CATCHMENT - Area from which rainfall flows into a drainage line, reservoir, etc. The area from which a lake, stream or waterway receives surface water which originates as precipitation. Also called "***DRAINAGE BASIN***, ***RIVER BASIN***". The drainage area of a basin is the plane area enclosed within its divide; that is the area of its horizontal projection.

AVERAGE RIVERBED - The average riverbed profile in a cross section survey which is set in order to establish the elevation of the longitudinal profile and to compute the area of river cross section.

BACKWATER - The rise of water level that occurs immediately upstream from a structure (dam) or obstructions in a river to a considerable distance brought about by the presence of structure. Backwater effect in a river is also caused by tidal influence.

BAR - An alluvial deposit at the mouth of a stream or at any point in the stream itself which causes an obstruction of flow and to navigation, in the case of a bay or inlet.

BASIN, RIVER - A term used to designate the area drained by the main river and its tributaries. SEE ***AREA, CATCHMENT.***

BENCHMARK - A permanent point or monument, whose elevation above a given datum is known, and which is used as a point of reference in the determination of other elevations.

CHANNEL, OPEN - Any conduit in which water flows with a free surface. Channel in which the stream is not completely enclosed by solid boundaries and therefore has a free surface subjected only to atmospheric pressure.

CROSS SECTION, COMPOUND - A river cross sectional form which is composed of low-water and high water channels. This section is usually applied for large rivers wherein the ratio of the design flood level with the normal water level is relatively large.

CUT-OFF CHANNEL - A channel connecting the beginning and the end of a meandering portion of a stream.

DEEPEST RIVERBED - The lowest elevation in a river section which should be taken into account during cross-sectional survey as basis for deciding foundation depth of flood control structure - particularly revetment.

DEGRADATION - Progressive lowering of riverbed elevation at the downstream caused by the insufficient supply of sediment from the upstream. Rapid degradation in the downstream usually occurs when a structure (like dam or weir) is constructed upstream due to the sudden cut of sediment supply.

DELTA - A relatively wide area with a very gentle ground slope towards the river so that its profile is almost parallel to the river stage. Once overflow to the area occurs, it finds hard to drain into the river.

DEPTH OF SCOUR - The depth of materials removed below the set datum.

DESIGN DISCHARGE - The calculated discharge based on the frequency of a return period.

DESIGN FLOOD LEVEL - The design floodwater elevation of a river to which the flood will rise in relation to the design flood frequency used (e.g., 1-year, 2-years, 5-years return period, etc) in computing the design discharge.

DIKE - An embankment, sometimes called levee, constructed parallel to the banks of a stream, river, lake or other body of water for the purpose of protecting the landside from inundation by flood water, or to confine the stream flow to its regular channel.

DITCH - An artificial open channel or waterway usually constructed parallel to the dike to drain the overflow or seepage water from the river.

EDDY - A whirling and/or circular motion of water that usually occurs in an irregular cross section of a waterway, like on outer bends.

FLOOD PLAIN - Flat land bordering a river. A habitually flood-prone area.

FLOW ATTACK ZONE - See ***CONCAVE BEND***

FREEBOARD - Allowance in height (of a revetment/levee) to arrest overtopping of water due to wave action.

GROUNDSILL – A flood control structure, usually built downstream of the bridge in order to fix the riverbed and prevent further degradation. Groundsill is classified into two (2) types, the head type and the non- head type.

INNER BEND – - A curvature and/or a meander stretch of a river wherein low velocity or sometimes no flow is observed. This is the part of the river where sediment accumulation is formed. Also known as dead water zone.

LEVEL, MAXIMUM FLOOD - The highest recorded flood level.

MASTER PLAN – The overall description of the project area. Sometimes this is referred to as a basin-wide comprehensive study of a river system. Master plan explains the flood control policy, strategy, target flood magnitude, main works, etc. of a river system.

MEAN MONTHLY-HIGHEST WATER LEVEL (MHW) - The average of the monthly-highest water level, where the monthly-highest water level for a particular month is defined as the highest water level occurring in the period of two (2) days before the day or the lunar syzygy to four (4) days after the day of the lunar syzygy.

NARROW PLANE – A plane composed of sand and other fine materials which is formed and conveyed by the overflowing of flood water from the river running between two mountains. Width of narrow plane ranges from 50 to 200 meters.

ORDINARY WATER LEVEL - Refers to the average water elevation of a river during rainy season. Average water level elevation is established/derived from the measurement of water level elevations during the months of July to November.

OUTER BEND - A curvature and/or a meander stretch of a river wherein high velocity usually occurs resulting to heavy scouring and forming a drift stream. Also known as direct water attack or flow attack zone. (See attack zone)

REFERENCE POINT - In a river, the place or location of observation point where the planned discharge is observed and fixed, Term "Control point" is used in Yellow Book.

RETARDING BASIN - A natural or man-made reservoir designed and operated to reduce the peak volume of the flood flow of a stream or river through temporary storage.

RETURN PERIOD - The probability, expressed in years, where a phenomena (i.e., flood, rainfall) of a targeted size/magnitude will likely to occur.

REVETMENT - A flood control structure for protection of the riverbank from collapse brought about by erosion, scouring and riverbed degradation.

RIVER BANK - River bank is herein defined as the highest point and/or ground elevation of a river which can contain flood water without flooding the adjacent land areas.

RIVER, DIKED - A river where improvement (like dike) has been introduced.

RIVER, NON-DIKED – A natural river or an improved river where the Design Flood Level is lower than the ground elevation and no dike/river wall has been introduced.

RUN-OFF ANALYSIS - Calculation of discharge from rainfall analyzing the basin and river characteristics.

SAFETY LEVEL – is the design period applied planning and design of flood control projects where socio-economic, technical feasibility and environmental sustainability are attained

SCOUR - Lowering of streambed or undermining of foundations caused by the tractive force of flowing water.

SCOURING, LOCAL - Scouring concentrated on a specific part or location of the river. Local scouring occurs in areas like the pier of bridge.

SPUR DIKE - A flood control structure to reduce the flow velocity near the bank by directing the flow away from the bank and in order to protect the riverbank from collapse.

TIME OF CONCENTRATION - This refers to the period of time for the storm water or rain water to flow from the most distant point of the drainage area to the point under consideration. The sum of inlet time + flow time.

TRACTIVE FORCE – The velocity-resisting force or action of riverbed materials.

TRIBUTARY - A stream or other body of water, surface or under ground, which contributes its water, either continuously or intermittently, to another larger stream or body of water.

TRIBUTARY RIVER - A confluence river usually smaller that the main river

WATERSHED - The line which follows the ridges or summits forming the exterior boundary of a drainage basin, and which separates one drainage basin from another. Watershed is equivalent to Drainage Divide.

WATERWAY - General term denoting a river, stream and other similar tributary area.

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