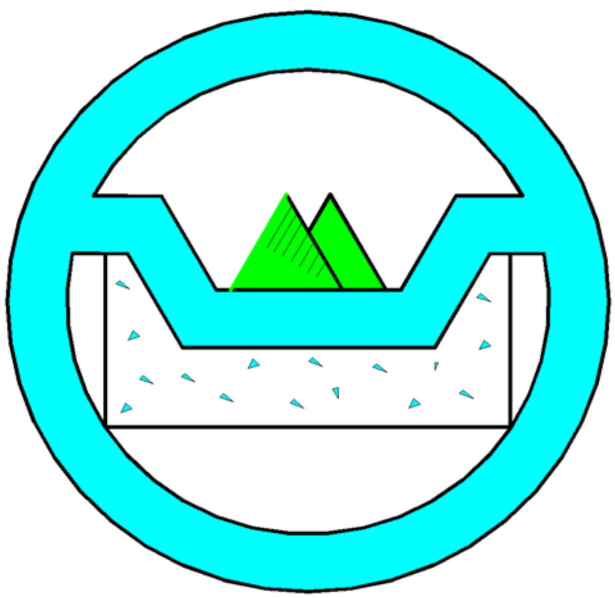
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS
JAPAN INTERNATIONAL COOPERATION AGENCY

TECHNICAL STANDARDS AND GUIDELINES FOR PLANNING OF FLOOD CONTROL STRUCTURES



June 2010

Project for the Strengthening of Flood Management Function of the DPWH

MESSAGE

Climate change is unabated with varied degrees of impacts in different regions. The Philippines comprising of 7,107 islands leads the list of nations most in danger of frequent and more intense storms, being in the typhoon path and close to storm breeding zone. On the average, 20 storm disturbances annually and monsoon bring heavy rainfall and floods leading to disasters.

For the last decades, rain-induced disasters are becoming more frequent because of unpredictable weather pattern, which claimed countless lives, losses and damages to properties, agriculture and infrastructure, which have adverse socio-economic effects. Realizing these phenomena, formulation of integrated adaptation and mitigation measures combining structural and non-structural is indispensable to reduce the vulnerability and increase the awareness and preparedness of the people at risk in the flood and disaster prone areas.

As the agency mandated in infrastructure development, our previous flood control projects were geared to address these disasters. Technical Standards and Guidelines and Manuals were formulated, published and issued nationwide in 2000-2005 under the JICA Grant Project for Enhancement of Capabilities of Engineers in the Field of Flood Control and Sabo Engineering. It was envisioned that the DPWH engineers would be able to plan, design and construct structures suited to local environment with the available hydrological and river information.

As principles were applied from these publications in the field, the need for improvements of the contents and substance became evident. Hence, the recently completed JICA Grant Project for Strengthening the Flood Management Function of DPWH aimed to fill the gap by modifying and adjusting the parameters, and incorporating relevant and timely topics in the revised Technical Standards and Guidelines for Planning of Flood Control Projects.

This new publication is intended to be used widely, not only for DPWH engineers but also for consultants and other practitioners. Hopefully, better quality infrastructures will come up with considerations to climate change adaptation, to preservation of the environment and with human interests in mind. I understand that this should be a continuous activity to update and include new trends and development in the planning of flood control projects.

I highly commend and appreciate the DPWH engineers from PMO-FCSEC, Bureau of Design, Planning Service, Bureau of Construction, PMO-Major Flood Control Projects Clusters I and II, and other offices and the JICA experts who spent valuable time and shared their expertise in the revision of this Technical Standards and Guidelines. I am very much grateful to JICA not only in infrastructure development but for the enhancement of the technical capabilities of the DPWH staff in the field of flood control and sabo engineering.

VICTOR A. DOMINGO
Acting Secretary

FOREWORD

The Japan International Cooperation Agency (JICA) has been providing assistance to the Philippine Government in mitigating water-induced disasters through its experts, development studies, technical cooperation projects and grant-aid projects.

In January 2000, DPWH established the Flood Control and Sabo Engineering Center (FCSEC) and the Project for Enhancement of Capabilities in Flood Control and Sabo Engineering of DPWH (ENCA) started under the JICA technical cooperation project as one of those assistances. In July 2005, the Project for Strengthening the Flood Management Function of DPWH (SFMF) succeeded ENCA to enhance further the ability of FCSEC engineers.

Through these two projects, several technical standards and guidelines on flood control and sabo engineering have been formulated.

This manual is developed as one of the achievements of ten year cooperation projects, which consisted of compiling and revising previous standards and guidelines on planning of flood control structures.

Transforming rainfall data into discharge using other related data such as topography, field survey, riverbed material and etc. is essential technique for flood control planning, and the discharge is the foundation for river improvement. Structures inconsistently constructed along a river without basing on proper planning are easy to break and consequently become ineffective, besides they sometimes give bad influence with each other. But in this country most of the flood control structures have been constructed without collecting the necessary data and conducting rainfall analysis. Truly effective water-induced disaster mitigation is based on exquisite planning.

I hope this manual together with other manuals listed below will be fully utilized by not only the DPWH officials but also other concerned entities, and will contribute to the formulation of truly effective planning, and then water-induced disasters will be mitigated in this country.

S. Nakamura

SHINYA NAKAMURA
JICA Chief Advisor

1. Specific discharge Curve, Rainfall Intensity Duration Curve, Isohyet of Probable 1-day Rainfall (March 2003)
2. Manual on Runoff Computation with HEC-HMS (November 2009)
3. Manual on Non-Uniform Computation with HEC-RAS (November 2009)

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The Technical Working Group in charged for the revision of Technical Standards, who met weekly, painstakingly reviewed, shared technical insights and valuable inputs.

Perfecto L. Zaplan, Jr. Chief, Hydraulics Division, BOD	Head
Jesse C. Felizardo Engineer IV, PMO-FCSEC	Vice Head
Tirso R. Perlada Engineer IV, Bureau of Construction	Member
Leonila R. Mercado Engineer IV, PMO-MFCP I	Member
Aquilina T. Decilos Engineer III, Planning Service	Member
Elmo F. Atillano Engineer III, Planning Service	Member
Gil I. Ituralde Engineer V, PMO-FCSEC	Member

Likewise, invaluable contribution and participation of Mr. Martiniano M. de la Cruz, Engineer III, of Bureau of Design and Ms. Rosemarie del Rosario, Supervising Environmental Management Specialist of Environmental and Social Services Office (ESSO) are deeply recognized. Ms. Isabel P. Cruz, Secretary for JICA River Management Advisor, served as a support staff.

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MARIA CATALINA E. CABRAL, PhD.
Assistant Secretary for Planning

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ACRONYMS

BC	Benefit Cost
DAO	Department of Environment and Natural Resources Administrative Order
DENR	Department of Environment and Natural Resources
DPWH	Department of Public Works and Highways
ECA	Environmental Critical Areas
ECC	Environmental Compliance Certificate
CNC	Certificate of Non-Compliance
EIA	Environmental Impact Assessment
EIARC	Environmental Impact Assessment Review Committee
ECP	Environmentally Critical Project
EIRR	Economic Internal Rate of Return
EMB	Environmental Management Bureau
EO	Executive Order
FBI	Filed Bored Investigation
FPIC	Free and Informed Consent
FS	Feasibility Studies
HB	House Bill
HEC-HMS	Hydrological Engineering Center – Hydrological Modeling System
HEC-RAS	Hydrological Engineering Center – River Analysis System
IEE	Initial Environmental Examination
IPRA	Indigenous Peoples' Right Act
LLDA	Laguna Lake Development Authority
LWUA	Local Waters and Utilities Authority
MMDA	Metro Manila Development Authority
MP	Master Plan
NAMRIA	National Mapping and Resource Information Authority
NCIP	National Commission on Indigenous People
NECA	Non-Environmentally Critical Areas
NECP	Non-Environmentally Critical Project PIP – Project Implementation Plan
NPV	Net Present Value
PAGASA	Philippine Atmospheric and Geophysical Services Administration
PAP	Project Affected People
PAF	Project Affected Families
PD	Presidential Decree
PDR	Project Description Report
PEPPRMP	Programmatic Environmental Performance Report and Management Plan
RAP	Resettlement Action Plan
RIDF	Rainfall Intensity Duration Frequency
ROW	Right of Way
SCS	Soil Conservation Service, presently Natural Resources Conservation Service
TSG	Technical Standards and Guidelines

UH	Unit Hydrograph

Chapter 1 GENERAL PROVISONS

1.1 Scope and Application

This Technical Standards and Guidelines aim to provide the DPWH Engineers (including concerned Local Government Units and Government Consultants) with the basic knowledge and essential tools that will help establish uniformity in the planning of flood control and projects.

1.2 Governing Laws, Codes and Department Orders

Flood control projects are indispensable in the socio-economic development and the protection of lives, infrastructures, agricultural, and other resources of the country. To promote flood control activities, laws, codes and department orders governing flood control projects and drainage projects were formulated and executed, which include the following.

- P.D. 1067. A decree instituting a Water Code, thereby revising and consolidating the laws governing the ownership, appropriation, utilization, exploitation, development conservation and protection of water resources. Water Code, DPWH may declare flood control areas and promulgate guidelines governing floodplain management.

- P.D. 296. Directing all persons, natural or juridical, to renounce possession and move out of portions of rivers, creeks, esteros, drainage channels and other similar waterways encroached upon by them and prescribing penalty for violation hereof.

- P.D. 78 of 1972 creation of The Philippine Atmospheric, Geophysical and Astronomical Services Administration (PAGASA).

- Letter of Instruction (LOI) No. 19 dated Oct. 2, 1972 directed then Secretary of Public Works and Communications, to remove all illegal construction including buildings on and along esteros and riverbanks, and to relocate, assist in the relocation and determine sites for informal settlers and other persons to be displaced

- Presidential Decree No. 772 of 1972, for penalizing informal settlers and other similar act.

- PD No. 296 in 1973, directed all persons, natural or juridical, to renounce possession and move out of portions of rivers, creeks, drainage channels and other similar waterways encroached upon and prescribed penalty for violation

- The Provincial Water Utilities Act by PD No. 198 of 1973, for declaration of a national policy of local water utilities and for creating the Local Water Utilities Administration (LWUA).

- P.D. 1149 of 1977 organized the National Flood Forecasting Office as one of the major organization units of the PAGASA. The present PAGASA is attached to the National Science and Technology Authority by Executive Order (EO) No. 128 in 1987.

- P.D. 1566 of June 11, 1978 establishment of a National Program on Community Disaster Preparedness. The National Disaster Coordination Council (NDCC) issued the Calamity and Disaster Preparedness Plan in 1988. Flood fighting is undertaken nationwide by virtue of this PD.

- Ministry Order No. 20, Series of 1982. Guidelines for the Preparation, Evaluation and Ranking of Flood Control and Drainage Projects.

- P.D. 187 as amended by P.D. 748 and Batas Pambansa Blg. 8, An act defining the Metric System and its Units, providing for its implementation and for other purposes; and MPWH Memorandum Circular No.6, dated January 6, 1983, re Metric System (SI) Tables. Under the Local Government Code, a city or municipality may reclassify agricultural lands and provide the manner of their utilization and disposition.

- Executive Order No. 192 of 1987 mandates Department of Environment and Natural Resources (DENR) for conservation, management, development and proper use of the country's environment and natural resources including those in the watershed.

- Republic Act No. 4850, creating the Laguna Lake Development Authority (LLDA).

- Republic Act No. 6234, creating the Metropolitan Waterworks and Sewerage System (MWSS).

- Executive Order No. 215 and 462, for private sector participation in hydrological endeavors.

- Republic Act No. 7924 of 1994, for creating the Metropolitan Manila Development Authority (MMDA), defining its powers and functions, providing funding therefore and for other purposes.

- RA 9003 - Solid Waste Management Act, overall institutional framework of managing solid wastes including functions and responsibilities

- IRR of RA 9003 Section 6 – Creation of Local Solid Waste Management Committee (Creation of Barangay Solid Waste Management Committee)

- PD No. 296 – directed all persons, natural or judicial to renounce possession and move out of portions of rivers, creeks drainage channels & other similar waterways encroached upon & prescribed penalty for violation

- Presidential Decree No. 825 – Providing Penalty for Improper Disposal of Garbage

- Presidential Decree No. 856 – Sanitation Code

Chapter 2 NECESSITY OF FLOOD CONTROL PLAN

The formulation of a Flood Control Plan for a target river basin is necessary, and must be undertaken in a “basin-wide” approach, considering the influence/effect of flood and the future related plans such as:

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

Some effects/influences of other development plans in the formulation of flood control plan may include examples, such as height of levee affecting the design height of bridge; the design riverbed profile affecting the design of irrigation intake/canal and other related facilities.

2.1 Classification of Flood Control Plan (Stage of Flood Control Plan)

Flood control plan is classified as:

1) Master Plan (MP)

2) Feasibility Study

3) Project Implementation Plan (PIP)

2.2 Design Flood Frequency (Safety Level)

Basically, all flood control projects must have a flood control plan. It is necessary to make a flood control plan based on the forecasted river phenomena which is likely to occur as a result of the discharges corresponding to the design flood frequencies.

Design Flood Frequency is expressed by return period, i.e., the probability (expressed in years) where a flood of a target size/magnitude is likely to occur. The return period shall 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.

2.3 Procedure for the Formulation of Flood Control Plan

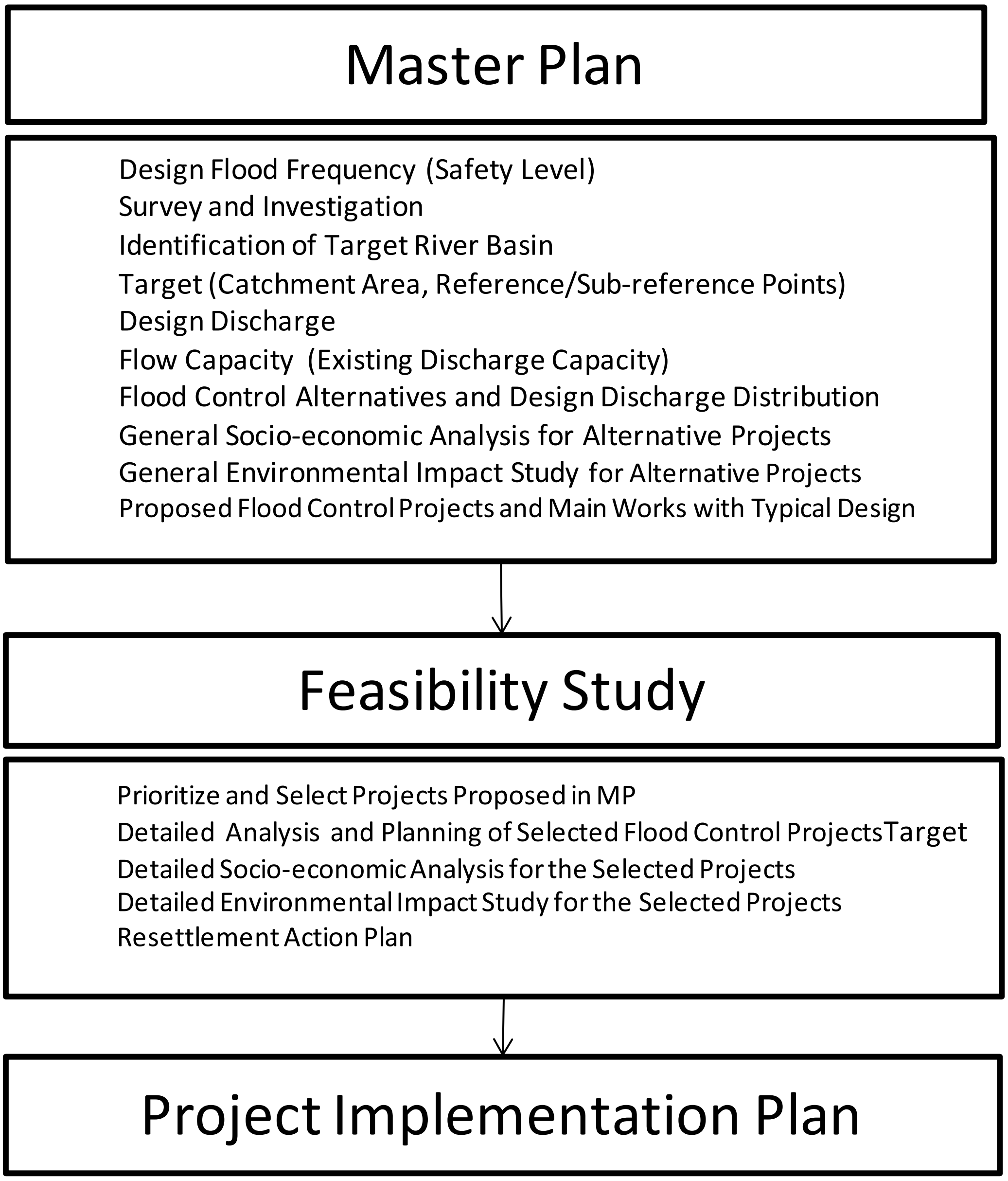


Figure 2.3: Formulation of Flood Control Plan Flow Chart

2.4 Flood Control Master Plan

The Master Plan incorporates flood control policy, strategy, target flood magnitude and main works, etc. in the river system. Extensive survey, investigation and analysis to formulate the flood control master plan are necessary.

Each identified project in the Master Plan shall be formulated for a long-term to have optimum benefits and in consideration of the effect in the implementation in other areas of the river basin.

Master Plan shall include the following:

1) Project area: The project area shall describe, among others the natural condition, topography and/or historical background (flooding history).

2) Safety level described by return period.

3) Flood control main objective: This takes into consideration which appropriate improvement has to be undertaken (i.e., widening the river, excavating the river mouth, embankment, etc). The structures shall be decided based on the entire river basin flood management.

4) Basin-wide rainfall-runoff model: A simulation model for the estimation of the probable flood discharge at all the control points is necessary to be developed.

5) Diagram of design discharge: A diagram at the control points to determine the critical areas affected by high water stages is necessary for improvement plan.

6) Main works: What are the main works to be undertaken (i.e., dike, dredging, etc.).

7) Survey works

    a) Longitudinal Profile

    b) Typical cross section of the river.

    c) Etc.

8) Typical structure design (i.e., embankment/revetment, etc.).

9) Location map of main works.

10) General Socio-Economic Analysis

11) General Environmental Impact Study

12) Proposed Flood Control Projects and Main Works with Typical Design

2.5 Feasibility Study

From the list of project proposed in the Master Plan, implementation for medium term are selected and prioritized based on the urgency to mitigate flood damages within the framework of socio-economic importance. The affected core areas should be given higher attention. Feasibility of each project is done in more detailed study. The study includes technical, economic, implementation schedule, operation and maintenance.

Prioritized and Selected Projects Proposed in MP

1) Detailed Analysis and Planning of Selected Flood Control Projects
2) Detailed Socio-economic Analysis
3) Detailed Environmental Impact Study
4) General operation and maintenance plan

2.6 Flood Control Project Implementation Plan

The Flood Control Project Implementation Plan specifies the works selected from the Feasibility Study including the funds and benefits to be derived from the project. Implementation period is usually 5 to 10 years. Economic analysis shall be conducted to determine the scope of the Project Implementation Plan (Calculation of Economic Internal Rate of Return (EIRR) is explained in Chapter 9)

Flood Control Project Implementation Plan shall include the following:

1) Channel plan (1:500 – 1:10,000)
2) *Validation of Survey and Investigation Works*
    a) *Cross section (Existing/Design)*
    b) *Longitudinal* profile (Existing/Design)
    c) *Etc.*
3) Structural design
4) Cost estimates
5) Benefit estimation
6) Environment/Social Management Plan
7) Detailed Operation and Maintenance Plan
8) Project Evaluation

Chapter 3 SURVEY AND INVESTIGATION

The primary purpose of survey and investigation is to provide basic data and information for flood control planning and design. Data collection, analysis and utilization are basically important for more appropriate plan/design, which relies greatly on the veracity and/or authenticity of the available basic data and information.

3.1 Master Plan Stage

3.1.1 Topographic Survey

To understand the general profile of a river system, catchment and flood prone areas, the following maps are required:

1) Topographic map with a scale of 1:50,000 or better
2) Land use map
3) Available flood hazard maps
4) Geologic map consisting of soil and rock formation from the surface, fault lines, liquefaction prone areas, etc. (optional).
5) Other related available map from the Local Government Units

Validate the maps in the field.

From the maps mentioned above, the activities include the following:

1) Delineate catchment area. (Refer to Section 4.1)
2) Classify the geological/geographical features of each sub-catchment area.
3) Classify the existing vegetation (forest, etc.) by each sub-catchment area.
4) Identify roughly the flood prone sites or low lying areas. (The area shall be identified and determined from the field investigation and water level analysis).
5) Identify the cities and municipalities in the flood prone area.
6) Identify the important public facilities such as coastal dikes, river dikes, national road, provincial road, bridges, city hall, church and school, etc. within the flood prone area.
7) Classify the land use in flood prone area, such as commercial area, residential area, industrial area, agricultural area, etc.
8) Identify the changes in the river course and longitudinal profile.
9) Classify the segment along the river.（see　Chapter 8.)

3.1.2 General Information

Collect past/present information on land use, population, economic activities, future development plans, etc. within the catchment and flood prone areas.

1) Population by city/municipality

2) Increasing of rations by city

3) Statistics of commercial activities per year by region and city

4) Statistics of industrial product per year by region and city

5) Statistics of agricultural products per year by region and city

6) Long term and medium term development plan by region, city and municipality.

3.1.3 Hydrological Data

Collect the following hydrologic data of the river basin:

1) Daily rainfall of all gauging stations within and around the catchment area throughout the recording period from PAGASA and other related agencies.

2) Hourly rainfall of all gauging stations within and around the catchment area during the duration of the flood.

3) Hyetographs of past typical floods on all synoptic rainfall gauging stations from PAGASA and other related agencies.

4) Maximum water levels during peak floods at all gauging station from BRS or other agencies and from interviews. (For rainfall and runoff analysis).

5) Discharge measurement record for all water level gauging stations.

6) H-Q (Height-Discharge relationship) rating curve for all water level gauging stations (with location, cross-section and flow velocity during flooding time).

3.1.4 Field Survey and Investigation

Steps in field survey are as follows:

1) River cross sections at typical sites.

    - 500 to 1000 m interval is used for stretches for initial determination of discharge capacity

2) Longitudinal profile.

    - Profile of the river derived roughly from topographic map

    - Longitudinal profile taken from cross section survey

3) Identification of the riverbed material

- By segment features of the river

Investigation and interviews will give the following information.

1) Records of past floods. (Frequency, area, depth, duration of flooding)

2) Conditions of the existing river facilities.

3) History of flood control activities in the basin.

4) Determine man made activities

3.1.5 Riverbed Material Investigation

Riverbed material is an important element to have an overview of the river characteristics. Investigation of the riverbed material consists of grain size analysis and measurement of specific gravity. The results shall be arranged in accordance with the required recommended sample quantities taken as follows:

1) River width less than 50 m: minimum of 1 sample /site/2 km

2) River width = 50m or more: minimum of 3 sample/site/5 km

(Center, left and right banks)

Single sampling at a site, the following are considered:

- During flood receding stage, fine materials or sediments are usually found predominantly on the riverbed surface. Riverbed surface materials after the flood are finer than flood time; therefore, it is appropriate to extract the sample after removal of the surface material.

- To extract the riverbed materials during the flood, select and determine the most appropriate sampling site.

3.2 Feasibility Study Stage

River length of channel is assumed about 200 m to 10 km.

3.2.1 Topographic Survey

Considerations:

1) Map with a scale of 1:500 to 1:10,000 or better. (Depending on the size of the river)

2) All the river improvement stretch should be covered.

3) The width of survey area shall be extended at least 50 m beyond both banks (The extension is necessary to determine the ground elevation of the main flood prone area.)

3.2.2 Cross Section Survey

River width is assumed 50 m to 500 m and the depth is about less than 20 m.

Considerations:

1) Section with a horizontal scale of 1:500 to 1:2,000 (depending on the size of the river)

2) Section with a vertical scale of 1:100 to 1:500 (depending on the topographic condition)

3) Interval of cross section survey ranges from 20 m to 200 m stationing.

4) Interval of point measurement for river cross-section ranges from 2 m to 5 m on narrow rivers and 5 m to 20 m on wide rivers.

5) The survey shall be extended at least 20 m beyond both banks (This shall be further extended to determine the ground elevation of main flood prone area.)

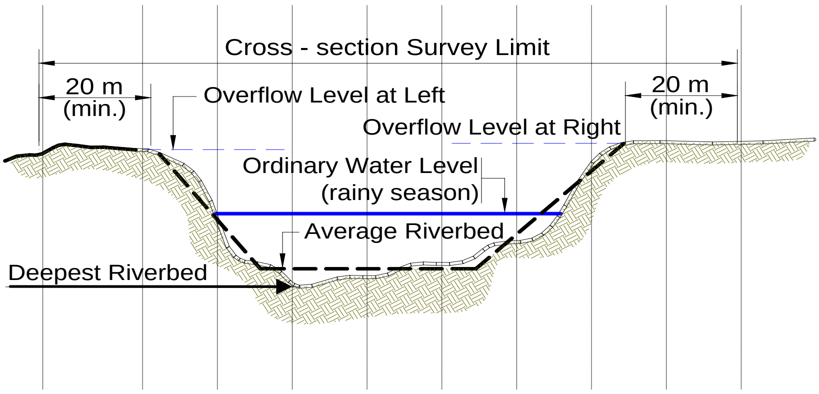


Figure 3.2.2: Cross section Survey (Project Implementation Plan)

Other considerations:

1) The overflow level of both banks shall be identified and indicated on cross section profile.

2) The water level at the time of survey (if any), shall be indicated in the survey .

3) Water level gauging stations shall be identified and indicated.

4) The ordinary water level (OWL) during the rainy season shall be indicated in the cross-section. (This water level is identified based on the interview of the residents in the absence of installed water level gauge.)

5) The deepest and the average riverbed shall be identified and indicated in the cross-section.

6) The information of land use adjacent to the banks shall be identified and indicated.

7) All elevations shall be reckoned from an established benchmark.

3.2.3 Longitudinal Profile Survey

The average riverbed profile/gradient shall be plot in longitudinal profile, which is used in Chapter 5: "Design Discharge". The stationing of cross section is indicated in the longitudinal profile including existing structure in the river (e.g.; bridge foundation/pier, groundsill, etc.), if any.

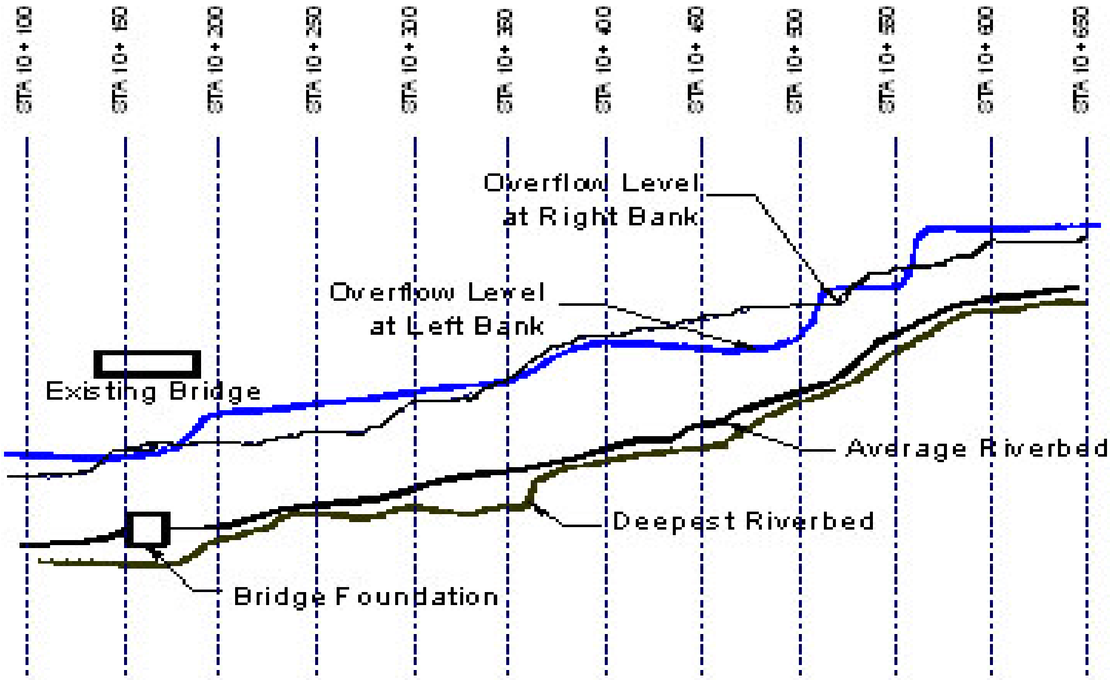


Figure 3.2.3: Longitudinal Profile Survey

3.3 Implementation Stage

River stretch is assumed 50 m to 500 m.

3.3.1 Topographic Survey

Considerations:

1) Map with a scale of 1:100 to 1:500 (Depending on the size of the river)

2) All structural design area should be covered.

3) The width of survey area shall be extended at least 20 m beyond both banks (This shall be increased when it is necessary to determine the ground elevation of main flood prone area.)

3.3.2 Cross Section Survey

River width is assumed 50 m to 500 m while the river depth is less than 20 m.

Considerations:

1) Section with a horizontal scale of 1:100 to 1:500 (depending on the size of the river).

2) Section with a vertical scale of 1:10 to 1:100 (depending on the topographic conditions).

3) Interval of cross sections shall be 20 m for straight and uniform river reaches, 10 m at minor river bends and 5 to 10 m at sharp bends.

4) The width of survey area is at least 20 m beyond both banks.

   - Sufficient space should be surveyed for planned structure. (The relationship between the planned structure and the ground level behind the structure should be indicated.)

5) Interval of measurement ranges from 2 m to 5 m on narrow rivers and 5 m to 20 m on wide rivers.

6) When a structure is proposed for construction on one side of a wide river, cross section survey may be on only one side covering the deepest riverbed in the survey area. For narrow river , both banks are recommended..

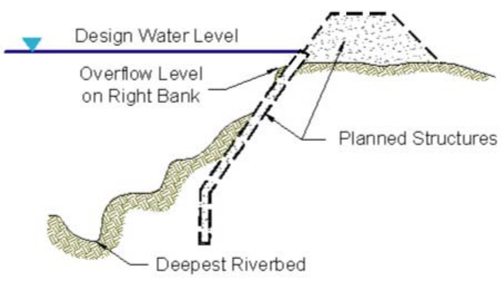


Figure 3.3.2: Cross section Survey (Structure Design)

3.3.3 Soil Investigation

The stability and performance of a structure such as weir, gate, or dam, etc. founded on soil depend on the subsoil conditions, ground surface features, type of construction, and sometimes the meteorological changes. Subsoil conditions can be explored by drilling and sampling, seismic surveying, excavation of test pits, and by the study of existing data.

Some geological concerns are as follows:

- The bearing capacity of foundation materials (bedrock or stone)
- The settlement characteristics of deposits
- The expansion potential of shale / clay
- The orientation of the rock layers (dip direction)
- The excavation difficulty
- The drilling problem

If site condition indicates presence of weak or permeable foundation or in the absence of any available soil profile, soil investigation is required. Depending on the proposed structure types and the site conditions, the following investigation is recommended.

1) Recommended List

Table **3.3.3: Recommended Soil Investigation**

Item	Application	
	Dike: Weak foundation Revetment: (No available information) Spur Dike: (No available information) Groundsill: all conditions	Dike: Permeable foundation
Machine Boring		
- Site	1 site (Representative location)	2 sites (slope ends at river and land sides)
- Depth	Until depth affected by construction of dike. (refer to DPWH guidelines)	Until continuous impermeable layer or 20 m
- Sampling	1 sample/1 m	1 sample/1 m
-Standard Penetration Test (SPT)	1 test/1 m	1 test/1 m
On-site permeable test		
- Site	-	2 sites (slope ends at river and land sides)
- Test	-	1 test/layer
Laboratory test (per 1 sample)		
- Test item (clay)	Direct Shear Stress (Grading, moisture content, density, consistency), unconfined compression or tri-axial shear, consolidation,.	Grading, moisture content, density, consistency, consolidation, tri-axial shear.
- Test item (Loose sand)	Grading, moisture content, density.	Grading, moisture content, density.

3.3.4 Riverbed Material Survey

The type of riverbank materials and the water surface area shall be surveyed and indicated in the topographic map and cross section profiles to:

- Determine the riverbed characteristics (Manning's "n")
- Determine the quality of riverbed materials (if suitable for construction use).
- Determine the relationship of the diameter of riverbed materials, riverbed gradient, etc. with the velocity of flow.
- Classify the river segment based on river morphology. (Refer section 8.2.1)

Representative Grain Size Diameter of Riverbed Material

The riverbed material is an important factor in understanding the river characteristics, which is expressed in its mobility and roughness.

This manual focuses on the grain size diameter of riverbed material consisting of grains with various sizes, as shown in Figure 8.4b. The grain size diameter smaller than 60%, is represented by (D_{60}).

1) By sieve analysis (for grain diameter below 10 cm)

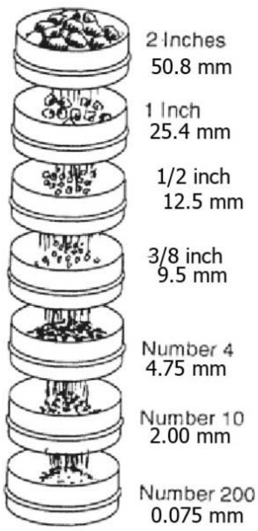


Figure 3.3.4a: Sieve Analysis Arrangement

Sieving is by size, with the smallest being on the bottom. Sieve pan is designated by the number with openings per mm. Dimensioned sieves indicate the actual size of the opening. For example, the Number 4 standard sieve has four openings per lineal inch (or 16 openings per square inch), whereas the 1/4-inch sieve has a sieve opening of 1/4 inch. The practical lower limit for the use of sieves is the Number 200 sieve, with 0.074-millimeter-square openings.

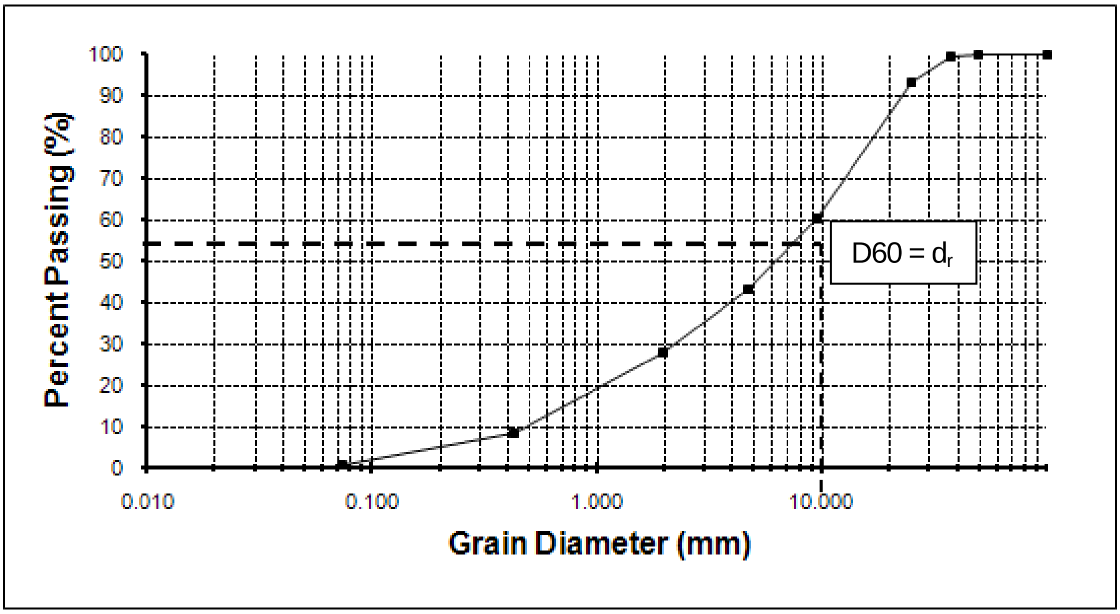


Figure 3.3.4b: Grain-Size Distribution Curve

The cumulative percent distribution curve represents the cumulative weight percent by particle size of the sample. Essentially, for each grain size, the curve shows how much of the sample was finer or coarser. The cumulative weight percent passing, which is the fraction finer than each subsequent grain size is the y axis, and the grain size is the x axis. The riverbed materials shown in the graph represent grain size diameter (**d_r**) equal to 10.0 mm.

The Procedure to chart Grain Size Distribution is explained below.

a) The sample must be weighed on the balance and the mass of the sample recorded.

b) Arrange the sieves such that the screen with the smallest opening is at the base and the largest is at the top. The number of the sieves refers to different types of size scales. Pour the sample onto the top screen and put the cover on the top screen.

c) For five minutes, shake in circular motion the screen and occasionally rap gently the bench top.

d) Remove the first screen carefully from the stack. Dump its contents on a receptacle.

e) Weigh the sample and the receptacle and record the results on a form.

f) Repeat (d) and (e) for each screen and the pan.

g) Sum up all the weights from each screen and the pan. Take note of the difference between the total weight and the original weight taken in step 1 to get the measurement error.

h) Using the excel program, plot the grain size (**x-axis**) versus cumulative percent (**y-axis**). The y-axis will have a scale of 0 to 100% using a linear scale (uniform spacing) while the x-axis will be in logarithmic scale.

2) By measurement at the site (for grain size diameter 1cm & above)

The following two (2) methods are used in determining the representative riverbed material grain size diameter at the site:

a) One Dimensional Sampling Method

This method is used when maximum grain size diameter of riverbed material is more than 20cm.

i Find the best sampling spot in the river where representative sample of riverbed material is exposed.

ii Within the sampling spot, find the biggest riverbed material and approximately determine its size. Measure 20 evenly spaced sampling point on the ground using a steel tape with interval the same as that of the biggest riverbed material. If the maximum riverbed diameter is 50cm, then the sampling interval should also be 50cm as shown below:

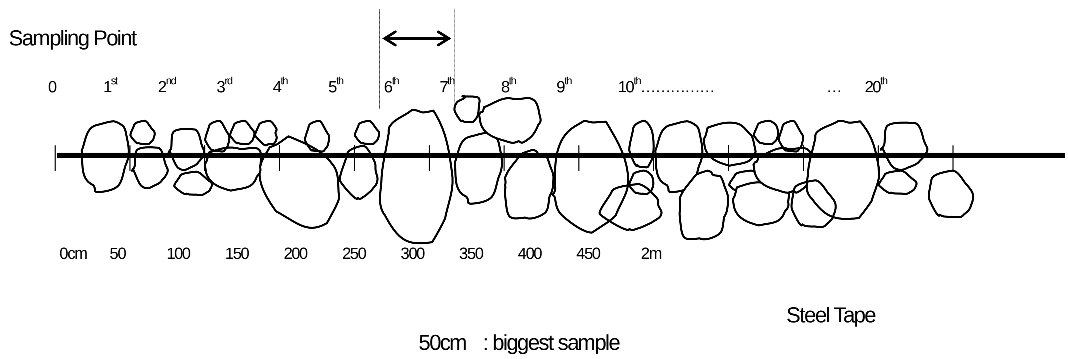


Figure 3.3.4c: One Dimensional Sampling Method

iii Pick the stones beneath the sampling interval point and arrange it in a straight line, from smallest to biggest. Select the 12th smallest sample from the arrangement. This is the equivalent 60% of the riverbed material samples and the corresponding representative riverbed diameter (d_r).

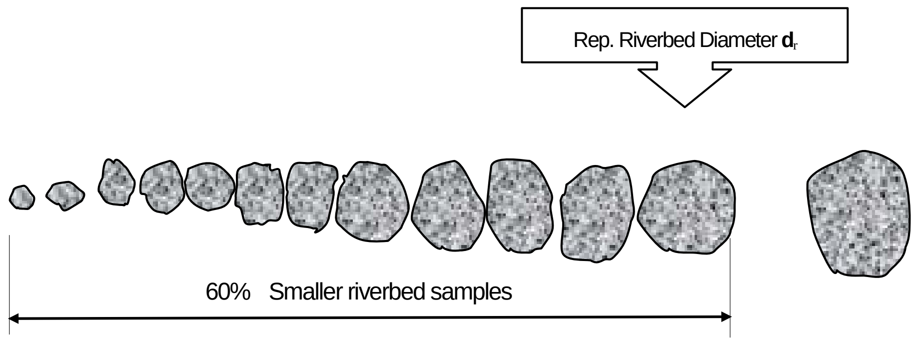


Figure 3.3.4d: Representative Grain Size of Sample

iv This sample can be measured using a ruler, and d_r can be computed using the formula:

$$d_r = (X_1 \cdot Y_1 \cdot Z_1)^{\frac{1}{3}} \qquad \text{Equation 3.3.4a}$$

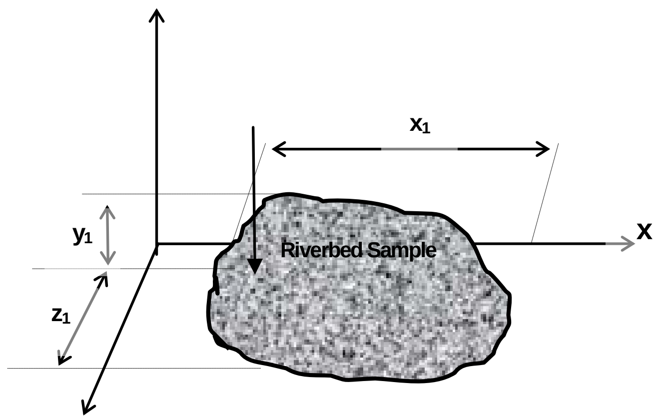


Figure 3.3.4e: Measurement of the Sample

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

$$P(d_i) = \frac{d_1^3 + d_2^3 + \cdots + d_i^3}{d_1^3 + d_2^3 + \cdots + d_{20}^3} \cdot 100 \qquad \text{Equation 3.3.4b}$$

b) Two-Dimensional Sampling Method

This procedure is applicable to riverbed materials with less than 20cm maximum grain size diameter. An improvised screen with equally spaced string on a 1 m square wooden frame is used for sampling.

i. Find the best sampling spot in the river where representative sample of riverbed material is exposed.

ii. Get a sample riverbed material and approximately determine its size.

iii. Within the sampling spot, find the biggest riverbed material and approximately determine its size. When maximum riverbed diameter D < 10 cm, use a 1.0m x 1.0m improvised screen with openings evenly spaced at 10cm both ways. When maximum riverbed diameter D > 10 cm, use a 1.0m x 1.0m improvised screen with openings uniformly spaced at 20cm both ways at the middle.

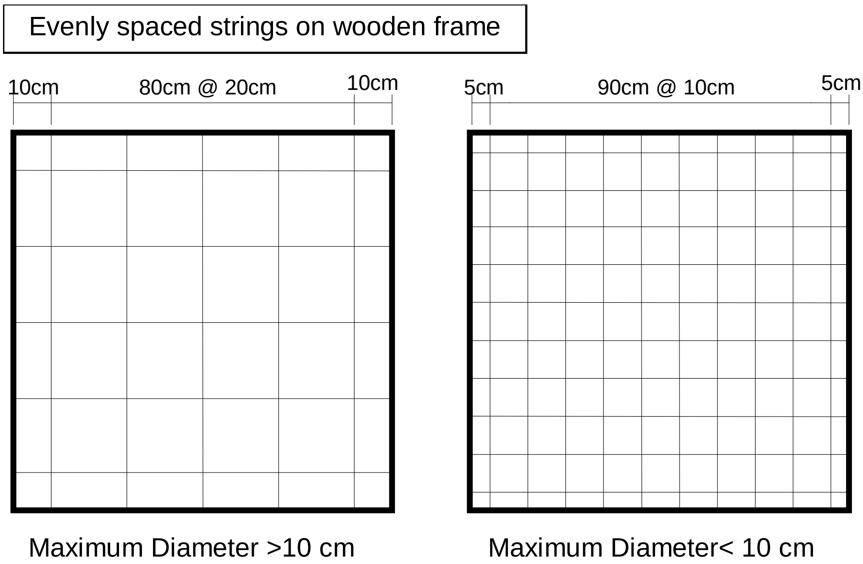


Figure 3.3.4f: Improvised Screen for Two-Dimensional Sampling Method

iv Lay the improvised screen on the exposed ground making sure that representative riverbed materials are contained within the $1m^2$ area.

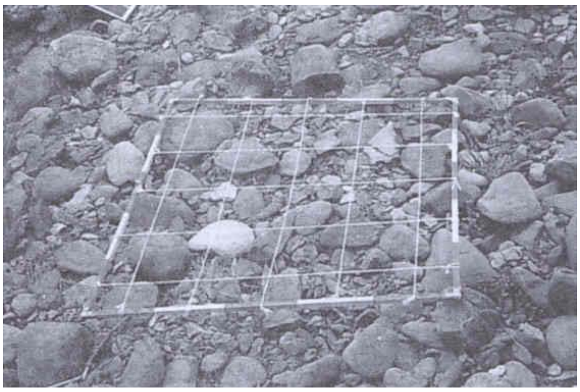


Photo 3.3.4: Two-Dimensional Sampling Method

v Pick gravels just beneath of each intersection of strings of the improvised screen and arrange it in a straight line, from smallest to biggest. Select the

60 % smallest sample from the arrangement. Say, the 15th sample in the 20 cm spacing strings (within 5 x 5 = 25 samples) or the 60th sample in 10 cm spacing strings (within 10 x 10 = 100 samples). Refer to Figure 3.3.4g

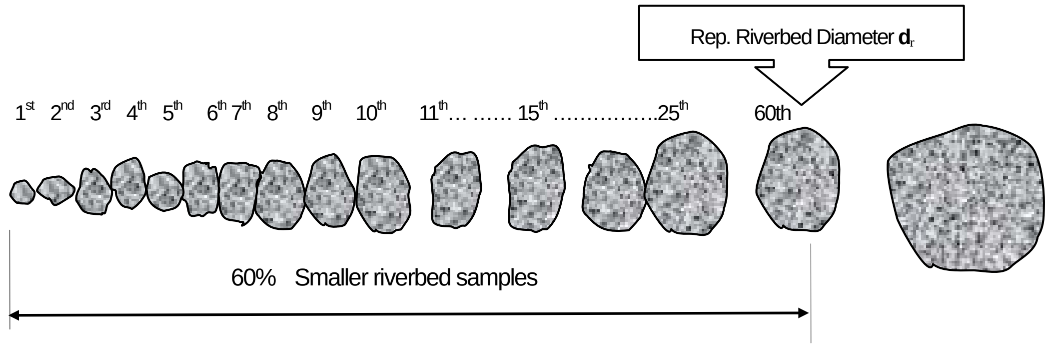


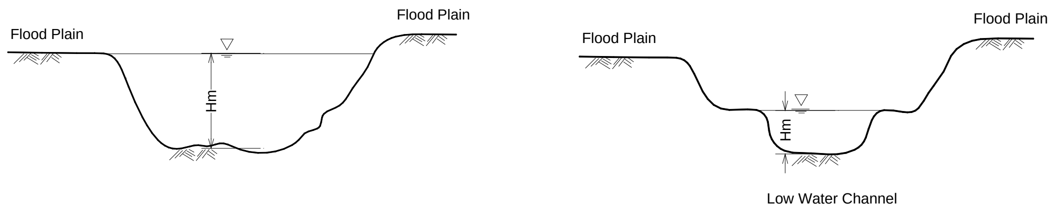
Figure 3.3.4g: Representative Grain of Sample

vi Measure the dimensions of the selected grain and calculate the representative grain diameter of the site. Calculation procedure is same as the One-Dimensional Sampling Method.

3.3.5 Mean Annual Maximum Discharge and Riverbed Slope

1) Mean Annual Maximum Discharge (Q_m)

The bank full discharge forming the low-water channel of the river corresponds to the mean annual maximum flood, which is almost equivalent to 2-year or 3-year return period. The bankfull discharge is estimated by uniform flow or non-uniform flow computation method as explained in Chapter 6. The obtained bankfull discharge shall be compared with the mean annual maximum discharge and/or the probable discharge.



a) Simple Channel b) Compound Channel

Figure 3.3.5a: Bankfull Discharge in Simple and Compound Channels

2) Riverbed Slope (S_b)

Riverbed slope is estimated from the average riverbed elevation.

Riverbed slope is the main factor for determining the river characteristics and for estimating the discharge capacity of the river. The gradient of riverbed at the site can be estimated by using any of the following two (2) methods.

- Use of longitudinal profile

- Use of NAMRIA Map (1:50 000)

The detailed explanation of the above two (2) methods is described below:

a) Estimation of gradient of riverbed by the use of longitudinal profile

Topographic survey data, especially cross-sectional survey results, are the basis of this method. The procedures are as follows:

i. Identify the river element at respective cross-sections

The plotted cross-section illustrates the features of the river. The right and left bank shoulders, shape, etc. of the river cross-section should be established first.

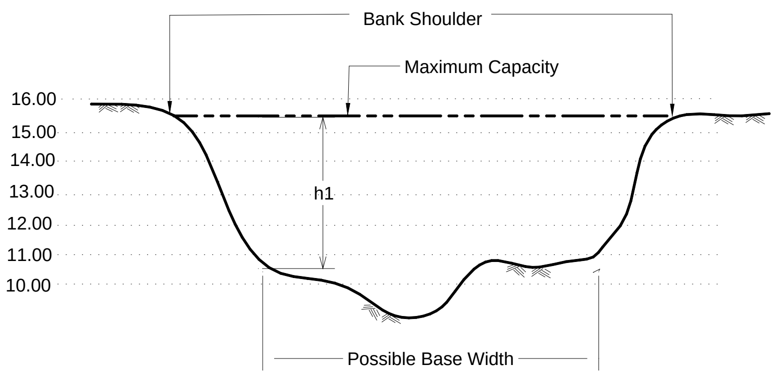


Figure 3.3.5b: River Elements

ii Estimate the average riverbed elevation at respective cross-sections

From the plotted cross-section, the average riverbed can be determined. There are many methods used in determining the average riverbed elevation. The following method is popularly used.

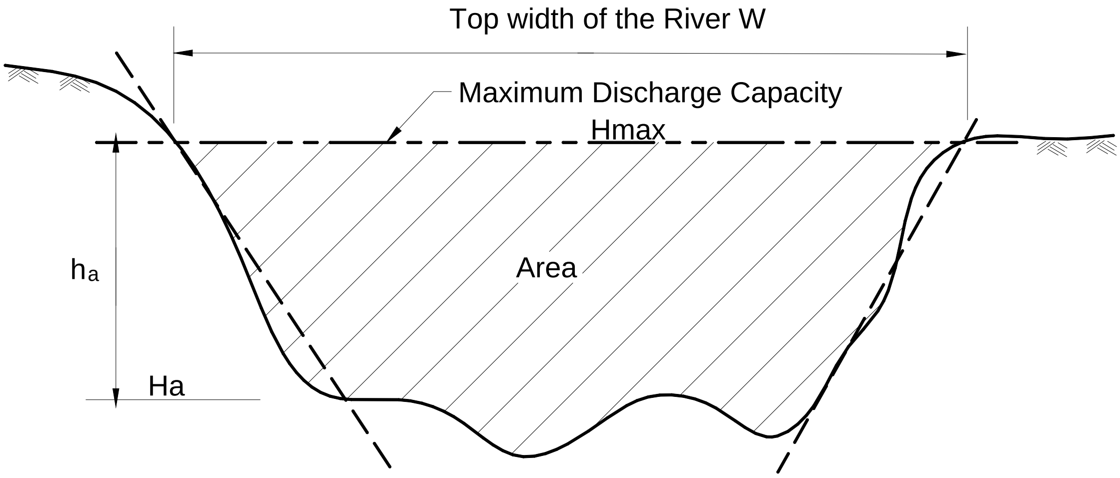


Figure 3.3.5c: Estimation of Average Riverbed Elevation

Identify the area (A) below the elevation (H_{max}) of maximum capacity of the each cross-section from the plotted cross-section by using planimeter, cross-section millimeter paper, triangulation, computer software or any applicable method. Identify the top width (W) of the area. The average depth (ha) and the average riverbed elevation (H_a) can be calculated using the formula below:

ha = A / W　　　　Equation 3.3.5a

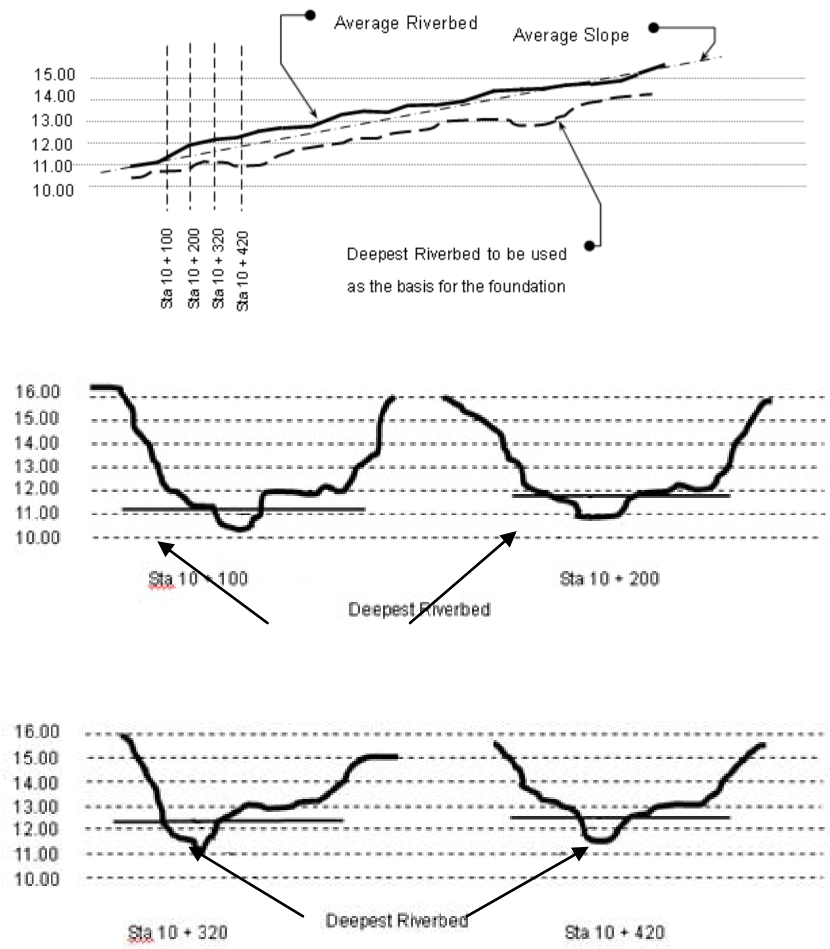
H_a = Hmax – ha　　　　Equation 3.3.5b

iii　Make longitudinal profile of average riverbed elevation

Using the average riverbed elevation mentioned above, the longitudinal profile of average riverbed elevation can be prepared as shown in Figure 8.3d. In this figure, the deepest riverbed should also be indicated as reference for the foundation depth of revetment.

iv　Estimation of gradient (slope) of riverbed elevation

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



Legend :

______________ Average riverbed
------------------- Average slope

Figure 3.3.5d: Profile of Riverbed Elevation and Average Slope

b) Estimation of gradient (slope) of riverbed by using NAMRIA map.

If there is no available longitudinal profile, a NAMRIA map with a scale of 1:50 000 or better can be used. Using the contour as a guide, the gradient can be roughly computed.

Identify the project site in the map, then the two contours that will pass upstream and downstream of the project.

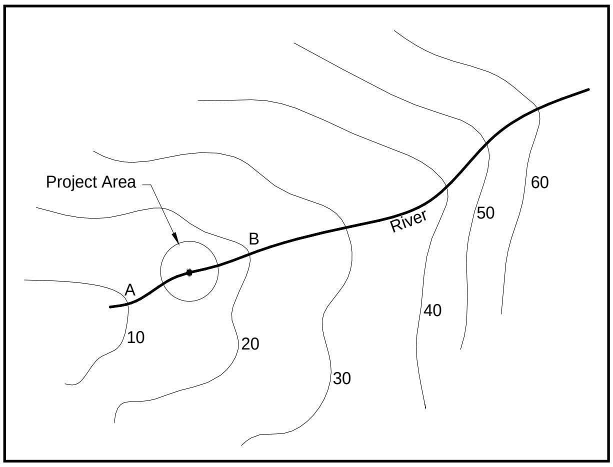


Figure 3.3.5e: Gradient of Riverbed by NAMRIA Map

To compute for the gradient (slope), the formula is:

$$S = \frac{ElevB - ElevA}{Distance} \qquad \text{Equation 3.3.5c}$$

Where:

ElevB	:	the elevation of upstream
ElevA	:	the elevation downstream.

Distance is the length of the river from point (A) to point (B).

To get the distance, measure the length using a ruler, a string, culvimeter or any method applicable on the map.

For 3 or more contours passing along the proposed project site, consider the first segment (the segment between the two contours starting from downstream). Then the next segment, and etc.

For example,

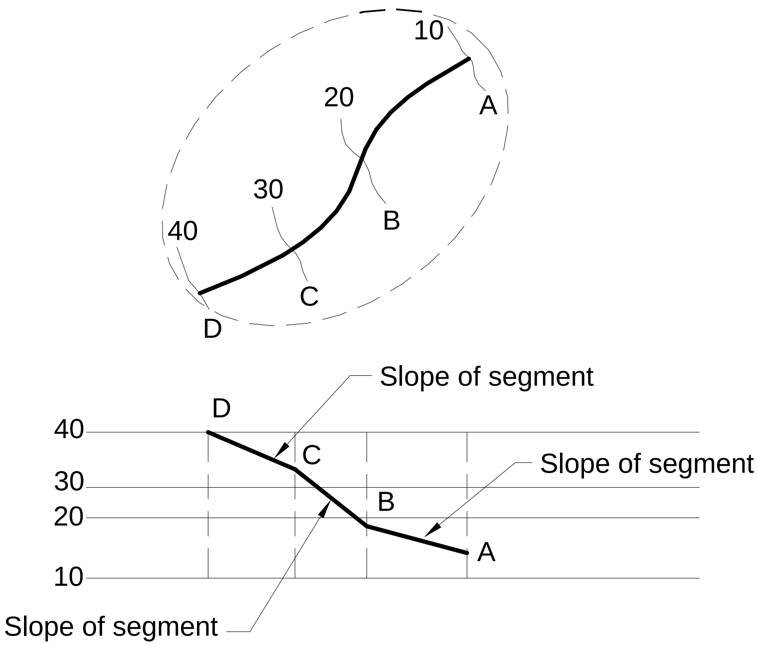


Figure 3.3.5f: Project Along Three or More Contours

The computation is basically the same.

Chapter 4 IDENTIFICATION OF TARGET RIVER BASIN

4.1 Catchment Area

The catchment area is derived by delineating the basin boundary (in polygon) in a contour map. A perpendicular curve to the contour lines is drawn using the latest topographic map with a scale of 1:50,000 or better from the National Mapping and Resource Information Administration (NAMRIA) may be used. The catchment area is then computed using the following:

1) planimeter
2) triangulation
3) cross-section mm paper
4) AutoCAD / GIS software

In runoff analysis, divide the catchment areas into several smaller areas (100 to 200 km^2) depending on the control points, sub-control points, tributary, expected dam location, etc.

Catchment areas shall be subdivided as follows:

1) Delineate from a NAMRIA Map with scale of 1:50,000 or better or from a GIS platform.
2) Measure the inland flood area separately to reflect the flood retarding effect for development of flood run-off model.
3) If a dam is planned, delineate the catchment from the proposed site.

4.2 Establishment of Reference/Sub-reference Points

Reference/Control points are usually where the design discharges are set, strategically accessible for data collection (e.g., observer's nearby house, bridge, etc.) and adjacent to or in significant areas. Reference/control points are established to provide sufficient hydraulic data as bases for hydraulic and hydrologic analyses.

When the catchment area is larger than 100 km^2, sub-reference/control points in the main stream and its tributaries are set. If there is no water level gauge at the control point, installation of a gauge is necessary to verify the output of the runoff model discussed in Section 5.4. At least one reference point is required, where the safety level for the river system is determined, at the neighboring city/town to be protected from flood. Water level and discharge data at the reference/sub-reference points are then collected for flood and probable discharge analysis for flood control planning.

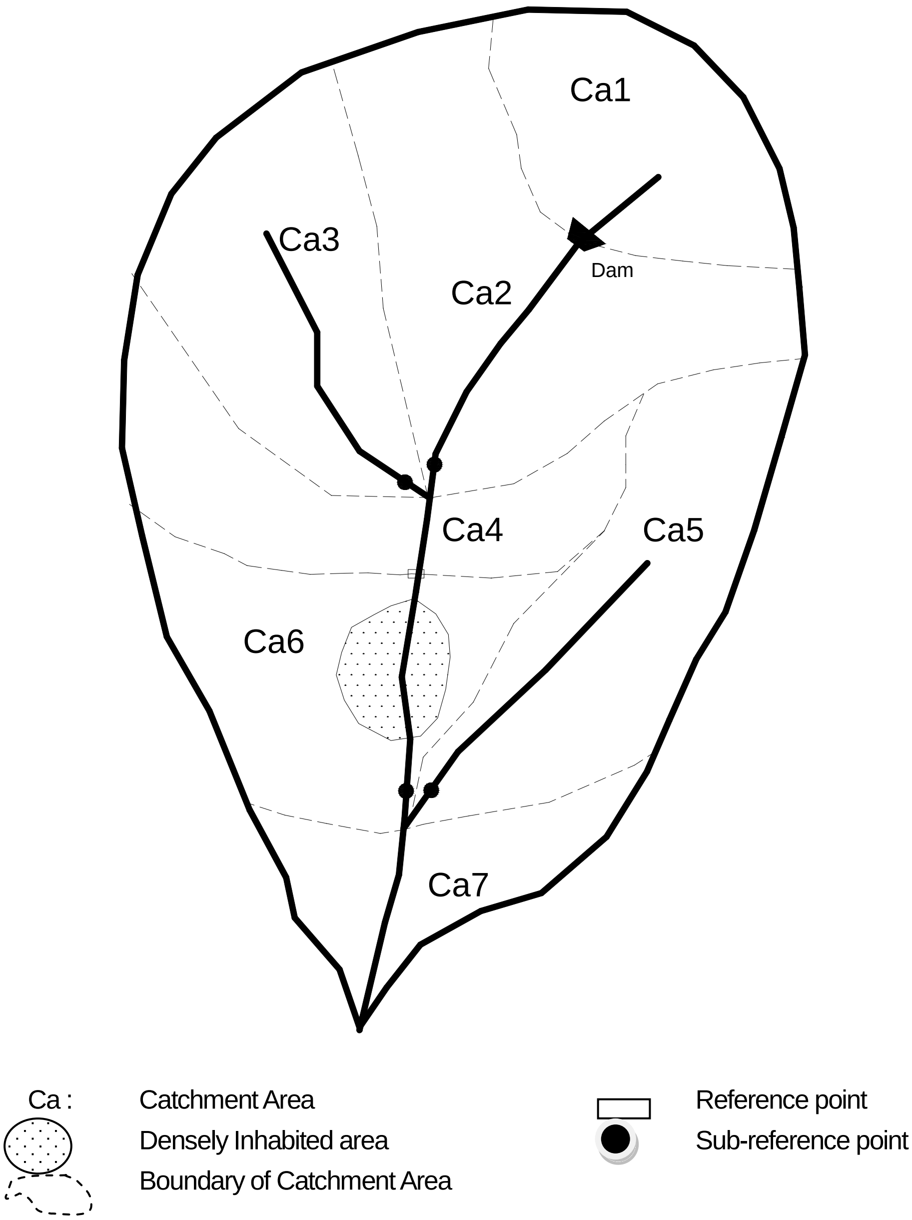


Figure 4.2: Typical Catchment Configuration

Chapter 5 DESIGN DISCHARGE

The design flood frequency and the design discharge hydrograph are determined and established, respectively, at the reference points in consideration of the significance of the river system catchment area

5.1 Procedure in the Determination of the Design Discharge

Design discharge is required to determine the appropriate type of countermeasures in flood control planning and structural design. It can be estimated by 1) specific discharge method or 2) runoff model using rainfall data through the procedure described in Figure 5.1a

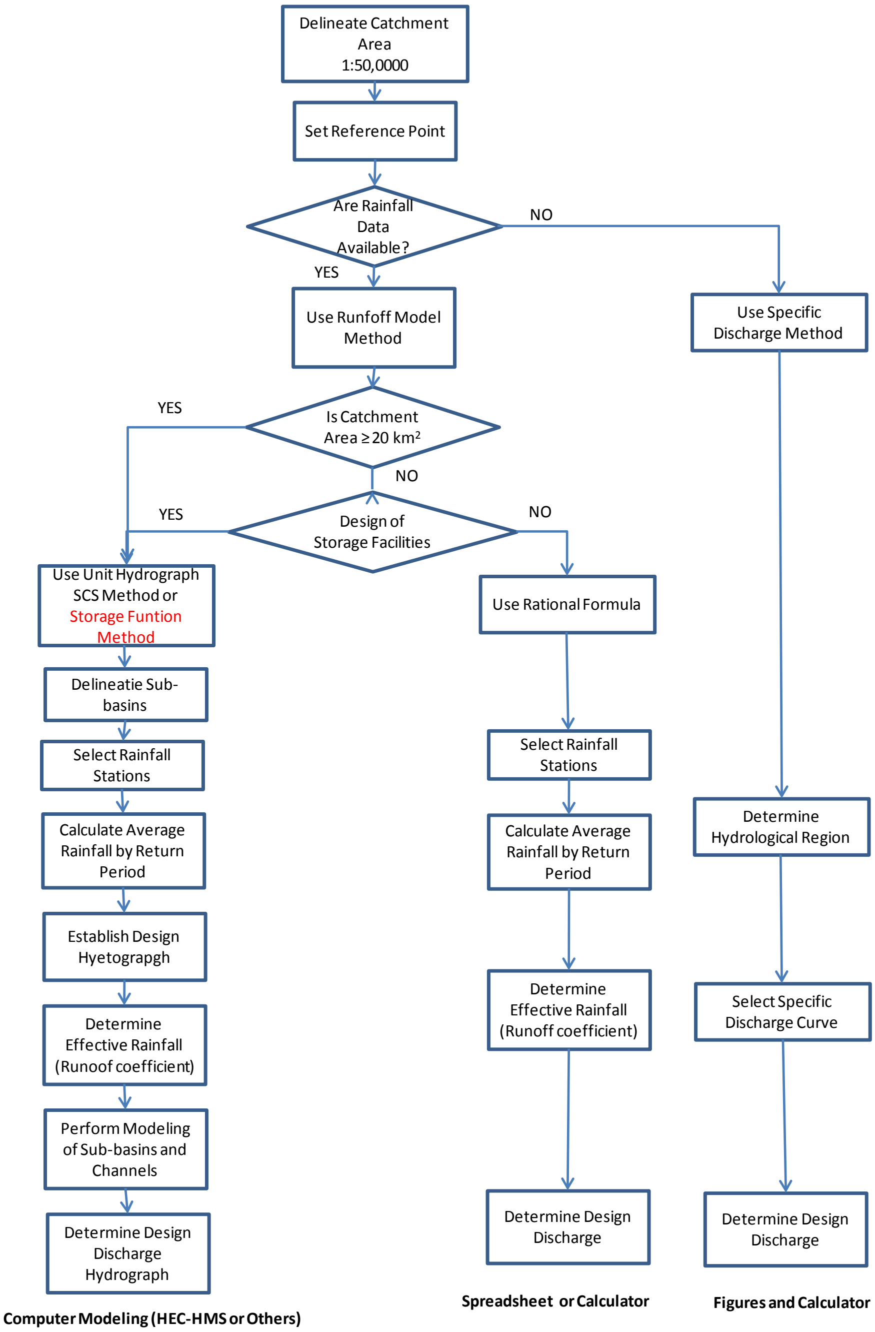


Figure 5.1a: Flow diagram in determining Design Discharge

<Q> = Long-term target discharge (Master Plan)

Q = Design discharge (Implementation Plan)

[Q] = Existing discharge capacity

Figure 5.1b: Diagram of Discharges (Example)

After calculating the design discharge corresponding to the return period, the following step is done:

1) Long-term target discharge is determined based on the degree of importance of area surrounding the river (e.g.; major city located near the river compared to paddy fields, etc.) and defined by its return period.

2) Design discharge sets the improved river capacity (target level)

3) If it is found out that a control point has insufficient capacity, any of the following countermeasure or a combination may be considered;

   - Dam
   - Retarding basin
   - Embankment
   - Widening
   - Etc.

Based on the above parameters and the attainable budget appropriation, estimate roughly the cost. If not economically viable, then the design discharge is reduced. (For further and more detailed discussion refer to Chapter 9, Socio-Economic Analysis)

5.2 Specific Discharge Method

The specific discharge is the flood peak discharge per unit catchment area. (Refer to equation 5.2a). Generally, the specific discharge for small rivers is comparatively larger than that of the bigger rivers. The specific discharge curve explains this (refer to Figure 5.2a Specific Discharge Curve, where the specific discharge is the ordinate and the size of the catchment area as the abscissa). From this curve, design discharge is roughly calculated even without any runoff analysis. The reliability of the design discharge estimated by runoff methods can be easily assessed by comparing it with specific discharge method.

$$Q = A\,q \qquad \textit{equation 5.2a}$$

Where :

q	:	specific discharge ($m^3/s/km^2$)
Q	:	*design discharge* (m^3/s)
A	:	*catchment area* (km^2)

Table 5.2 indicates constants of the Creager type specific curve for the following equation.

$$q = c \cdot A^{(A^{-0.048} - 1)} \qquad \text{Equation 5.2b}$$

Where:

c	:	constant (Table 5.2a)
A	:	catchment area (km^2).

Table 5.2a: Constant (*c*) 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 discharge curve or equation, design discharge or probable discharge is obtained as follows.

1) Determine the catchment area (A).
2) Determine the return period or safety level.
3) From the specific discharge curve, find the region where the project is located, the return period and the catchment area in Figure 5.2a.
4) Another way is to compute specific discharge (***q***) from the equation, using catchment (A) and constant (c) from Table 5.2 a with corresponding regions and return periods.

Example:

Location : Visayas
Catchment Area : 412 km^2
Return Period : 50 years
c = 14.52 from Table 5.2a

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

$$q = 14.52 x 412^{(412^{-0.048} - 1)}$$

q	=	$3.20 \quad m^3/s/km^2$
Q	=	qA
	=	$1{,}319.89\ m^3/s$

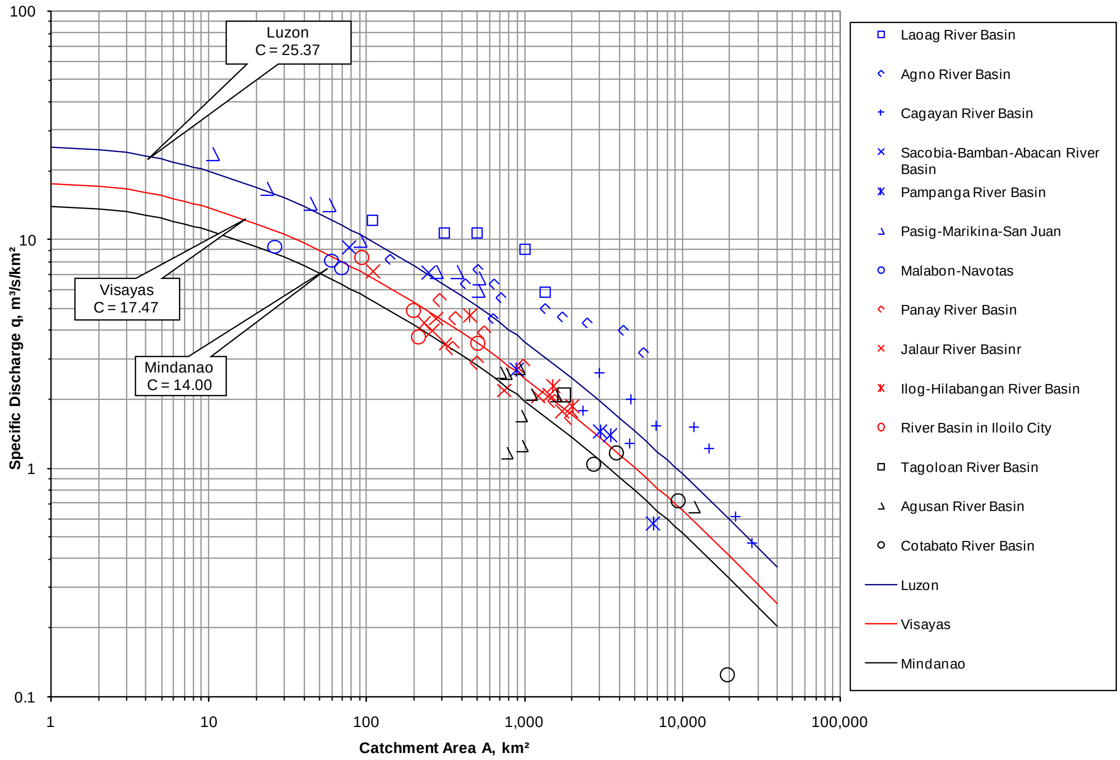


Figure 5.2: Specific Discharge Curve

Referring to the formula for finding q and Table 5.2a Constants for Regional Specific Discharge Curve, the values for q are obtained for Luzon, Visayas and Mindanao with equivalent values similar to Figure 5.2. Please refer to Specific Discharge Curve for different return period.

Table 5.2b: Specific Discharge Example

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. Specific Discharge Method is applicable only to catchment areas with more than 20 km^2, otherwise Rational Formula is recommended.

In addition to the above explanation, the percentage of urbanized area within the catchment area is also an important factor in assessing the flood peak discharge per unit catchment area. In case there are two (2) catchment areas with the same size, the one with the higher percentage of urbanized area will most likely have a higher flood peak discharge than the other.

5.3 Rainfall Analysis

The return-period of the flood is determined through the rainfall data available from PAGASA. These are the only readily available data to calculate the discharge of probable flood .

For catchment areas below 20 km^2, a Rainfall Intensity Duration Frequency Curve shall be utilized in calculating the discharge using the Rational Formula Method. However, in cases where there are no available rainfall data, the RIDF Curve may be utilized in calculating the discharge for catchment areas up to 100 km^2 by the use of Rational Formula (applicable to larger basins where rainfall of uniform space distribution is expected and without retarding effect), otherwise use unit hydrograph method.

For catchment areas greater than 20 km^2, the following procedure shall be followed:

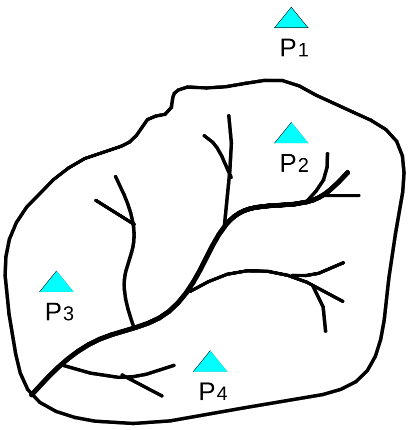
1) Calculate average rainfall in catchment area (see section 5.3.1).
2) Calculate design total rainfall amount by return periods (see section 5.3.2).
3) Collect typical rainfall patterns (hyetographs) of past major floods (see section 5.3.3).
4) Modify typical rainfall patterns based on return period (see section 5.3.4).

5.3.1 Average Rainfall in Catchment Area

There are three (3) methods of determining the catchment area average rainfall, i.e. a) Arithmetic-Mean Method, b) Thiessen Method and c) Isohyetal Method.

1) Arithmetic-Mean Method

   This is the simplest method by averaging the rainfall depths recorded at a number of gages. This method is satisfactory if the precipitation is almost uniformly distributed within the catchment area.



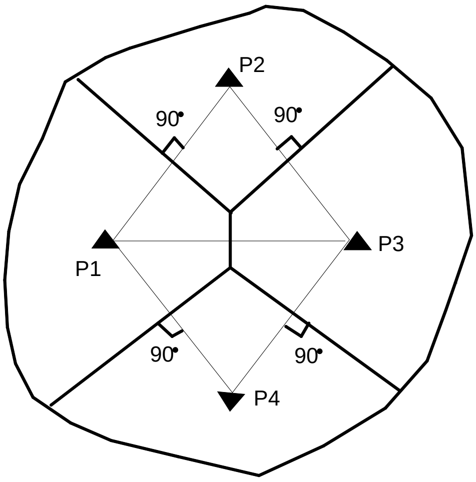
Station	Observed rainfall within the catchment area (mm)
P2	30.0
P3	40.0
P4	50.0
Average Rainfall = 120.0 / 3 = 40.0 mm	

Figure 5.3.1a: Arithmetic-Mean Method (Example)

2) Thiessen Method

   This method assumes that at any point in the catchment area, the rainfall is the same to the nearest rainfall gauge. The value recorded at a given rainfall gauge can be applied halfway of the next station in any direction.

   The relative weights for each gauge are determined from the corresponding areas of application in a Thiessen polygon network, the boundaries of the polygons formed by the perpendicular bisectors of the lines connected to the adjacent gauges.



Station	Observed	Area	Weighted
	Rainfall (mm)	(km^2)	Rainfall (mm)
P1	20.0	8.0	160.0
P2	30.0	5.0	150.0
P3	40.0	8.0	320.0
P4	50.0	7.0	350.0
Total		28.0	940.0
Ave Rainfall = 980.0/28.0 = 35 mm			

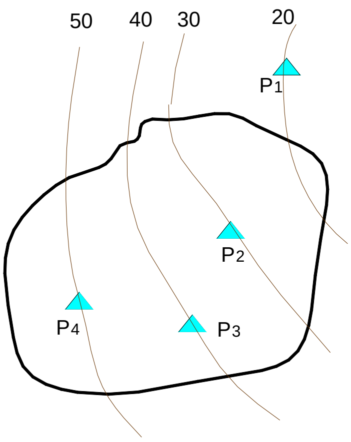
Figure 5.3.1b: Thiessen Method (Example)

3) Isohyetal Method

This method takes into account the orographic influences (mountains, terrain, etc.) on rainfall by constructing isohyets, using observed depths at rain gauges and interpolation between adjacent rain gauges.

Once the isohyetal map is constructed, the area A_j between isohyets, within the catchment, is measured and multiplied by the average rainfall depths P_j of the two adjacent isohyets to compute the average rainfall.

Information of the storm patterns can result in more accurate isohyets; however, a fairly dense network of rain gauges is needed to accurately construct the isohyetal map from a complex storm.



Station	Observed Rainfall (mm)	Area Enclosed (km^2)	Average Rainfall (mm)	
P1		0.5	15.0	7.5
	20.0			
		3.5	25.0	87.5
P2	30.0			
		6.5	35.0	227.7
P3	40.0			
		6.0	45.0	270.0
P4	50.0			
		4.5	55.0	247.5
Total		21.0		840.0
Average Rainfall = 840.0 / 21.0 = 40.0 mm				

Figure 5.3.1c: Isohyetal Method (Example)

5.3.2 Design Total Rainfall Amount by Return Period (1-day, 2-day, 3-day, etc.)

To determine the design total rainfall amount by return period (1-day, 2-day, 3-day, etc.), collect available rainfall data records from PAGASA, other government/non-government institutions and/or private firms with a period of preferably fifteen (15) years or more.

Design rainfall duration should be determined based on the observed lag-time between the peak rainfall and peak flood. If the lag-time is within one (1) day (this is the case in the Philippines), hourly rainfall distribution should be developed based on design total one (1) day rainfall amount. If lag-time is more than one (1) day, design rainfall duration should be more than four (4) days. Lag-time between peak rainfall and peak flood reflects the basin capacity for floodwater storage.

Table 5.3.2: Annual Maximum 1-day Rainfall Amount (Example)

Year	Dates of Occurrence	Maximum Annual 1-day Rainfall Amount (mm)
1985	September 2	200
1986	August 14	315
1987	October 4	510
……		
2000	September 16	**280**

Table 5.3.2 shows the annual maximum 1-day rainfall amount recorded during the 16 year period.

Hydrological quantities for a certain return period may be roughly estimated using probability paper. Available annual maximum rainfall amount data (1-day, 2-day, 3-day, etc.) are collated from the highest to the lowest value, arranged from 1 (n = 1) and 2 (n = 2) and so forth, from the highest value to the number of data. The data are then plotted using a probability paper with plotting positions using the Weibull or Hazen plots. Usually, the value obtained using the Weibull plot is higher than that of the Hazen plot in the upper range of the distribution, hence Weibull plot is on safety side.

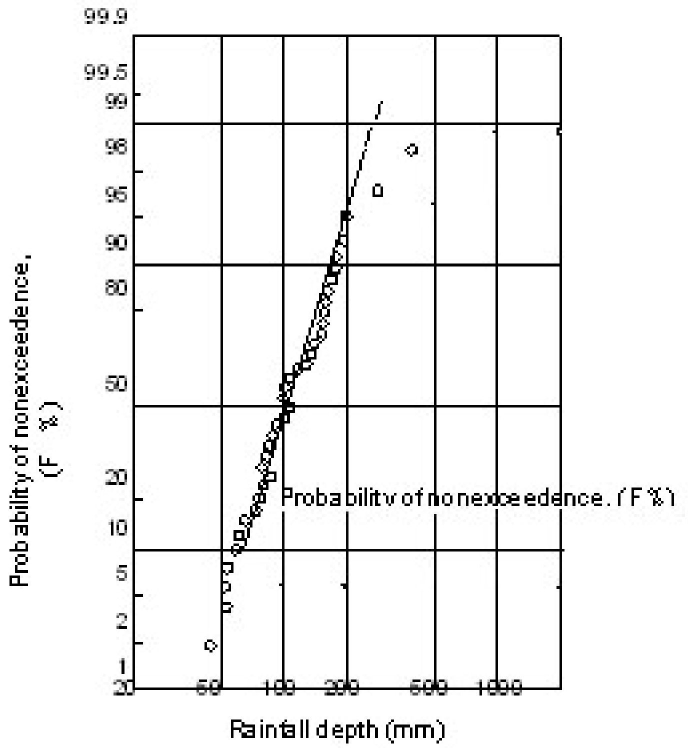


Figure 5.3.2: Plot of Annual Maximum Rainfall 1-day Amount (Example)

Weibull Plot:

$$F(X_n) = \frac{n}{N+1}$$ Equation 5.3.2a

Where:

$F(x_n)$	:	probability of non-exceedance
N	:	number of data
n	:	order from the highest value (1, 2, 3 …. N)
x_n	:	rainfall of order n

To estimate the value of the return-period, a line is manually drawn based on the plotted points and extended to the upper range of the distribution, as shown in Figure 5.3.2. Thus, a relationship between the probability of non-exceedance/return period and the design total rainfall amount could be established.

The return period is calculated as:

Return Period = 1 / (1-F) Equation 5.3.2b

For example, if f = 0.95 the return is 20 years and the rainfall amount is 200 mm in Figure 5.3.2

5.3.3 Typical Rainfall Patterns of Past Major Floods

After deriving the design total rainfall amount, the hourly distribution of the rainfall shall be arranged by expanding or contracting some past rainfall patterns. The total rainfall amount is the design total rainfall amount. In selecting past rainfall events, those that caused severe floods or with high recurrence patterns in the basin shall not be exclude.

The maximum extension of the y-coordinates is about 200%.

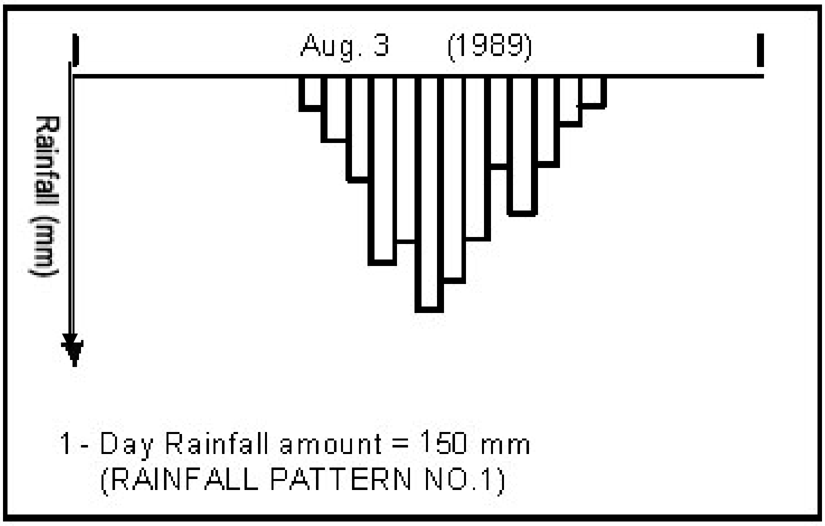


Figure 5.3.3: Rainfall Pattern of Past Major Floods (Example)

5.3.4 Modification of Typical Rainfall Patterns Based on Return Period

Modify the typical rainfall patterns selected in Section 5.3.3 by expanding or contracting to the design rainfall amount calculated in Section 5.3.2. For example, Rainfall Pattern No. 1 (1-day rainfall amount of 150 mm) is modified by multiplying each hourly rainfall by the ratio of the total rainfall amount in Section.5.3.2 (200 mm) and that of Rainfall Pattern No. 1, i.e., hourly rainfall of Rainfall Pattern No. 1 multiplied by the ratio 200 mm/ 150 mm (see Figure 5.3.4)

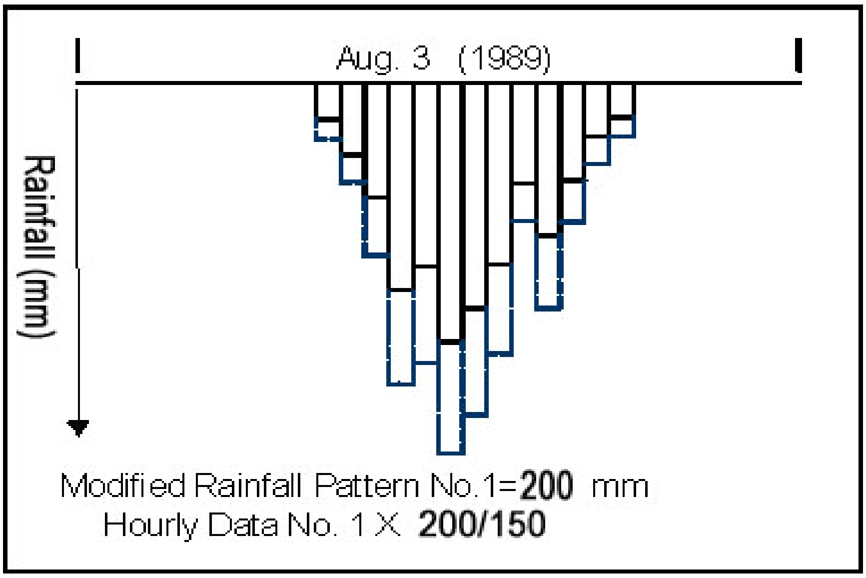


Figure 5.3.4: Modified Rainfall Pattern (Example)

When the actual rainfall duration is larger than the design rainfall duration, expand the time up to the design rainfall duration only.

5.4 Runoff Analysis

Runoff analysis is required for planning of flood control facilities/structures and discharge control, particularly for the construction of dams and retarding basins. It aims to establish the relationship between the amount of rainfall and the river discharges.

There are many methods for runoff analysis; however, introduced in this Volume are the following:

1) Rational Formula

2) Unit Hydrograph Method

3) Storage Function Method

5.4.1 Rational Formula

The Rational Formula Method is convenient for estimating flood peak discharge. It is widely applied in rivers where storage phenomena are not required, where the catchment is treated as rectangular, symmetrical about the river course and where the rainwater flows down the river course at a constant speed.

Maximum flood discharge is given by the rational formula:

1) Basic Equation

The principle behind the Rational Formula Method is that a rainfall intensity (I) begins and continues indefinitely and then the rate of runoff increases until it reaches the time of concentration (tc), where all of the watersheds are contributing to the flow at the outlet point or point under consideration. The Rational Formula is applicable to a catchment area smaller than 100 km^2 . However, it is applicable to larger basins where rainfall of uniform space distribution is expected and without retarding effect.

$$Q_P = \frac{cIA}{3.6}$$ Equation 5.4.1

Where:

Q_P	:	maximum flood discharge (m^3/s)
c	:	dimensionless runoff coefficient
I	:	rainfall intensity within the time of flood concentration ($^{mm}/_{hr}$)
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 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.

2) Runoff Coefficient (*c*)

The runoff coefficient (*c*) is the least precise variable of the Rational Formula implying 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, the slope and the 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, aerial photographs, and present land use maps are useful in estimating the nature of the surface within the target basin. Runoff coefficient will increase with urbanization due to increased impervious surface and installation of drainage system. In a large-scale development, projected runoff coefficient due to development should be used to determine the design discharge and the expected safety level. The future land use plan can be obtained from the LGUs.

Table 5.4.1a: 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
For watershed having steep gullies and not heavily timbered	0.55-0.70
For watershed having moderate slope, cultivated and heavily timbered	0.45-0.55
For suburban areas	0.34-0.45
For agricultural areas	0.15-0.25

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

Table 5.4.1b: Runoff Coefficient applied for Metro Manila and other Urban Cities

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

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

After the present and the future land uses are obtained, the area is categorized and measured to obtain the percentage of each category to the total catchment area. From the percentage of each area, the weighted average runoff coefficient is calculated.

Table 5.4.1c: Example of Percentage of Land Use Category

Total area of the Sub-Basin 22.83 km^2	Urban area			Industrial 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).

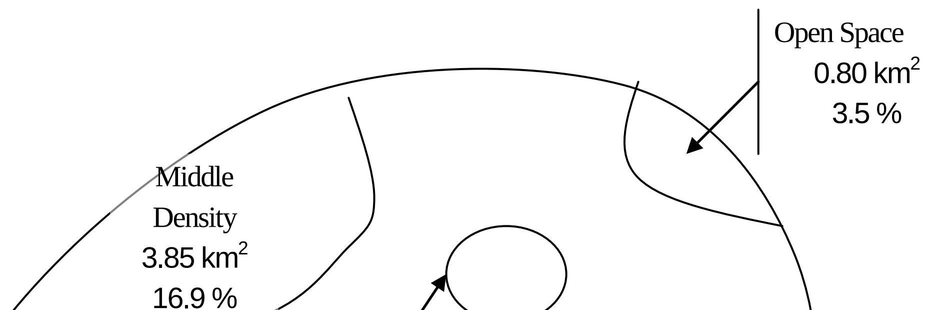


Figure 5.4.1a: Example of Land Use Category

3) Time of concentration

a) Kirpich's Formula

Time of concentration (t_c) is determined by applying Kirpich Formulas below.

$$t_c = \frac{0.0195 L^{0.77}}{S^{0.385}} \qquad \text{Equation 5.4.1aa}$$

$$t_c = \frac{L^{1.15}}{51 H^{0.385}} \qquad \text{Equation 5.4.1ab}$$

Where:

t_c	:	Time of concentration (minutes)
L	:	Length of watercourse (m)
S	:	Average basin slope (S=H/L)
H	:	Difference in elevation (m)

b) Dividing time of concentration into inlet time and flow time

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

$$t_c = t_i + t_f \qquad \text{Equation 5.4.1b}$$

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)

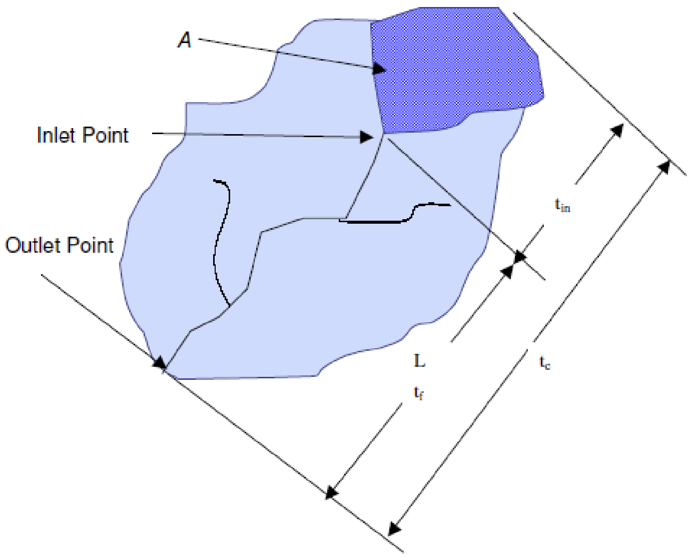


Figure 5.4.1b: Inlet and Outlet Points of Rational Formula

Inlet Time

Inlet time is computed as follows.

i . Find the inlet point (Figure 5.4.1b). If the estimated inlet catchment area is over 2 km^2 , the inlet time is t_i = 30 min.

ii . 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

$$t_i = \frac{30\sqrt{A}}{\sqrt{2}}$$ Equation 5.4.1c

Flow Time

Kraven's Formula

Flow time is computed from Kraven's Formula (Table 5.4.1d), which gives relations between slope of water course and flow velocity as shown below.

Table 5.4.1d:Kraven's Formula

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

[Example] Computation of Flow Time

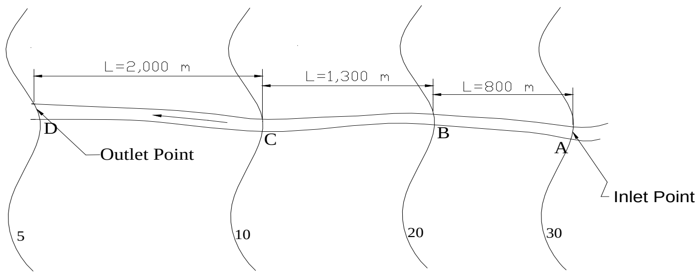


Figure 5.4.1c: Sample River with Contour Lines

ⅰ. Find the intersection of river and contour line from the map (As A, B, C, D in Figure 5.4.1c)

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

ⅲ. Compute the gradient using the following formula

$$Gradient = \frac{(ElevationA) - (ElevationB)}{(Dis\tan cefromAtoB)}$$ Equation 5.4.1d

$$S = \frac{E_A - E_B}{L}$$ Equation 5.4.1e

Where :

S	:	Gradient from A to B
E_A	:	Elevation A
E_B	:	Elevation B
L	:	Distance from A to B

iv. Compute for the flow time

Flow velocity

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

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

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

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

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

$S_{C-D} < \frac{1}{200}$ 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 safety level indicated in the form of return period for a catchment area during the concentration time.

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

a) RIDF of PAGASA Synoptic Rainfall Station

The following section requires publication "Specific Discharge Curve, Rainfall Intensity Duration Curve, Isohyet of Probable 1-day Rainfall" by Project ENCA, March 2003, as reference materials.

PAGASA operates/maintains 52 Synoptic stations equipped with automatic rainfall gauges mentioned in the above mentioned publication. The location of the selected 39 PAGASA Synoptic stations are recommended for runoff analysis. (Refer to “LOCATION OF SELECTED CLIMATIC AND SYNOPTIC PAGASA STATIONS). When one of the stations is inside or near the target river basin, its RIDF is used to obtain rainfall intensity during the concentration time. See RIDF OF SELECTED SYNOPTIC PAGASA STATIONS.

RIDF can be expressed as follows:

Type 1: $$I = \frac{A}{(C + T^b)}$$ Equation 5.4.1fa

Type 2: $$I = \frac{A}{(C + T)^b}$$ Equation 5.4.1fb

Where:

I	:	*Rainfall intensity in mm/hr (Please note that R instead of I is used in the* Rainfall Intensity Duration Curve)
A	:	Catchment Area (square kilometers)
C	:	*Coefficient from the table*
T	:	*time of concentration in (minutes)*
b	:	*coefficient from the table*

Type and coefficient of best-fit equations for the selected station are indicated in Rainfall Intensity Duration Frequency Curve (RIDF) with return period of 2-yr., 3-yr., 5-yr., 10-yr., 25 yr., 50 yr. and 100 yr.

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

[Example] The above RIDF is expressed for Vigan.

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

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

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

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 (refer to ISO-SPECIFIC COEFFICIENT) and 1-day probable rainfall (refer to ISOHYET OF PROBABLE 1-DAY RAINFALL). The procedures to obtain RIDF are as follows.

i. Read representative specific coefficient (β) of the target river basin from ISO-SPECIFIC COEFFICIENT. The representative point can be the centroid of the river basin.

ii. Read probable 1-day rainfall (I_{24}) corresponding to the n-yr return period from ISOHYET OF PROBABLE 1-DAY RAINFALL.

iii. Compute b from the following equation.

$$b = \frac{\log \beta}{\log 24 - \log 1} = \frac{\log \beta}{1.3802} \quad \text{Equation 5.4.1g}$$

iv. Obtain RIDF from the following equation.

$$I_t = \left(\frac{24}{t}\right)^b I_{24} \quad \text{Equation 5.4.1h}$$

Where :

I_t	:	rainfall intensity for duration t (mm/hr)
I_{24}	:	1 day rainfall intensity (mm/hr)
t	:	time in hours hr.

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

Table 5.4.1e: Example of Application of Specific Coefficient ß

Step 1	**Location**		**Capas Tarlac**			
	Coordinates		1,6952941.14 E		232,920.94 N	
Step 2	**Read I (I_{24})**					
	Return Period				**I, 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 centroid 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 $I_t = \{24/t)^b x(I/24)$

Note: Values of I were taken from the Specific Discharge Curve, Rainfall Intensity Duration Frequency Curve, Isohyet of Probable 1-day Rainfall, Project for the Enhancement of Capabilities in Flood Control and Sabo Engineering of the DPWH, March 2003

Kirpich Formula is most commonly used in the rational formula, while; Kraven's Formula may be used to validate the values obtained from Kirpich Formula. Kraven's Formula is widely used in Japan; however, there is no reference written in English.

5.4.2 Unit Hydrograph Method

A hydrograph is a continuous representation of instantaneous rainfall against time. The combination of physiographic and meteorologic conditions integrated with effects of climate, losses, surface runoff, interflow and groundwater flow in a river basin or catchment area results to hydrograph.

1) Basic Assumptions

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 5.4.2a,b,c, and d).

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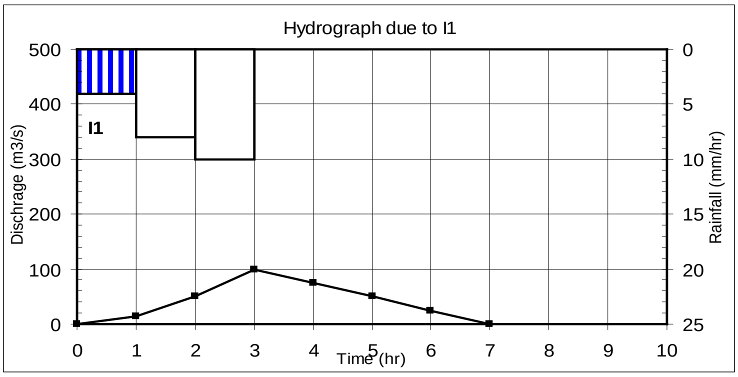


Figure 5.4.2a: Hydrograph Due Rainfall I1

The hydrograph is produced by the rainfall I_1 with a certain base width.

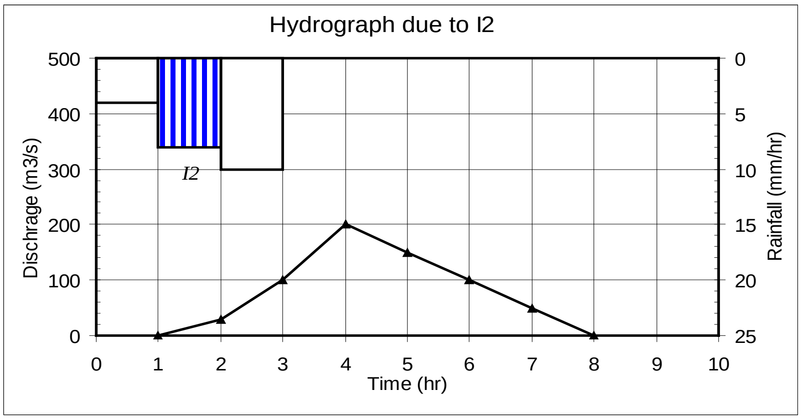


Figure 5.4.2b: Hydrograph Due Rainfall I2

The hydrograph is caused by I_2 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 I_2 occurred after I_1.

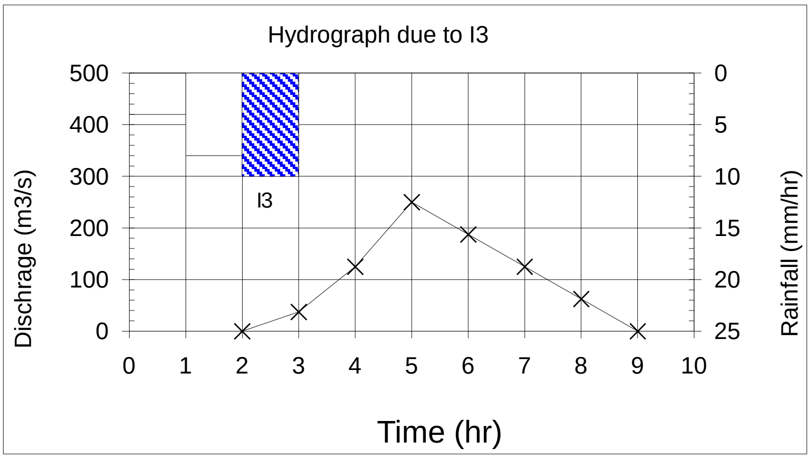


Figure 5.4.2c: Hydrograph Due Rainfall I3

Similarly, the hydrograph has the same base width and time duration, and the rise follows the occurrence of I_3 with proportional volume of runoff.

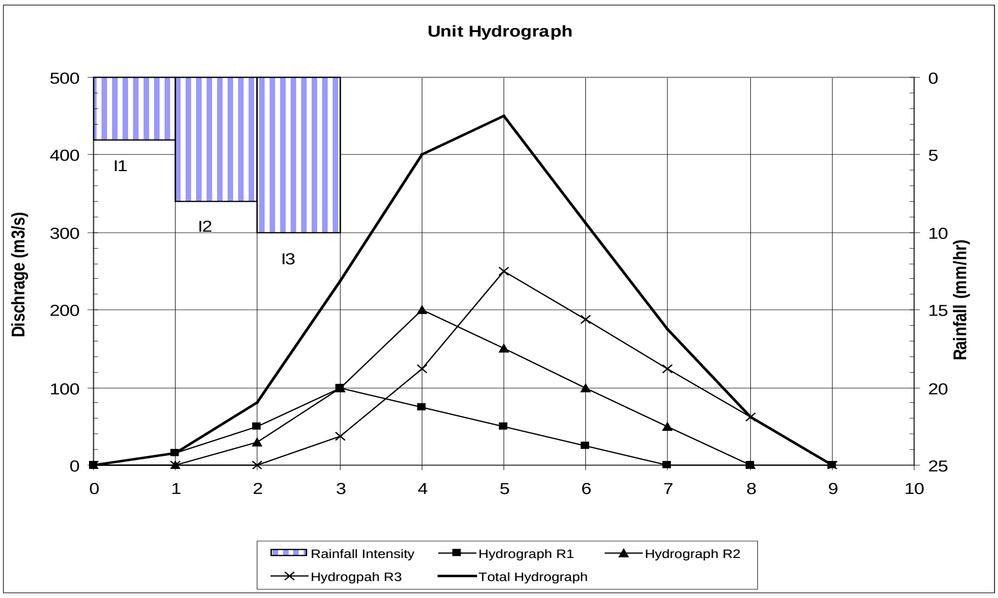


Figure 5.4.2d: Hydrograph Due Rainfall I1, I2, and I3

Collectively, the discharges produced by I_1, I_2 and I_3 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:

$$Q_n = \sum_{i=1}^{n} I_i U_{n-i+1}, \quad \text{or} \qquad \text{Equation 5.4.2a}$$

$$Q_n = I_n U_1 + I_{n-1} U_n + I_{n-2} U_3 + ... + I_1 U_n \qquad \text{Equation 5.4.2b}$$

Where:

Q_n	:	storm hydrograph ordinate
I_i	:	effective rainfall
$Uj\ (j=n-I+1)$	:	unit hydrograph ordinate

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

Table 5.4.2a: Computation of Direct Runoff by Unit Hydrograph Method

Time (hr.)	Effective Rainfall (mm)	Unit hydrograph ordinate ($m^3/s/mm$)						Direct Runoff ($m^3/s/km^2$)
		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

Unit hydrograph for one km^2

The unit hydrograph cannot be applied for basins larger than 5000 km^2. For basins larger than 5000 km^2, unit hydrographs for the principal sub-areas or sub-basins are developed and the hydrographs of runoff determined for each sub-area. These hydrographs are then combined, through flood routing procedure, to get the resulting hydrograph at the required section.

(Source: Hydrology, Principles, Analysis, Design, H.M. Raghunath, New Age International Publisher, New Delhi, 2006)

2) SCS Unit Hydrograph

The unit hydrograph method has been applied to many river basins in many countries where several synthetic unit hydrographs have been developed. 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.

Emphasized in this section is the SCS Unit Hydrograph (UH), by the Soil Conservation Service (SCS, presently Natural Resources Conservation Service, NRCS). it is recommended because of 1) ease in determining the shape of the unit hydrograph and 2) it is widely used in many countries.

The SCS dimensionless hydrograph is a synthetic UH in which the discharge is expressed by the ratio of discharge (Ur) to peak discharge (U_p) and the time by the ratio of time t to the time of peak of *UH, Tp*. 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 sub-basin) and from U_p and T_p, the unit hydrograph for the basin (or the sub-basin) can be obtained.

a) Basic Concept and Equations

The SCS UH is a dimensionless, single-peaked UH as shown in Figure 5.4.2e, which is expressed by the ratio of discharge Ut to peak discharge Up and the time by the ratio of time t to the time of peak of UH, Tp

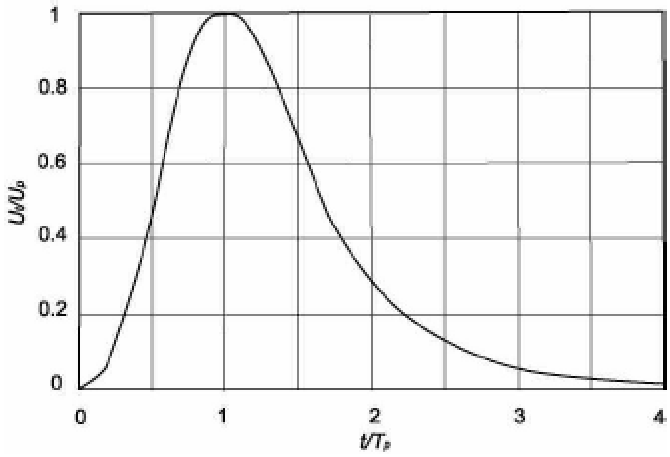


Figure 5.4.2e: SCS Unit Hydrograph

(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 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 5.4.2f.

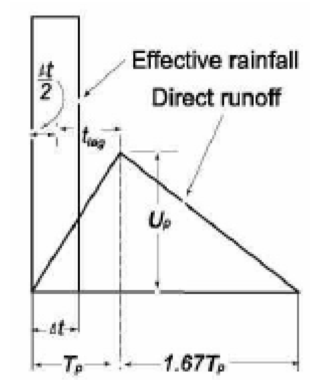


Figure 5.4.2f:A Triangular Unit Hydrograph

$$U_p = 2.08\frac{A}{T_p}$$ Equation 5.4.2c

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

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, Ì t, that is less than 29 % of t_{lag}

must be used.)

b) Estimating the SCS UH Model Parameter

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

For ungauged watersheds, the Modified Snyder Equation is widely applied for estimating the UH lag time in the Philippines.

$$Lg = 0.6865 C_t \left[\frac{LL_{ca}}{\sqrt{S}} \right]^{0.38}$$ Equation 5.4.2d

Where :

Lg	:	lag time
Ct	:	lag time coefficient - for mountainous area = 1.2 - for hilly area = 0.7 - for valley area = 0.35
L	:	length of water course from the upstream sub-basin boundary to the downstream end of the sub basin (km)
Lca	:	length of water course from the downstream end of the sub-basin to the intersection on the stream perpendicular from the centroid of the sub-basin (km)
S	:	average basin slope (overall slope along the longest water course from the downstream end of the sub-basin boundary at the upstream end.)

Time of concentration is estimated as discussed in 5.4.1(3).

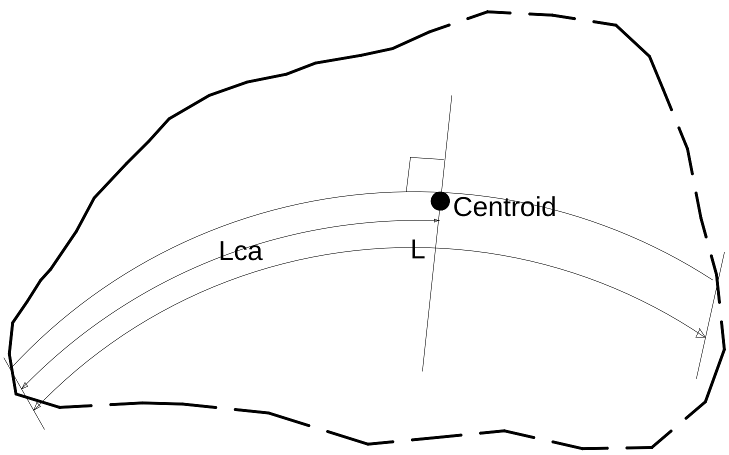


Figure 5.4.2g: Concentration Time for Residual Sub-basin

3) Delineation of Sub-basins

The target basin is subdivided by sub-basins and channels for the application of the SCS unit hydrograph, considering 1) reference point, 2) important points in which flood control structures are planned and 3) river points to which tributaries join.

Area of sub-basins should not be too small, since time interval of computation becomes short. In general, the area of sub-basins for unit hydrograph can be extended to the extent from 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/sub-basins and planned pump discharge is added to the design discharge of the river.

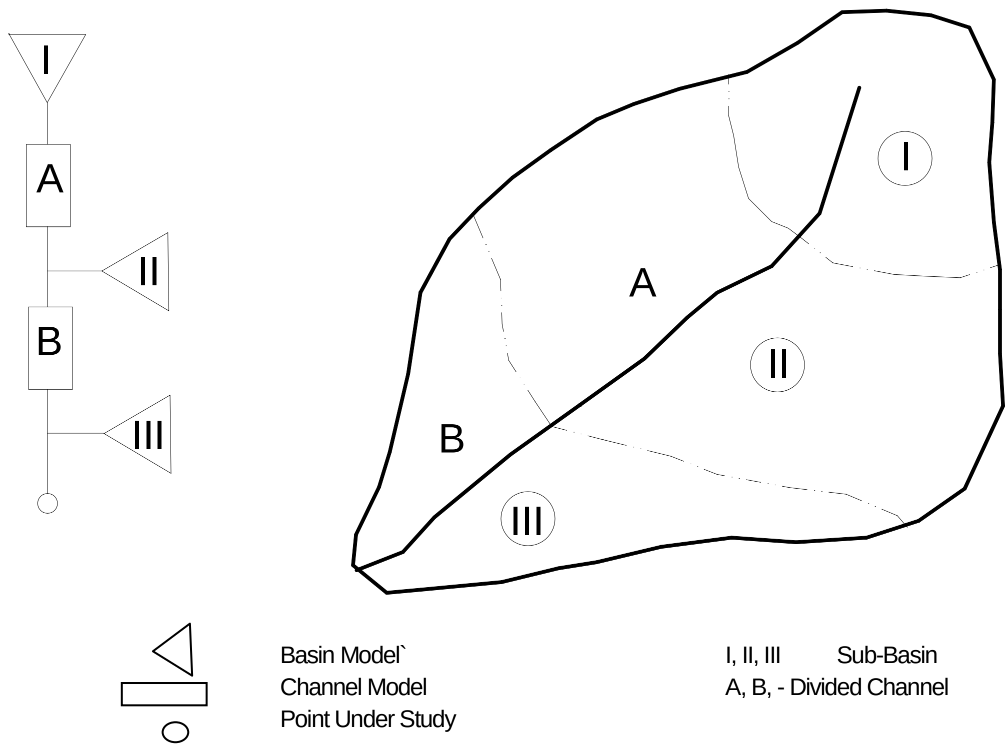


Figure 5.4.2h: Model for Flood Routing and Basin Division

Figure 5.4.2i shows a sample of the San Juan River Basin, a tributary of the Pasig Marikina River. In this case, the basin is subdivided into 10 sub-basins including the junction with its tributaries and 3 channels.

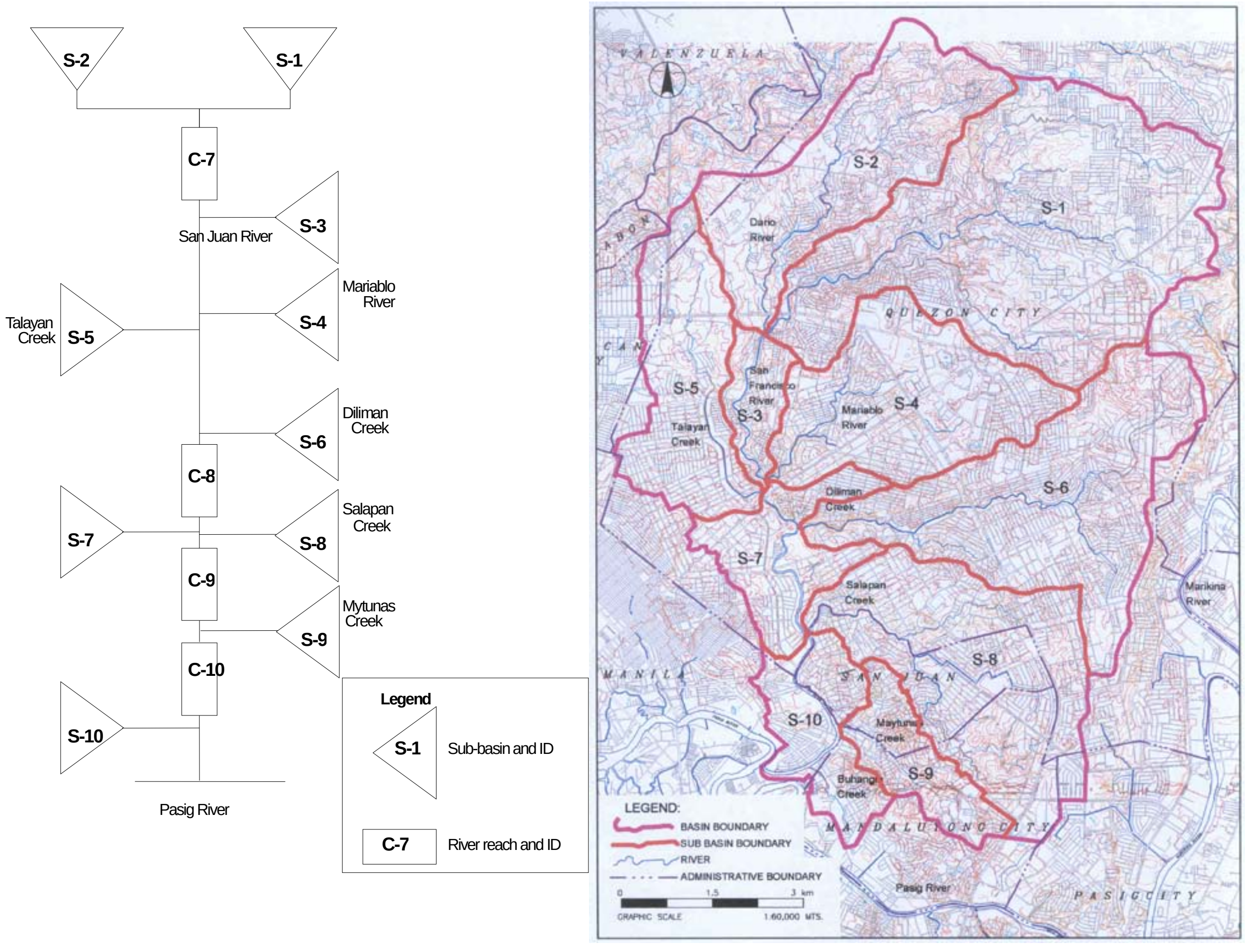


Figure 5.4.2i: Runoff Model for San Juan River Basin

4) Design Hyetograph

a) Methods to Establish Design Hyetograph

There are two methods to prepare design hyetograph, namely 1) one rainfall station method and 2) multiple rainfall station method.

The characteristic of rainfall is expressed by three (3) factors.

i. amount of rainfall,

ii. temporal distribution of rainfall and

iii. aerial distribution of rainfall.

At present, design rainfall cannot be theoretically established from statistical or climatic view points; hence, the two (2) methods 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 5.3 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, e.g., major rivers in the Philippines. The one rainfall station method is explained hereunder.

b) Alternating Block Method (One Rainfall Station Method)

i Rainfall Intensity Duration Frequency Curve

The RIDF curve is necessary to prepare the design hyetograph by alternating block method. As explained in 5.4.1.4) Rainfall intensity, two (2) methods are available to establish the RIDF curve of the target basin.

When the selected PAGASA Synoptic rainfall station is in or near the basin, the RIDF of the station should be used. Location of the selected PAGASA synoptic rainfall station is shown in" LOCATION OF SELECTED CLIMATIC AND SYNOPTIC PAGASA STATIONS" and the RIDF curves are shown in the publication, "Specific Discharge Curve, Rainfall Intensity Duration Curve, Isohyet of Probable 1-day Rainfall" by Project ENCA, March 2003.

When station cannot be located or there is no station, the RIDF can be estimated from the specific coefficient shown in Iso-Specific Coefficient and the probable daily rainfall value shown in Isohyet of Probable 1-Day Rainfall.

ii. 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:

$$I = I_o e^{\left[-0.1(0.386A)^{0.31}\right]}$$ Equation 5.4.2e

Where :

I	:	basin mean rainfall (mm)
I_o	:	point rainfall (mm)
A	:	catchment area (km2)
Fa	:	I/I_o area reduction factor

ⅲ. Hyetograph Preparation

The rainfall intensity is computed from the RIDF curve for each of the durations Δt, $2\Delta t$, $3\Delta t$, 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.

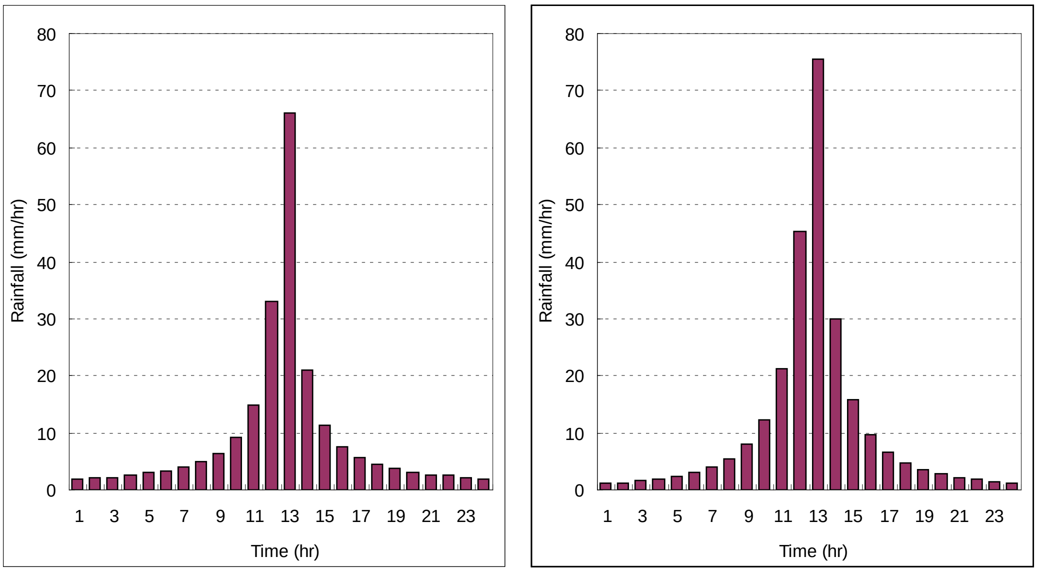
Table 5.4.2b: Hourly Rainfall Values Derived by Alternating Block Method

(1) 10-year Probable Rainfall

Time		R Intensity	Cum. Rain for T hours (b)x(a)	Hourly Rain Incremental of (c)	Position	Design Hyetograph	Hourly Rainfall	
						Hour	Point Rainfall	Reduced by fa
(a) hour	(a') min	(b) mm/hr	(c) mm	(d) mm/hr			mm/hr	mm/hr
1	60	85.25	85.25	85.25	13	1	2.39	1.85
2	120	64.00	128.00	42.75	12	2	2.80	2.17
3	180	51.67	155.01	27.01	14	3	2.83	2.19
4	240	43.56	174.24	19.23	11	4	3.28	2.54
5	300	37.79	188.95	14.71	15	5	3.83	2.97
6	360	33.46	200.76	11.81	10	6	4.26	3.30
7	420	30.08	210.56	9.80	16	7	5.11	3.96
8	480	27.36	218.88	8.32	9	8	6.43	4.98
9	540	25.13	226.17	7.29	17	9	8.32	6.44
10	600	23.26	232.60	6.43	8	10	11.81	9.15
11	660	21.67	238.37	5.77	18	11	19.23	14.89
12	720	20.29	243.48	5.11	7	12	42.75	33.11
13	780	19.10	248.30	4.82	19	13	85.25	66.03
14	840	18.04	252.56	4.26	6	14	27.01	20.92
15	900	17.11	256.65	4.09	20	15	14.71	11.39
16	960	16.28	260.48	3.83	5	16	9.80	7.59
17	1020	15.52	263.84	3.36	21	17	7.29	5.65
18	1080	14.84	267.12	3.28	4	18	5.77	4.47
19	1140	14.23	270.37	3.25	22	19	4.82	3.73
20	1200	13.66	273.20	2.83	3	20	4.09	3.17
21	1260	13.14	275.94	2.74	23	21	3.36	2.60
22	1320	12.67	278.74	2.80	2	22	3.25	2.52
23	1380	12.23	281.29	2.55	24	23	2.74	2.12
24	1440	11.82	283.68	2.39	1	24	2.55	1.98
						Total	283.68	219.72

(2) 25-year Probable Rainfall

Time		R Intensity	Cum. Rain for T hours (b)x(a)	Hourly Rain Incremental of (c)	Position	Design Hyetograph	Hourly Rainfall	
						Hour	Point Rainfall	Reduced by fa
(a) hour	(a') min	(b) mm/hr	(c) mm	(d) mm/hr			mm/hr	mm/hr
1	60	97.61	97.61	97.61	13	1	1.49	1.15
2	120	78.02	156.04	58.43	12	2	1.64	1.27
3	180	64.94	194.82	38.78	14	3	2.01	1.56
4	240	55.59	222.36	27.54	11	4	2.44	1.89
5	300	48.57	242.85	20.49	15	5	3.11	2.41
6	360	43.12	258.72	15.87	10	6	3.90	3.02
7	420	38.75	271.25	12.53	16	7	5.23	4.05
8	480	35.19	281.52	10.27	9	8	7.12	5.51
9	540	32.22	289.98	8.46	17	9	10.27	7.95
10	600	29.71	297.10	7.12	8	10	15.87	12.29
11	660	27.55	303.05	5.95	18	11	27.54	21.33
12	720	25.69	308.28	5.23	7	12	58.43	45.26
13	780	24.06	312.78	4.50	19	13	97.61	75.60
14	840	22.62	316.68	3.90	6	14	38.78	30.04
15	900	21.35	320.25	3.57	20	15	20.49	15.87
16	960	20.21	323.36	3.11	5	16	12.53	9.70
17	1020	19.18	326.06	2.70	21	17	8.46	6.55
18	1080	18.25	328.50	2.44	4	18	5.95	4.61
19	1140	17.41	330.79	2.29	22	19	4.50	3.49
20	1200	16.64	332.80	2.01	3	20	3.57	2.77
21	1260	15.94	334.74	1.94	23	21	2.70	2.09
22	1320	15.29	336.38	1.64	2	22	2.29	1.77
23	1380	14.69	337.87	1.49	24	23	1.94	1.50
24	1440	14.14	339.36	1.49	1	24	1.49	1.15
						Total	339.36	262.85



(a) 10-year Probable Rain (b) 25-year Probable Rain

Figure 5.4.2j: 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; however, when data are available effective rainfall can be established by the relationship between rainfall and runoff. But in most cases, losses are minimal or negligible because normally the soil is saturated before a big flood occurs.

6) 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 there are no data available, 0.05 $m^3/s/km^2$ can be used for the base flow.

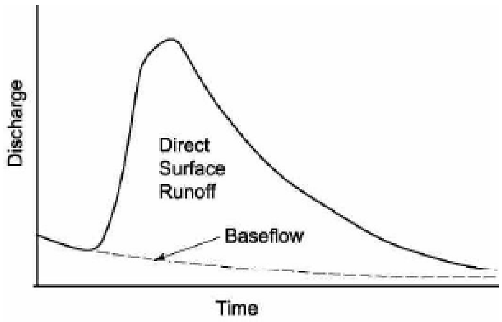


Figure 5.4.2k: Baseflow

7) 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.

Among these methods, the Muskingum-Cunge standard method is recommended due to ease of 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 5.4.1) a).

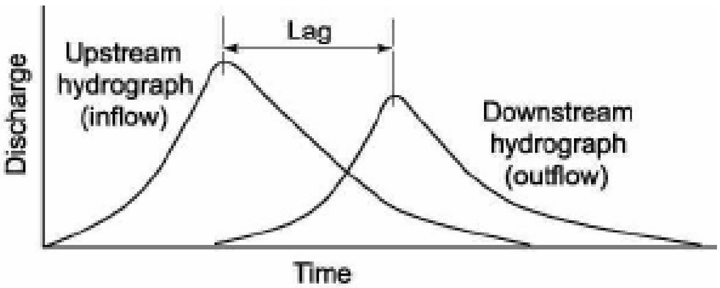


Figure 5.4.2l: Lag and Attenuation Due to Channel

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* (*Ot-1*) as follows.

$$Q_t = C_I I_{t-1} + C_2 I_t + C_3 Q_{t-1} + C_4 (q_L \Delta x) \quad \text{Equation 5.4.2f}$$

Where:

q_L	:	lateral inflow.
Δx	:	length of the channel

C_1 to C_4 : coefficients and functions of wave celerity (C),

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

5.4.3 Storage Function Method

Storage function model treats the behavior of flow of channel and flood plain as a single unit, and is most suitable when applied to the fixed type of flood flow. The storage function model, when compared with the non-uniform flow model, has an advantage that it can simulate a decrease of discharge to the flow direction by inundation.

The storage function model was derived based on the assumption that the relation between water level (H) and discharge (Q) is a single-valued function. Hence, this model cannot be applied to estuaries and confluence of rivers.

$$I - Q_l = \frac{dS_l}{dt}$$ Equation 5.4.3a

Where:

S_l : Storage Volume (m^3)

I : inflow discharge from upstream channel (m^3/s)

Q_l : Q (t + Tl)

discharge at downstream end of the channel with lag time (m3)

Tl : lag time

To express the non-linear characteristics of runoff phenomena, the storage function model introduces the following single-valued function between storage (S) of a basin or river course and the discharge (Q) from them.

$$S_l = KQ_l^{\ P}$$ Equation 5.4.3b

Where:

K and P are constants.

Flood flow analysis by storage function model is carried out using S-Q relation as shown in Figure 5.4.3, which is prepared from non-uniform flow calculations in channels and flood plains.

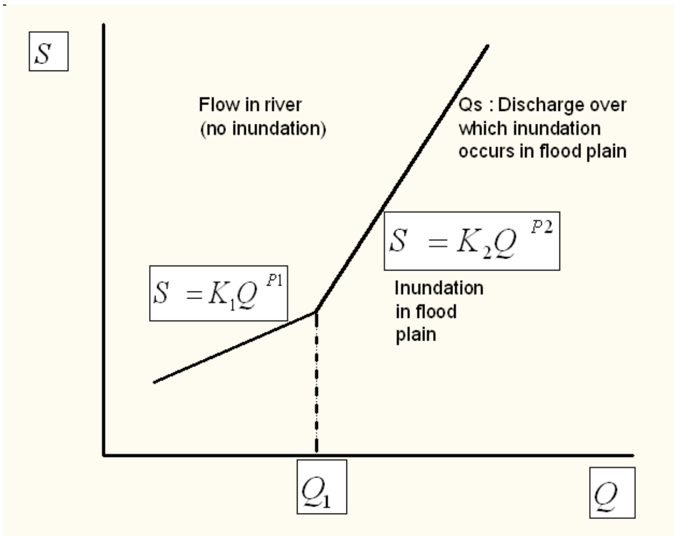


Figure 5.4.3: S-Q Relation for Flood Flow Analysis

This equation is used to substitute for the equation of motion which expresses runoff as proportional to the exponent of storage volume. In this equation, the runoff phenomena is considered to be similar to the runoff from a notch in a container filled up with water.

Runoff calculation is performed in combination with the following equation of continuity for a basin.

$$\frac{dS_1}{dt} = \frac{1}{3.6} f r_{ave} A - Q_l(t) \qquad \text{Equation 5.4.3c}$$

Where:

f	:	inflow coefficient
r_{ave}	:	average rainfall over basin (mm/h)
A	:	area of the basin (km2)
$Q_l(t)$	:	$Q(t + T_l)$ direct runoff discharge from the basin with lag time (m^3/s)
S_l	:	apparent storage volume in the basin (m^3)
T_l	:	lag time (h)

The equation of continuity for a river course is given below :

$$\frac{dS_1}{dt} = \sum_{j=1}^{n} f_j I_j - Q_l(t) \qquad \text{Equation 5.4.3d}$$

Where

f_j : inflow coefficient

I_j : inflow from basins, tributaries or the upstream end of a river course to the river course being considered (m^3/s)

$Q_l(t)$: $Q(t+T_l)$, discharge at the downstream end of river course with lag time (m^3/s)

S_l : apparent storage volume of river course (m^3)

T_l : lag time

The relationship between storage volume S and runoff Q can be obtained easily from the past flood runoff data. In general, the relationship between S and Q varies in the rising limb and recession limb of a hydrograph. The storage function model introduces the lag time T_l so as to express the S and Q relationship by a single-valued function during a flood.

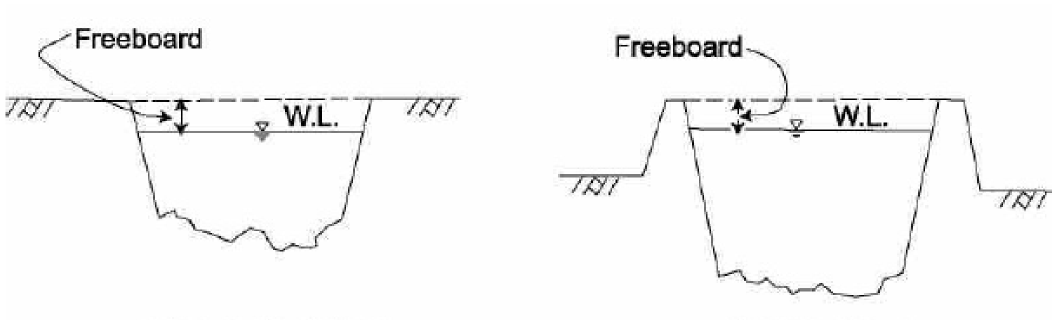
Chapter 6 FLOW CAPACITY

Flow capacity of the existing river channel should be estimated to identify the reaches with insufficient capacity in order 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 except for the channel with uniform shape and slope, for which uniform flow method is 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. Without Dike **b. With Dike**

Figure 6: Water Level for Flow Capacity Evaluation

6.1 Flow Capacity Computation

Uniform Flow Calculation and Non-uniform Flow Calculation are used in calculating the existing discharge capacity according to the types of flow and river condition. The succeeding sections discuss the principles.

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.

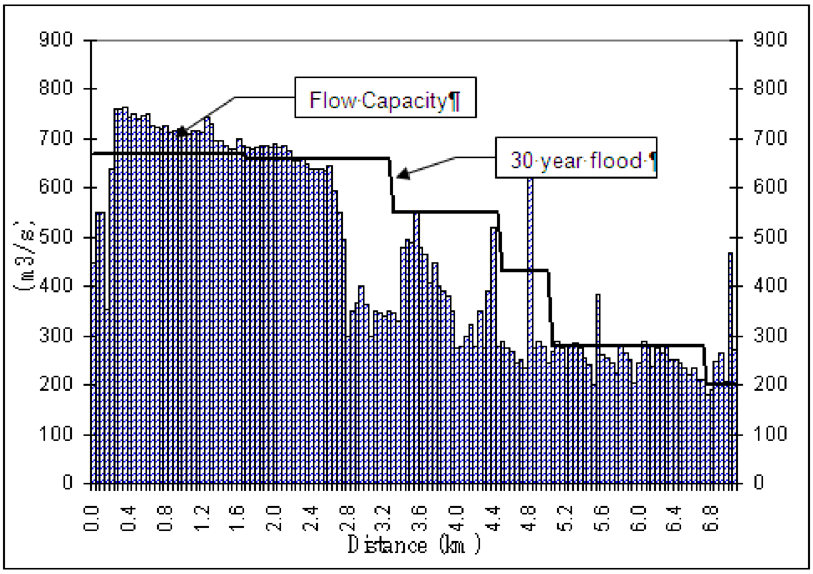


Figure 6.1: Flow Capacity Profile

6.2 Water Level Computation

After the determination of the river channel route, the alignment, longitudinal profile and cross section shall be a part in the formulation of the optimum plan by applying the water level computation using the uniform (e.g., cut-off channel or irrigation canal) or the non-uniform flow method. Water level of river channel is basically computed by non-uniform flow computation method. For rough estimate of discharge, uniform flow may be used for planning.

1) Flow Classification

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

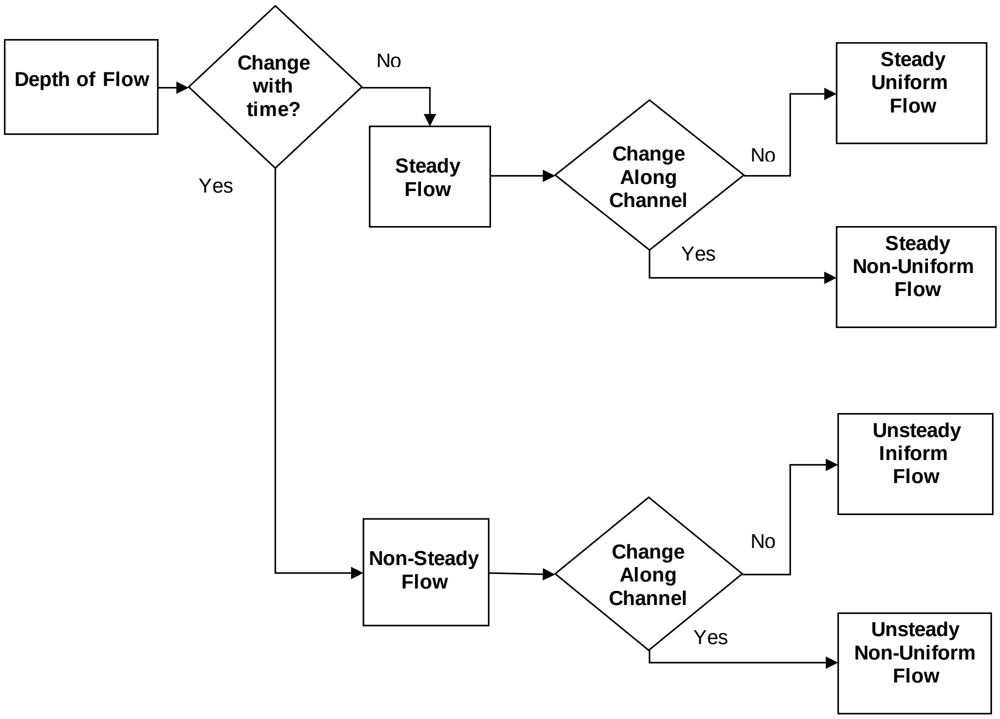


Figure 6.2: Type of Flow

Steady and Unsteady: Time is the Criterion

Steady flow exists when the depth of flow at a particular point does not change for the time interval under consideration. The flow is unsteady if depth changes with time.

Uniform and Non-Uniform : Space is the criterion.

In uniform flow, the depth and velocity of flow are the same at every section of the channel. Usually it occurs in prismatic channels. In steady uniform flow, depth and velocity is constant with both time and distance and the gravity forces are in equilibrium with resistance.

Steady non-uniform flow

Depth varies with distance but not with time. The flow may be either (a) gradually varied or (b) rapidly varies. The first requires energy and frictional resistance equations while the second requires the energy and momentum equations.

Unsteady non-uniform flow.

Depth varies with both time and space, the most common type of flow. Calculation requires energy momentum and friction equations with time. More often, this is analyzed as gradually varied 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.

6.3 Uniform Flow Calculation

Usage of Uniform Flow Computation

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

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

2) River of mild slope with uniform shape and slope along channel

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

$$S \geq n^2 g R^{-\frac{1}{3}}$$ Equation 6.3a

Where:

S	:	Channel bed slope
n	:	Manning's roughness coefficient ($m^{-2/3}/s^{-1}$)
g	:	gravitational acceleration (9.8 m/s^2)
R	:	hydraulic radius (m)

Uniform flow calculation is applicable for rivers with the following conditions:

1) When there are no points of abrupt change of riverbed gradients.

2) When there is no structures/obstruction that impedes the flow discharge.

3) When the cross sectional area of the river is almost the same longitudinally.

4) When there is relatively long straight river reach.

There are many velocity formulas, but generally, Manning's Equation, is the most commonly used as it includes river hydraulic elements.

Manning's Equation

$$V = \frac{1}{n} R^{2/3} \cdot S^{1/2} \quad (m/s)$$ Equation 6.3b

Where:

V:		Average river velocity (m/s)
R	:	Hydraulic radius (m)

Where:

R	=	A/P
P	:	wetted perimeter (m)
A	:	Average river cross-sectional area (m^2)
R	:	*h, if the river width is extremely larger than depth*

h	:	*flow depth*
S	:	Riverbed gradient
n	:	Manning's coefficient of roughness

Manning's coefficient of roughness (n) shall be determined based on the analysis of experienced floods; however, when the data of experienced floods are few or not so accurate, use the recommended values of (n) as shown in Table 6.3.

Table 6.3: Manning's Coefficient of Roughness

Channel Conditions	*Coefficient of Roughness*
General waterway	*0.030 – 0.035*
Rapid river of wide and shallow river	*0.040 – 0.050*
Temporary waterway excavated without timbering	*0.035*
Three-sided lined channel	*0.025*
River tunnel	*0.023*

Uniform Flow is described by the formula :

$$Qc = A\ V$$ Equation 6.3c

$$Q_C = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2} \quad (m^3/s)$$ Equation 6.2d

Where:

Q_C	:	Existing discharge capacity (m^3/s)
A	:	Average river cross-sectional area (m^2)

6.4 Non-uniform Flow Calculation

When the flow discharge changes with time, analysis is by unsteady flow calculations, however, most often in conjunction with non-uniform flow.

To calculate by non-uniform flow, investigate the characteristics of river sections, the location of water level controlling facilities such as weirs and groundsills, and determine whether a control section may occur at points where the riverbed gradient or section changes suddenly.

The boundary condition for non-uniform flow calculations is the water level at the downstream end (sea level at river mouth, water level from the rating curve, water level of control section, etc.) for subcritical flow and the water level at upstream section for supercritical flow.

To calculate sub critical flow water profile by non-uniform flow, the following formulas shall be used for cross -sections :

1) 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 6.4a).

When the discharge, Q, and the channel cross-section are given, the critical depth, h_c is determined from the following equations.

For rectangular Section :

$$h_c = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} \qquad \text{Equation 6.4a}$$

Where :

q	:	Q/b (m^3/s/m)
b	:	channel width (m)
g	:	acceleration of gravity (9.81 m/s^2)

For Non-Rectangular Section :

$$\frac{Q^2}{g} = \frac{A^3}{B} \qquad \text{Equation 6.4b}$$

Where :

B	:	width of water surface (m)
A	:	flow area (m2), which is a function of the critical depth,

When $F_r = \frac{V}{\sqrt{gD}} = 1$; then $D = h_c$ Equation 6.4c

Where:

Q	:	Discharge (m^3/s)
A	:	cross sectional area (m^2)
V	:	mean velocity (m/s) = Q/A,
B	:	water surface width (m).
D	:	A/B, Q = discharge (m^3/s),
g	:	acceleration of velocity (m/s/s),
F_r	:	Froude number

If F < 1, subcritical flow
F = 1, critical flow
F > 1 supercritical flow

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

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \text{Equation 6.4d}$$

Where:

n	:	Manning's roughness coefficient

R	:	hydraulic radius (m)
S	:	channel bed slope.

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

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, ho and hc 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 line

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 6.4a. 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 6.4a, 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.

a) At the normal depth *hn*, flow surface is parallel to the riverbed

b) At the critical depth *hc,* a hydraulic jump or drop occurs and flow profile is vertical

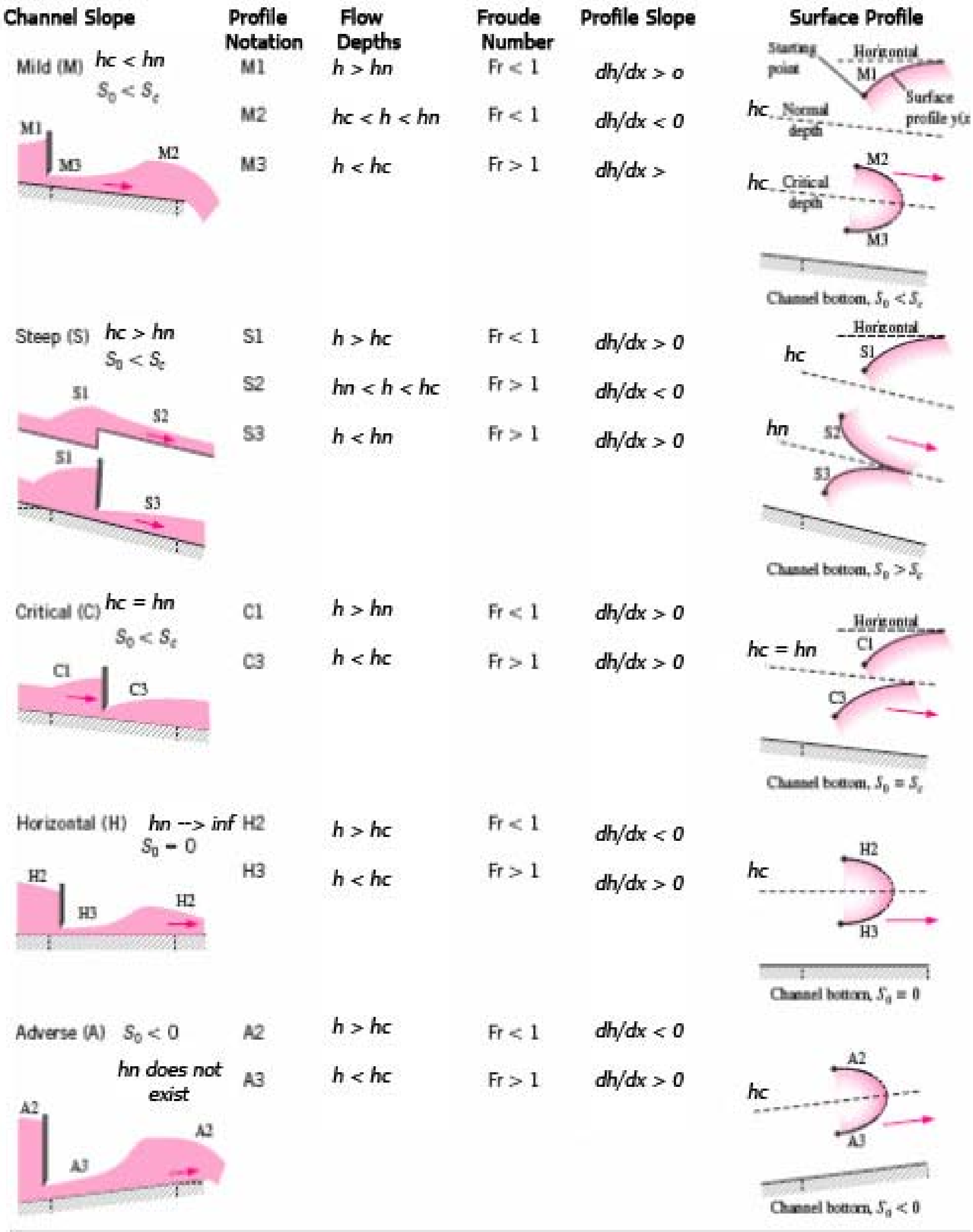


Figure 6.4a: Flow Profile

(Source: Open Channel Flow, Bryan Pearce, Department of Civil and Environmental Engineering, University of Maine, 2006)

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.

2) 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:

$$Y_2 + Z_2 + \frac{V_2^2}{2g} = Y_1 + Z_1 + \frac{V_1^{\ 2}}{2g} + h_e$$ Equation 6.4e

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.

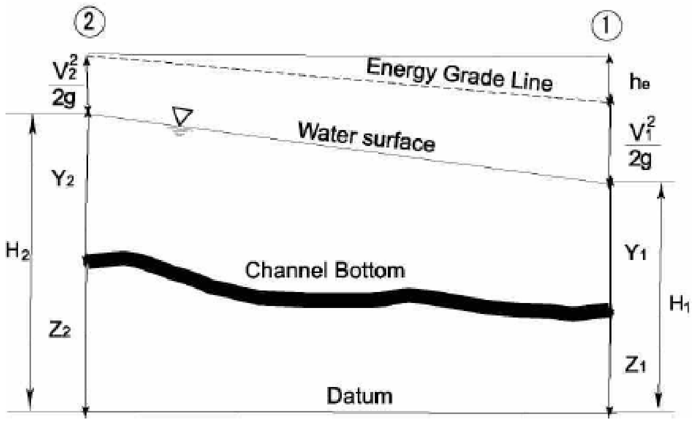


Figure 6.4b: in the Energy Equation

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

$$h_e = L\overline{S_f}$$ Equation 6.4f

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:

$$S_f = \left(\frac{Q}{K}\right)^2$$ Equation 64g

Where:

Q : discharge (m^3/s)

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

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

i Average Friction Slope Equation

$$\overline{S_f} = \frac{S_{f1} + S_{f2}}{2}$$ Equation 6.4h

ii Average Convergence Equation

$$\overline{S_f} = \left(\frac{Q_1 + Q_2}{K_1 + K_2} \right)^2$$ Equation 6.4i

Energy equation:

$$he = \left[H_2 + \frac{1}{2g} \left(\frac{Q_2}{A_2} \right)^2 \right] - \left[H_2 + \frac{1}{2g} \left(\frac{Q_1}{A_1} \right)^2 \right]$$ Equation 6.4j

Energy loss:

$$he = \frac{1}{2} \left[\frac{(n_1^2 Q_1^2)}{\left(A_1^2 R_1^{4/3} \right)} + \frac{(n_2^2 Q_2^2)}{\left(A_2^2 R_2^{4/3} \right)} \right] \Delta x$$ he = 1/2 (n

Equation 6.4k

Where:

He : energy head loss

subscript 1 is for the known hydraulic quantity of downstream sections, subscript 2 is for the unknown hydraulic quantity at upstream section, and Q_2 and n_2 are known.

Various methods of numerical solutions are available for non-uniform flow. The standard successive calculation method can be the most convenient in case of changes between river sections, where discharges vary. Nowadays numerical calculations of non-uniform flow are performed using computer software.

Chapter 7 DESIGN DISCHARGE DISTRIBUTION AND FLOOD CONTROL MEASURES

7.1 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 along the river reaches is then allocated for the various flood control measures and their corresponding costs determined and evaluated for the most optimum plan.

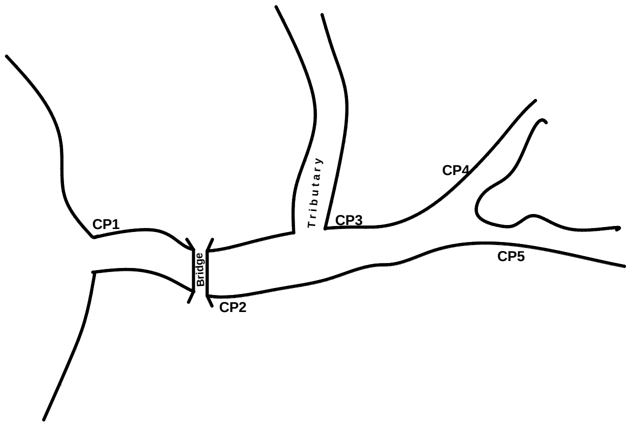


Figure 7.1: Example of Control Points

7.1.1 Flood Control Measures

There are various river engineering works, either individually or in combination, which provide flood protection and reduce flood damages along river reaches. These measures result to design discharge allocation appropriate for the specific purpose as listed below.

Table 7.1.1: Flood Control Measures

No	Category	Facility/Measure
1	Increase of river flow capacity	- Dike/Levee - Widening of waterway/river - Dredging/Excavation - Combination of above - Cut-off channel
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 - Pilot channel (small channel)
4	Prevention of riverbed degradation	- Groundsill - Regulated quarrying
5	Prevention of riverbed aggradation and, obstruction/interruption against river flow	- Sabo works (for sediment control) - Regular maintenance (channel excavation/dredging) - Vegetation/Reforestation - Jetties / estuary training dikes

As described in the above table, Items 1 and 2 are direct measures related to the design safety level against flood. Items 3 to 5 are appurtenances to maintain and accommodate the estimated design discharge.

In flood control planning, the design discharge is allocated to the facilities/measures of items.1 and .2. 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) Item 1

One of the functions of river improvement is to increase the flow capacity of the existing river channel, which may include widening, dredging/excavation, and dike construction as illustrated below.

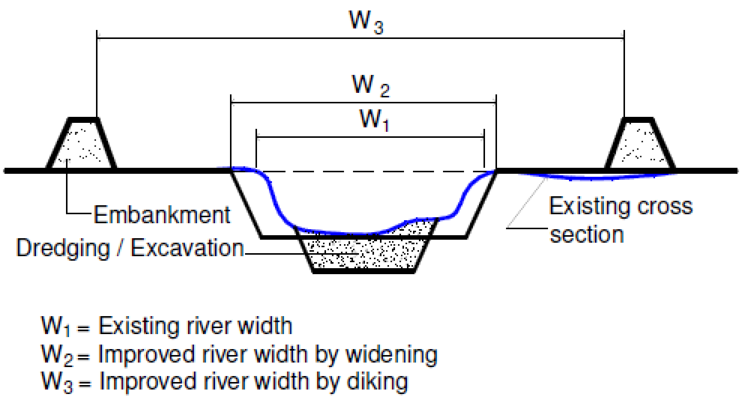


Figure 7.1.1a: River Improvement

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) Item 2

a) 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.

b) Retarding Basin

A retarding basin has the same function as a dam (Figure 7.1.1b). In general, a retarding basin is generally constructed in the middle reach to reduce flood discharge at the downstream reaches.

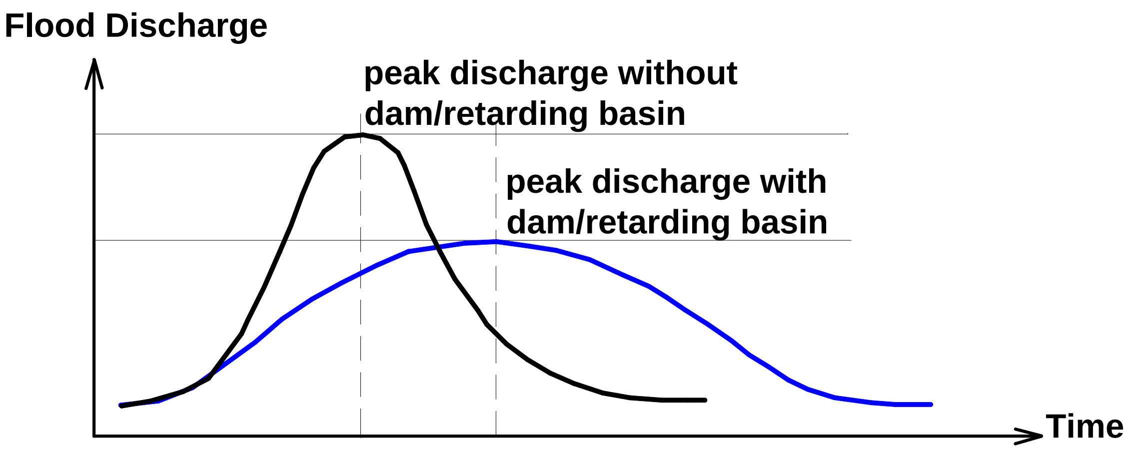


Figure 7.1.1b: Reduction of Flood Peak Discharge by Dam/Retarding Basin at the Downstream Central Point

c) Floodway

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

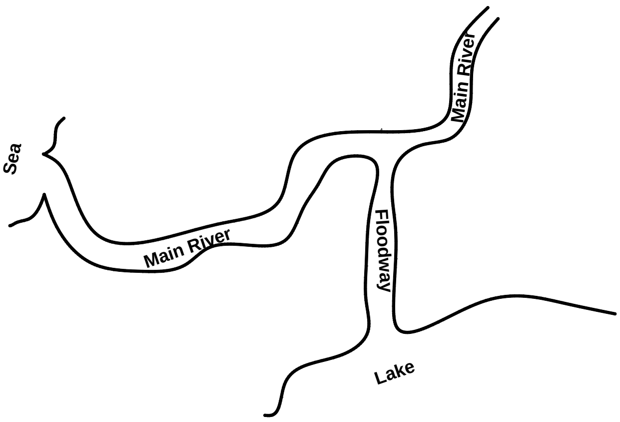


Figure 7.1.1c: Floodway

7.1.2 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 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 discharge to the river channel by increasing the flow capacity in consideration of the existing flow capacity and the land use of riverine area
2) If the discharge cannot be accommodated in Item 1, allocate the design discharge to alternative flood control facilities of item 2 such as ; dam, retarding basin, and floodway.
3) Determine the appropriate flood control facilities to accommodate the allocated discharge
4) Estimate the project cost based on the preliminary design of the flood control facilities
5) Estimate the benefit to be accrued after the project implementation
6) Calculate the cost/benefit ratio (Refer Chapter 9).

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 7.1.2: 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	1,500	5,000
3	2,000	1,000	2,000	5,000
4	2,000	0	3,000	5,000

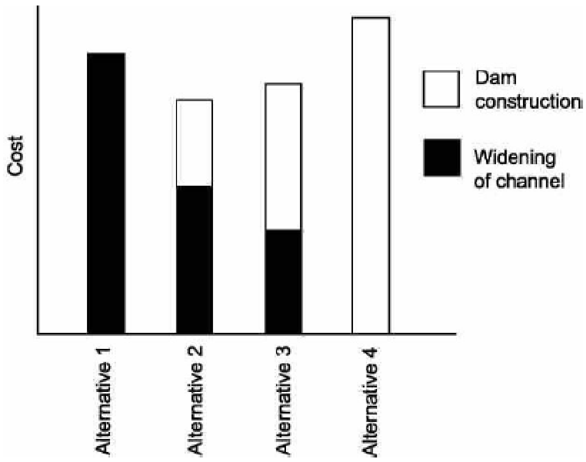


Figure 7.1.2: Comparative Cost Analyses of Alternatives (Example)

Alternative 1 applies widening of channel only; Alternative 2 introduces dam construction and widening of channel; Alternative 3 uses the same combination as alternative 2 but leans more on dam construction; and, Alternative 4 is purely dam construction. Alternative 4 gives the highest cost, followed by Alternative 1, then Alternative 3 and Alternative 2.

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

7.2 Modeling of Discharge Allocation to Flood Control Facilities/Measures

7.2.1 Runoff Model for Alternatives

The same runoff model used to obtain the design/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 7.2.1a is a runoff basin model for SCS unit hydrograph in HEC-HMS window to obtain the design discharge, while Figure 7.2.1b 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.

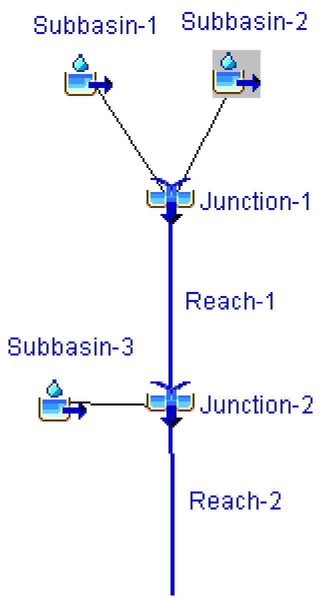


Figure 7.2.1a: Model for Design Discharge

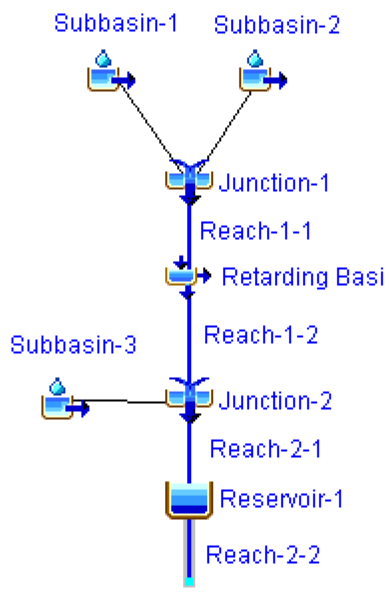


Figure 7.2.1b Model for Allocation of Design Discharge

Legend

Sub-basin – represents physical watershed

Junction – combines streamflow from upstream elements

Reach – conveys streamflow in a basin model

Retarding basin or Diversion temporary storage and mean for streamflow leaving the main channel

Reservoir – uses to detain or attenuate hydrograph by reservoir or pond.

Note : SCS – Soil Conservation Services Unit Hydrograph Method
HEC-HMS – Hydrological Engineering Center – Hydrological Modeling System of the US Army Corps of Engineer

7.2.2 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 7.2.2a illustrates typical hydrographs or attenuation type of flood discharge method.

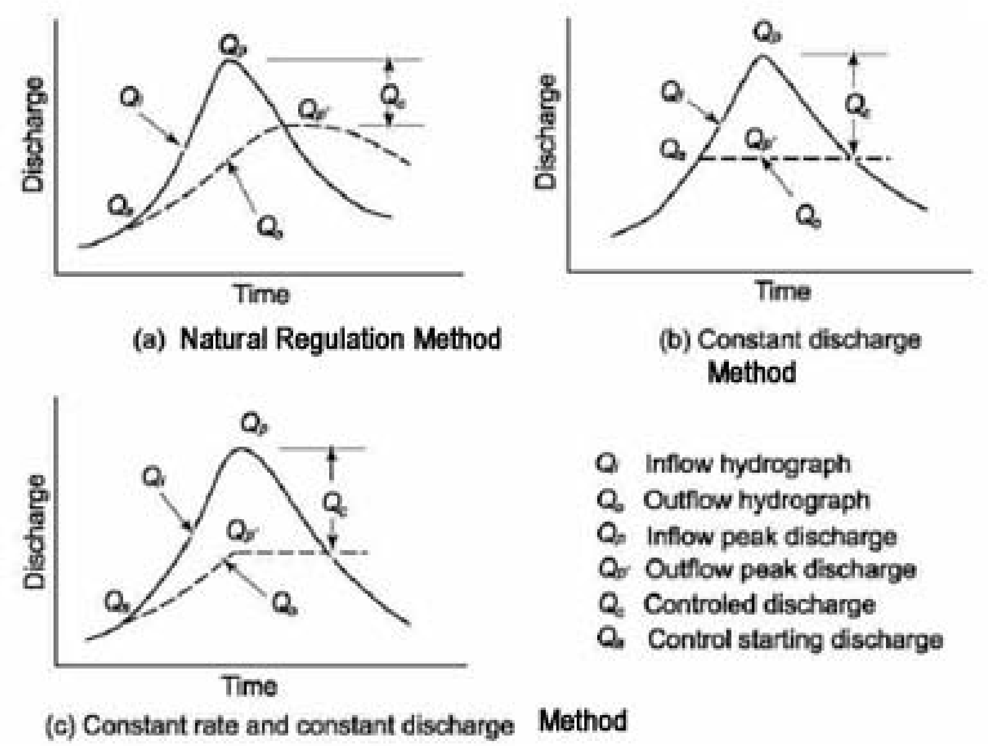


Figure 7.2.2a: Typical Discharging by Dam

From the figure above, the natural regulation method is usually used for small scale dams or in the river basin where flood discharge duration is considerably small. It does not require so much of human intervention, hence, management is easy. The outlet of this kind of dam may consist of outlet pipe (culvert) and emergency spillway (Figure 7.2.2b).

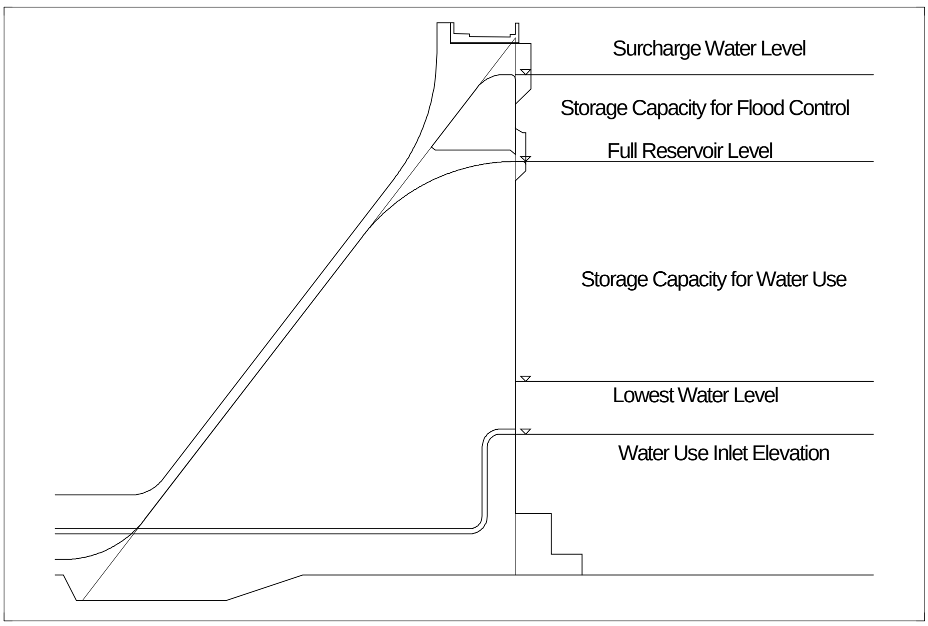


Figure 7.2.2b: Simple Outflow Structure of Dam

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

$$O = KA\sqrt{2gH} \qquad \text{Equation 7.2.2a}$$

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

$$O = CLH^{1.5} \qquad \text{Equation 7.2.2b}$$

Where:

O	:	outflow discharge (m3/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. Outflow from impoundment that has a horizontal water surface can be computed by model shown below. The model has discrete equal intervals of duration Δt. Then it solves recursively the following one-dimensional approximation of the continuity equation.

$$I_{ave} - O_{ave} = \frac{\Delta S}{\Delta t} \qquad \text{Equation 7.2.2c}$$

Where :

I_{ave}	:	average inflow during time interval
Oave	:	average outflow during time interval
ΔS	:	storage change during time interval.

With a finite difference approximation, this can be written as:

$$\frac{I_t + I_{t+1}}{2} - \frac{O_t + O_{t+1}}{2} = \frac{S_{t+1} - S_t}{\Delta t} \qquad \text{Equation 7.2.2d}$$

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:

$$\left(\frac{2S_{t+1}}{\Delta t}+O_{t+1}\right)=\left(I_t+I_{t+1}\right)+\left(\frac{2S_t}{\Delta t}-O_t\right)$$ Equation 7.2.2e

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.

For an impoundment, storage and outflow are related. This relationship corresponds to the values of O_{t+1} and $S_{t+1.}$ The computations can be repeated for successive intervals, yielding values of O_{tn}, O_{t+2}, - - - O_{t+n}, which are the required outflow ordinates of the hydrograph.
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 an inflow-diversion relationship. Figure 7.2.2c shows outflow is equal to inflow minus the volume diverted to the retarding basin. When inflow reaches storage stage of the retarding basin, outflow is kept constant. The volume in the retarding basin is computed as the sum of the diverted discharge.

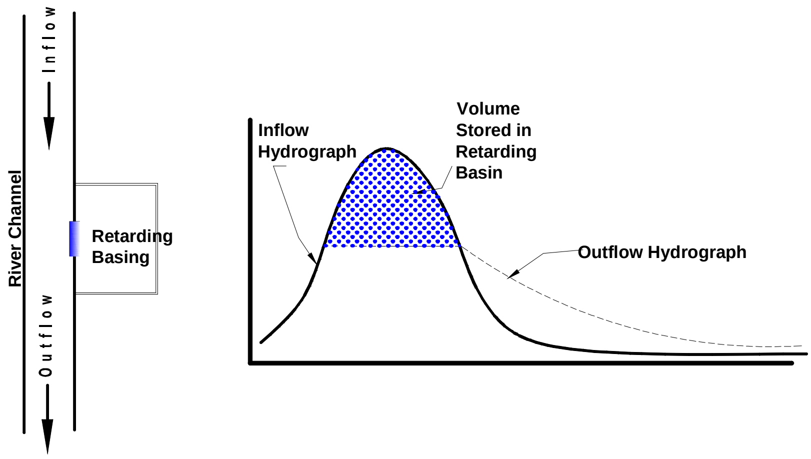


Figure 7.2.2c: Flood Control by Retarding Basin

3) Floodway
The diverted discharge and outflow discharge can be computed in similar principle as the above.

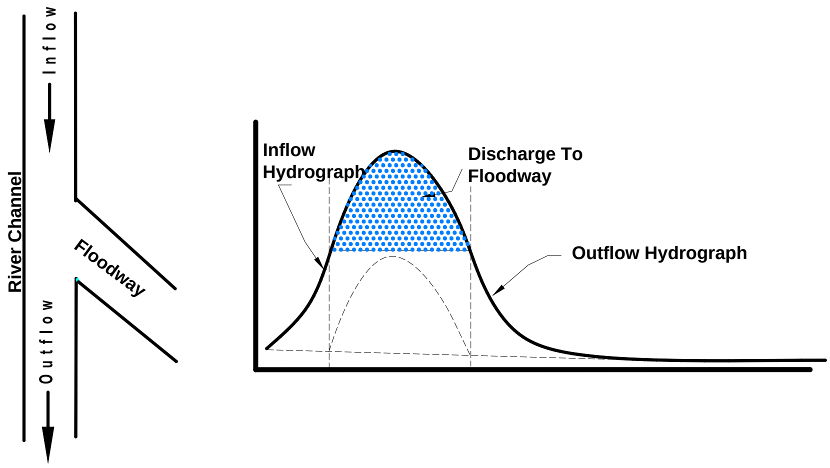


Figure 7.2.2d: Flood Control by Floodway

7.3 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 7.3. indicates design discharge distribution for the Ilo-ilo Flood Control Project.

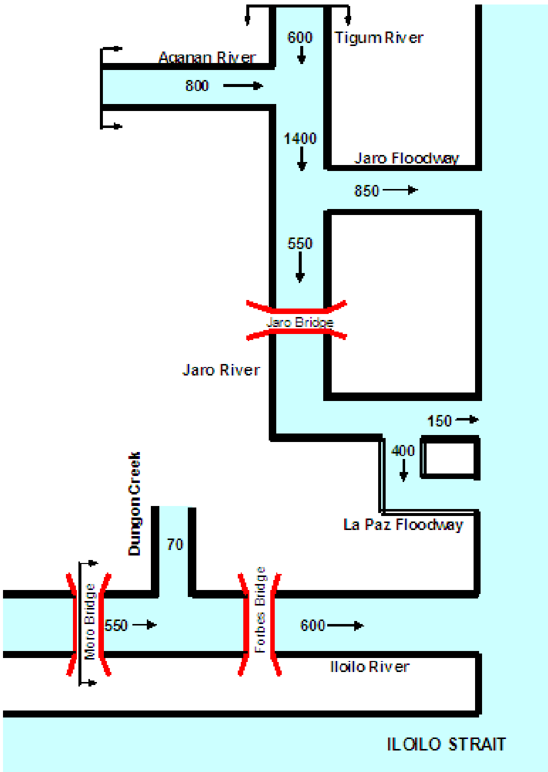


Figure 7.3: Design Discharge Distribution for the Ilo-ilo Flood Control Projects
50 year return period
Source : DPWH Ilo-ilo Flood Control Project Progress Report

Remarks:
The previous mentioned calculated variable discharge can be used in non-structural measures, such as hazard mapping, evacuation and warning codes .

Chapter 8 RIVER CHANNEL IMPROVEMENT

8.1 Goal of River Channel Improvement

Rivers generally originate from the mountains, then flow along the plains and finally discharge to the lakes, seas, oceans or other bodies of water. They form defined channels; drain the land water due to rainfall; and discharge water into the sea. The rivers not only carry water but also sediments from the catchment area and eroded from the beds and the banks.

Flood water discharges to sea, ocean or lakes, except when some portions evaporate or are diverted for other use. Sediments often do not reach the sea, but gradually deposit on the riverbed, which causes riverbed aggradation which reduce flood discharge capacity and eventually aggravates damages due to floods.

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 erosion such as bank erosion/collapse including dike and revetment and river bed aggradation and degradation.

In the Philippines, a lot of river structures are damaged during floods and inundations in the riverine areas and yet substantial budgets are allotted 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 studies 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 studies of river characteristics to attain the following:

1) To carry design flood discharge, which is allocated to the target river channel
2) To protect the target river channel from scouring and/or erosion based on the design flood discharge

8.2 River and Segment

8.2.1 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, which can 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 riverbed gradient.

“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 8.2.1 shows the classification of river system into several segments.

Generally, longitudinal profile of the river becomes gentle from upstream to downstream. However, the longitudinal profile and the size of the riverbed materials change abruptly in a certain point such as the boundary of mountainous areas and alluvial fans. The riverbed materials, such as gravel is scarce while rough sand is dominant 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 high flow velocity, which is relative to the river alignment, longitudinal and cross sectional profiles and types of riverbed materials, where one of the countermeasures is changing/adjusting the riverbed gradient by using groundsill. Thus, in the river improvement planning, the classification of each river segment should be recognized and not ignored.

Table 8.2.1: Classification of River Segment and its Characteristics

CLASSIFICATION	SEGMENT M	SEGMENT 1	SEGMENT 2		SEGMENT 3
			2-1	2-2	
Geography	Mountain	Alluvial Fan	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.
Riverside Materials	Rocks appear on riverside as well as on riverbed.	Riverside materials are same as the riverbed. Thin layer of sand and silt sometimes covers the surface.	Lower layer of the riverside material is the same with the riverbed. Mixture of fine sand, clay and silt.		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: Alluvial River Engineering, Dr Koichi Yamamoto, Japan, Sept 1994

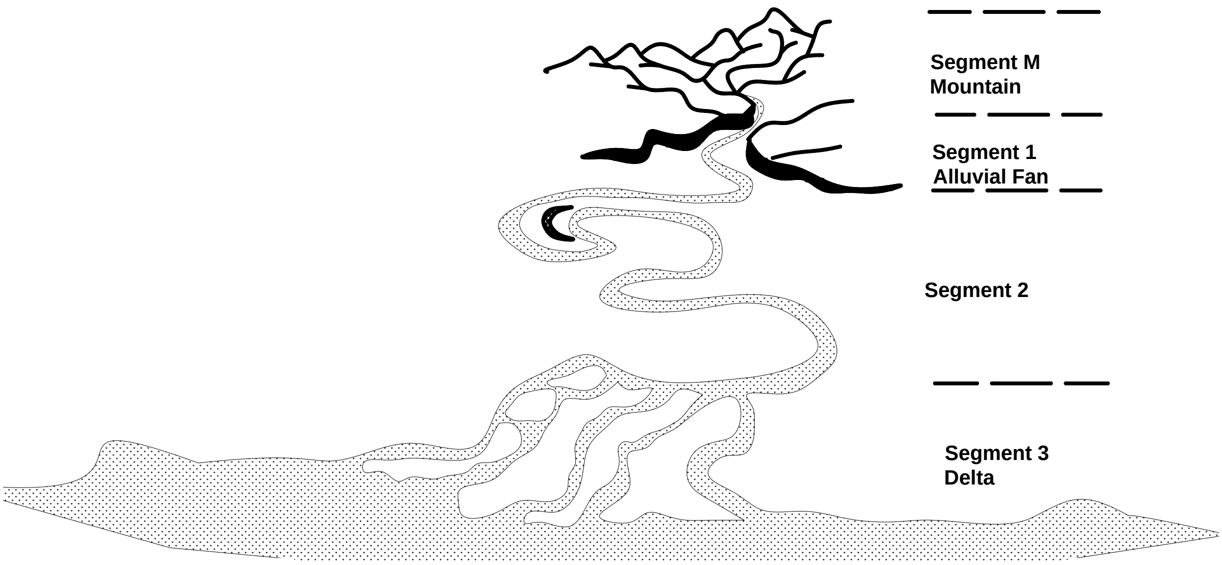


Figure 8.2.1: River System Showing Segments

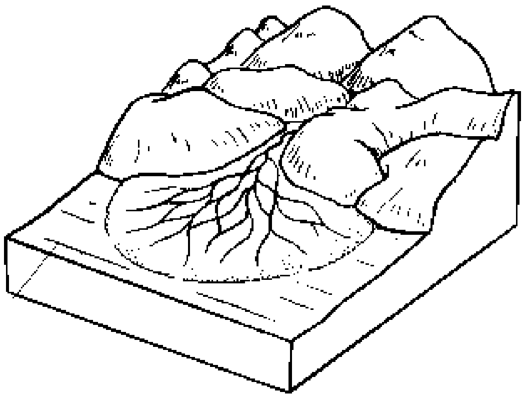


Figure 8.2.1a: Segment M and Segment 1

Source: http://www.sjgs.com/groundwater/alluvial.gif

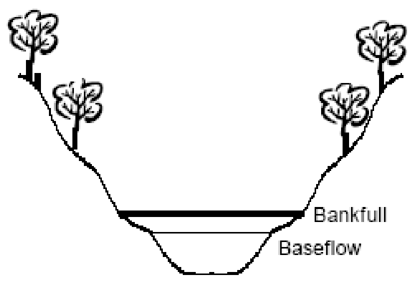


Figure 8.2.1b: Segment M

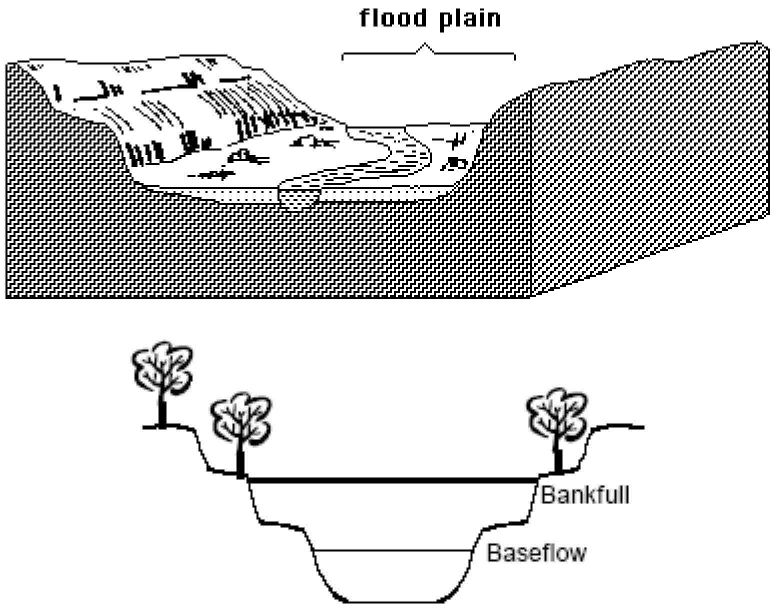


Figure 8.2.1c: Segment 2-1

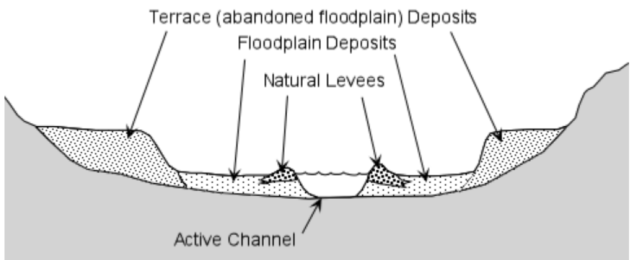


Figure 8.2.1d: Segment 2-2

Source : River Systems and Causes of Floodings, Prof. Stephen A. Nelson. Tulane University

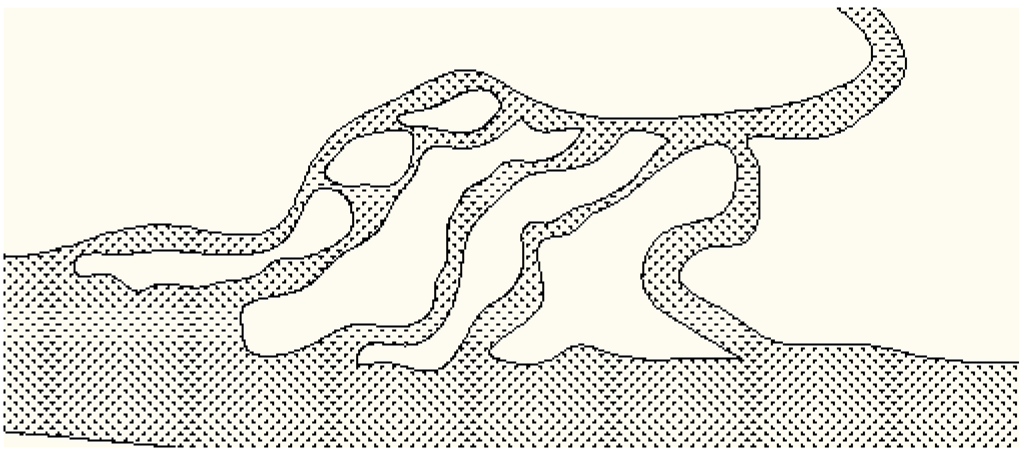


Figure 8.2.1e: Segment 3 Delta

8.2.2 River Segment Characteristics

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 conditions with sediment deposition.

Therefore, the river characteristics of the target stretches for river improvement should be roughly assessed/classified according to the particular river segment to analyze the phenomena related to the sediment movement, to select the optimum planning/design for the river improvement and consequently to minimize the initial/maintenance cost. Hereunder are 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. River characteristics are summarized below

   a) Sediment supply to river course is largely categorized into three.

      i. Gravel larger than1 cm is commonly river bed materials for segment 1 and segment 2.

      ii. Course and medium sand are usually found in Segment 2-2

      iii. Fine sand and silt are commonly in Segment 3.

   b) Gravel is not carried to segment 2-2, but predominantly deposited between Segment 1 and segment 2-2.

   c) Sand does not usually deposit in segment 1 and segment 2-1 due to high tractive force. Eventually, sand settles at segment 2-2 from mountain area

   d) There is high tractive force acting on the sand, silt and clay along segments 1, 2-1, and 2-2. Hence, these materials are not found on the riverbed but on the flood plain deposits along these segments.

   e) There are rivers where segment 3 is not found, where a large amount of materials smaller than fine sand are flushed out into the sea.

2) Bed Forms

   For rivers flowing in alluvium, 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 8.2.2a). Both small and medium scale sand waves can exist under the same hydraulic conditions.

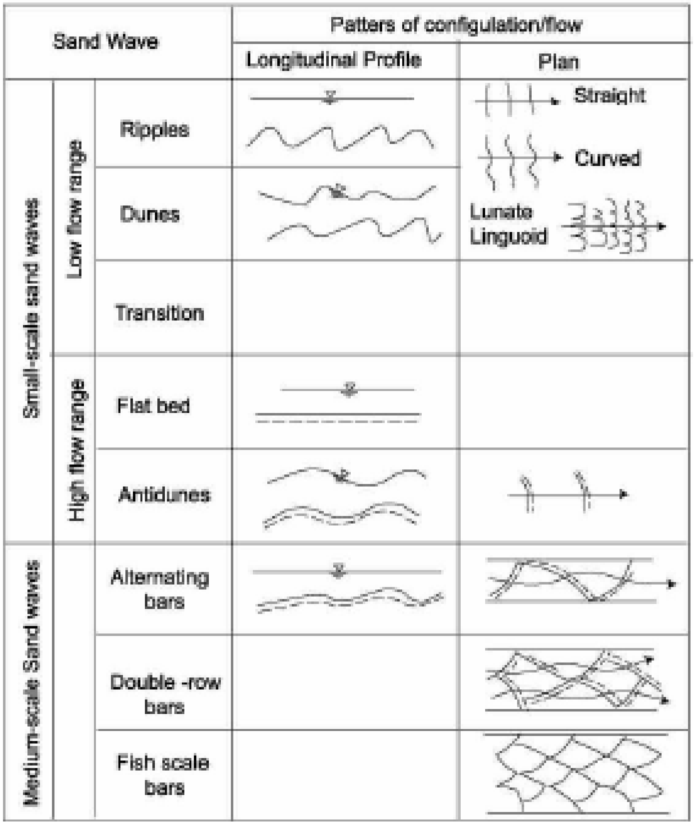


Figure 8.2.2a: Type of Bed Forms

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 Bed Forms

i. Small Scale Bed Forms

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.

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

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

<u>Transition</u>: Undeveloped ripples and dunes exist on flat bed.

<u>Flat bed / plain bed</u>: Flat bed with high sediment transport.

Small-scale bed forms change from ripples (Fr <<1), dunes (Fr<1), transition

(Fr<1) and flat bed (Fr <1 and d ≤ 0.4 mm)corresponding to water depth and velocity during a flood. Further, it changes to antidune (Fr >> 1) when velocity increases.

$$F_r = \frac{V}{\sqrt{gd}}$$ Equation 8.2.2a

Where :

Fr	:	Particle Froude Number
V	:	sediment transport velocity (m/s)
g	:	gravity (9.80 m/sec2)
d	:	particle size

ii Medium-scale Bed Forms (Sand Bars)

Width depth ratio (B/H) is a dominant parameter of sand bars (Figure 8.2.2b). Scale of sand bars is shown in Figures 8.2.2c and 8.2.2d.

H = water depth
L_s = Length of sand bar
B = river width
B_s = width of sand bar

Alternate bars: B < 100 (70 to 140) H

L_s =5 to 15B

Multiple bars: B> 100 (70 to 140)

L_s= 2 to 6B, Bs =100(70 to 140)H (Double-row bar)

L_s = 1.5 to 4B_s, B_s =100(70 to140)H (Fish-scale bar)

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

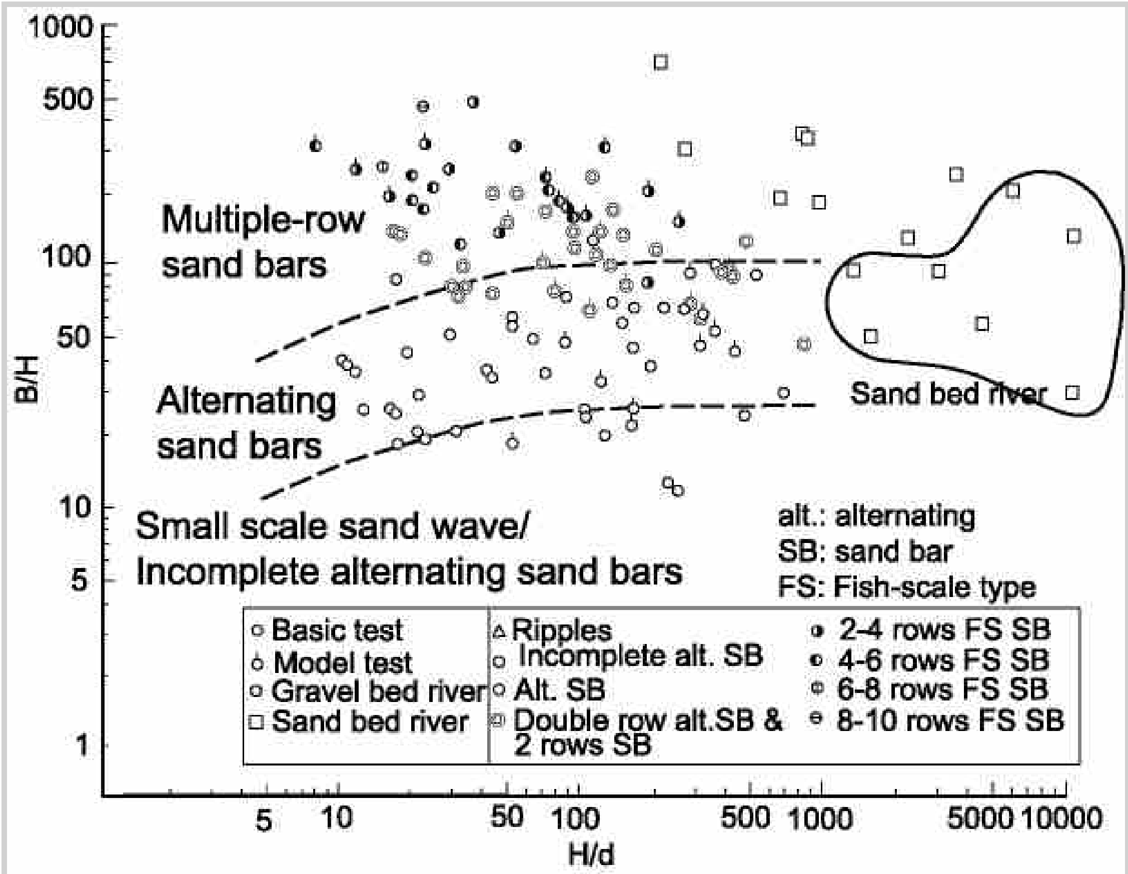


Figure 8.2.2b: Sand Bar Occurrence

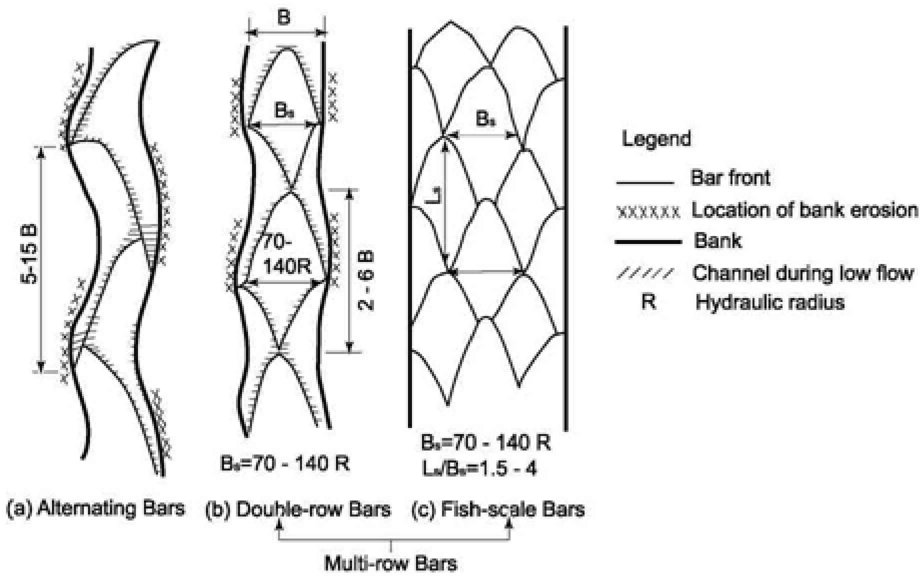


Figure 8.2.2c: Scale of Typical Sand Bars

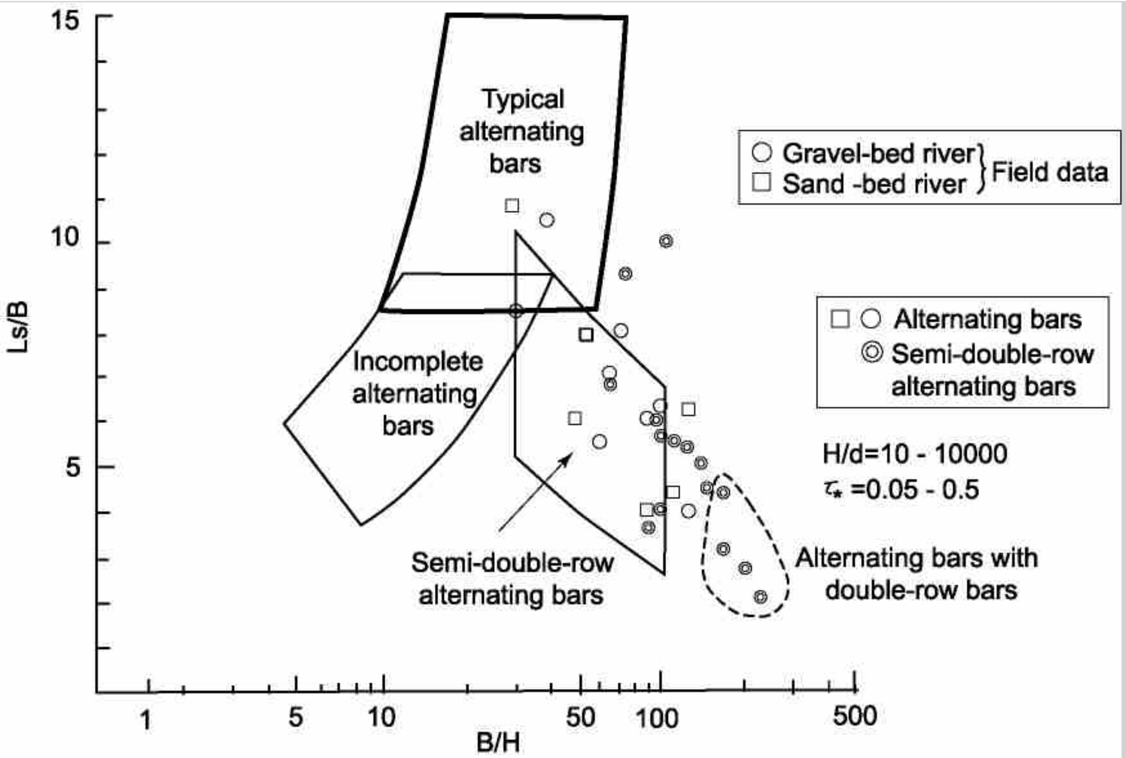


Figure 8.2.2d: Scale of Sand Bars and River Channel

b) Characteristic of Sand Bars

When the ratio of water width (B) to water depth (Hm) of mean annual maximum discharge exceeds 10 (B/Hm ≥ 10), sand bars are generally formed (Figure 8.2.2b).

When B/Hm is greater than 100 (70 to 140) , multi-row sand bars are produced. (Refer to Figure 8.2.2c).

i. 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 downstream while the bank directly hit by the flow moves along at the same time, thus restraining meandering. Consequently, straight river channel is generally formed.

ii Segment 2

When alternating sand bars are produced, flood flow drifts towards banks and causes scouring at the bank, which is directly hit by the flow at the front edges of sand bars, 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 dominant.

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 Qm (bankfull discharge), such as river width (B), cross-sectional area (A), mean water depth (Hm) and mean flow velocity (Vm), can be estimated from three (3) parameters; namely, Qm, bed slope (*Sb)* and diameter of typical riverbed materials (dr).

Figure 8.2.2e shows the relationships between water depth (Hm) of the low-water channel at the time of mean annual maximum discharge, the diameter of typical riverbed materials (dr) and different bed slope (Sb), based on the river data in Japan.

Figure 8.2.2f indicates the average flow velocity (Vm) in a low-water channel corresponding to the mean annual maximum discharge.

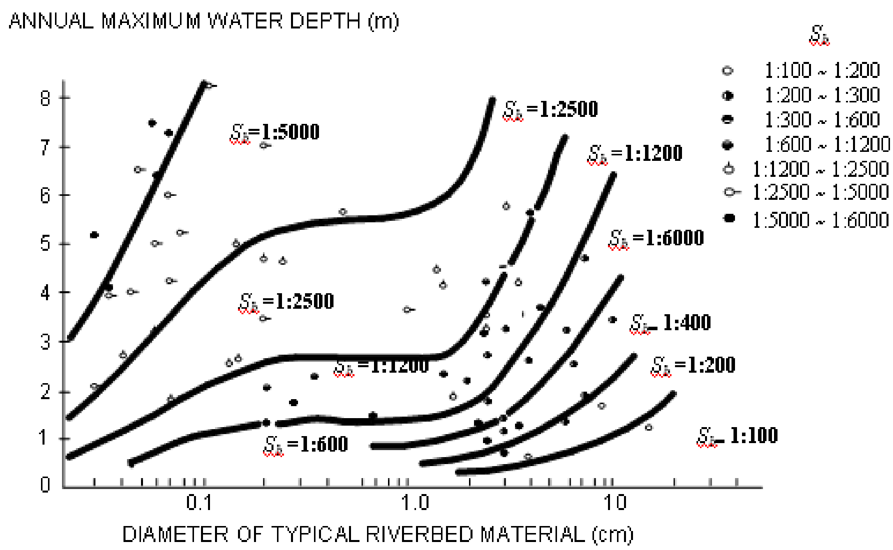


Figure 8.2.2e: Relationship with the Diameter of Riverbed Material and Annual Maximum Water Depth

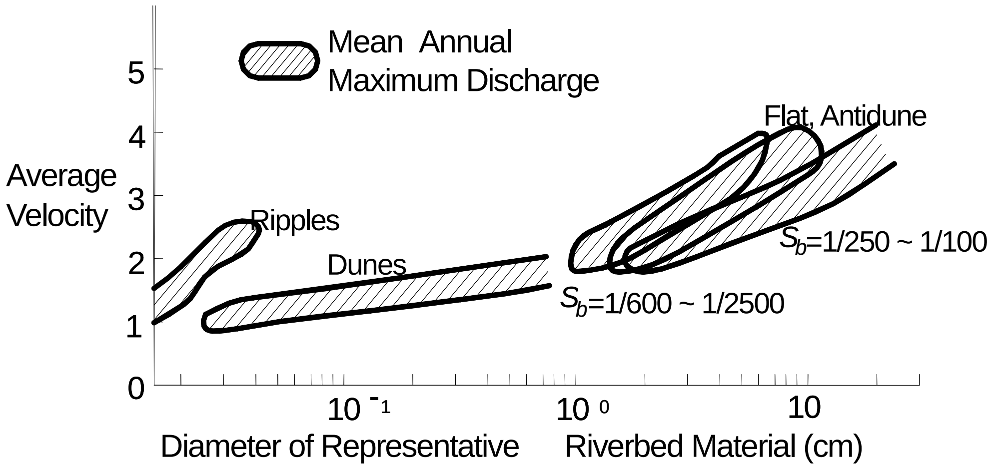


Figure 8.2.2f: Relationship with the Diameter of Riverbed Material and Mean Velocity

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 8.2.2: Type and Process of Bank Erosion

Type of bank erosion	River Segment	Processes	Size of Erosion	Typical Bank Material	Remarks
Simple erosion	Segment 1	River bed erosion and direct flow 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
Slip type erosion	Segment 2 Segment 3	Shear resistance reduction due to increase of water content of bank material during floods	Erosion width is wider than collapse type erosion	Sand Silt Clay	This erosion is rare
Collapse type erosion	Segment 2 Segment 3	Scouring of bank slope or flowing out of fine sand layer resulting in collapse	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

Source : Alluvial River Engineering, Dr. Koichi Yamamoto, Japan, Sept 1994

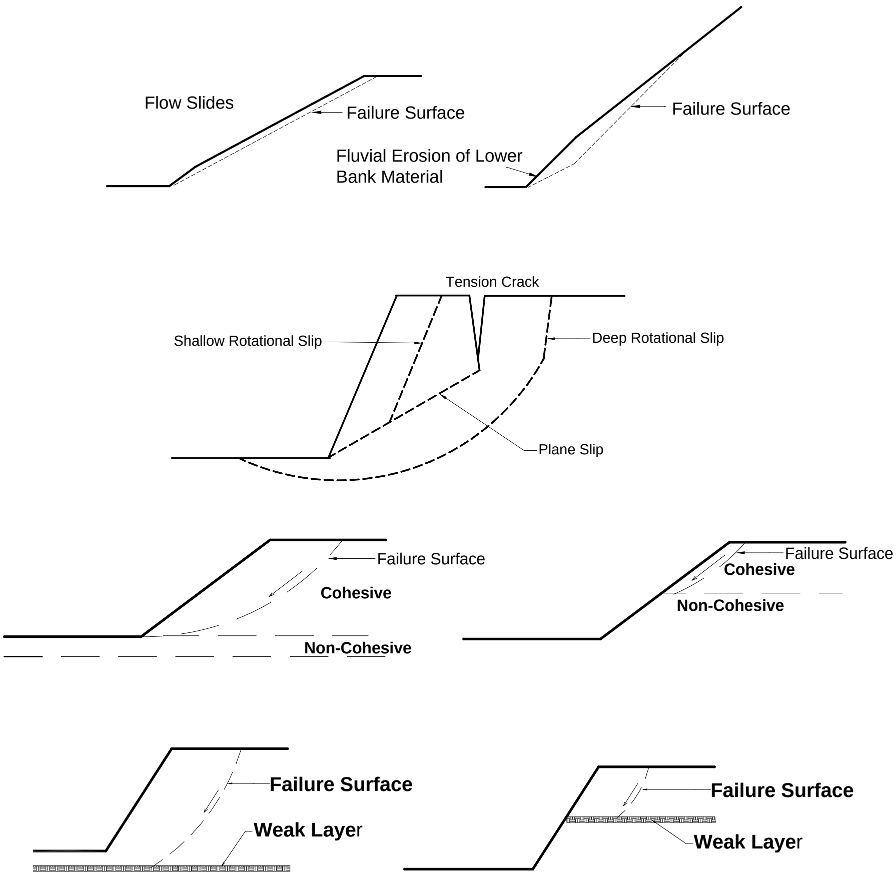


Figure 8.2.2h: Slip Type

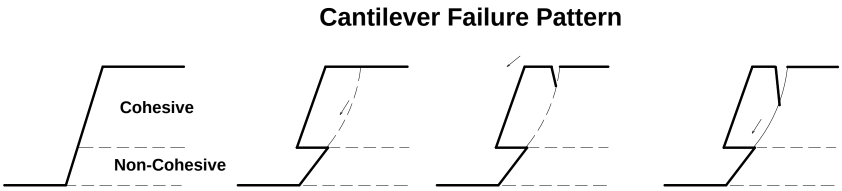


Figure 8.2.2i: Collapse Type

Source : River Engineering for Highway Encroachment, Highways in the River Environment, Federal Highway Administration, US Department of Transportation, December 2001

b. Causes of River Bed Erosion

Simple erosion and collapse type erosion are sometimes 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) variable river width 2) curved or meandering channel.

Variable River Width

Assuming $q_s = k \cdot u_*^p$ Equation 8.2.2b

Where:

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

$$u_* = \sqrt{gRS}$$ Equation 8.2.2c

Where,

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

Mean depth *Hm*, surface width *B*, wetted area *A* and channel slope *S* of the narrow channel is estimated from *Ho, Bo, Ao* and *o* (suffix for upper section).

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

$H_m/H_o = (B/B_o)^{-4/5 \text{ to} -6/7}$ $A/A_0 = (B/B_0)^{1/5 \text{ to } 1/7}$ $S/S_0 = (B/B_0)^{2/5 \text{ to } 4/7}$

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

$H_m/H_o = (B/B_o)^{-2/3}$ $A/A_o = (B/B_o)^{1/3}$ $S/S_0 = (B/B_o)^0$

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

$H_m/H_o = (B/B_o)^{-3/4}$ $A/A_o = (B/B_o)^{1/4}$ $S/S_0 = (B/B_o)^{1/4}$

Figure 8.2.2i indicates r**iver channel scouring at variable width** for p =3 and p = 4.

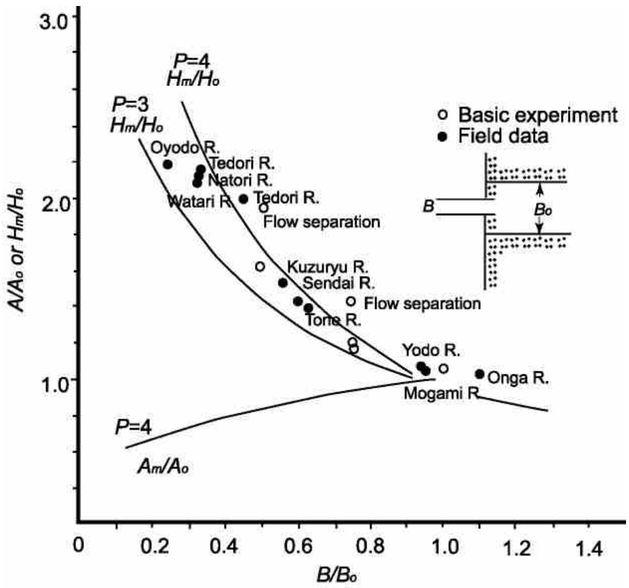


Figure 8.2.2i River Channel Scouring at Variable Width

Source: page 202, Alluvial River Theory, Koichi Yamamoto, 1994

Where :

Ho	:	depth at the upstream of the river mouth (m)
Hm	:	mean depth (m)
A	:	wetted area at the river mouth (m^2)
Ao	:	wetted area at the upstream of the river mouth (m^2)
B	:	river mouth width (m)
Bo	:	width at the upstream of river mouth (m)
S	:	river mouth slope
So	:	slope at the upstream of river mouth

Curved or Meandering 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 8.2.2k

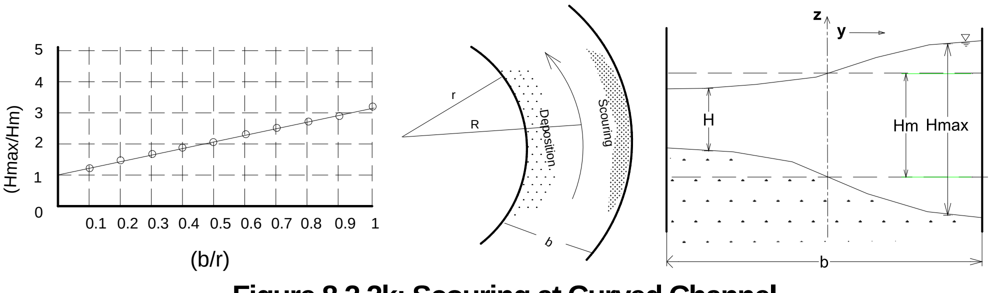


Figure 8.2.2k: Scouring at Curved Channel

The graph shows that as r becomes bigger, the curve becomes milder, the ratio of b/r becomes smaller and the ratio of Hmax/Hm approaches one, conversely, as r becomes smaller, the curve becomes sharper, the ratio of b/r approaches one, and the ratio of Hmax/Hm becomes bigger.

iii Sand Bars (Medium-scale Bed Forms)

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 (Hs) of the gravel-bed river is roughly estimated from 1) width of waterway (B) and 2) mean water depth (Hm) of mean annual maximum discharge and 3) representative grain size particle (dr).

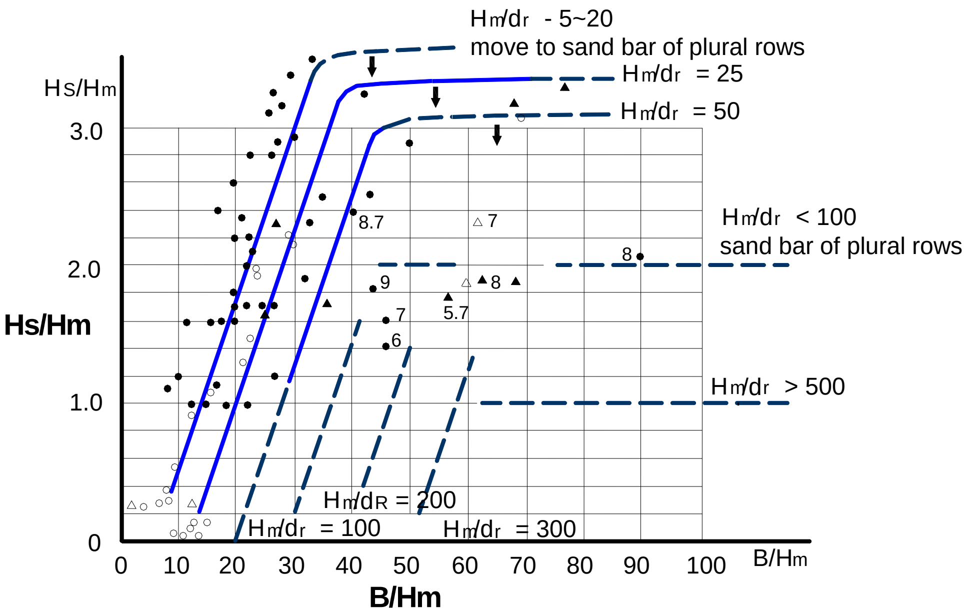


Figure 8.2.2l: B/Hm and Hs/Hm Relationship

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 pier or abutment and the spur dike.

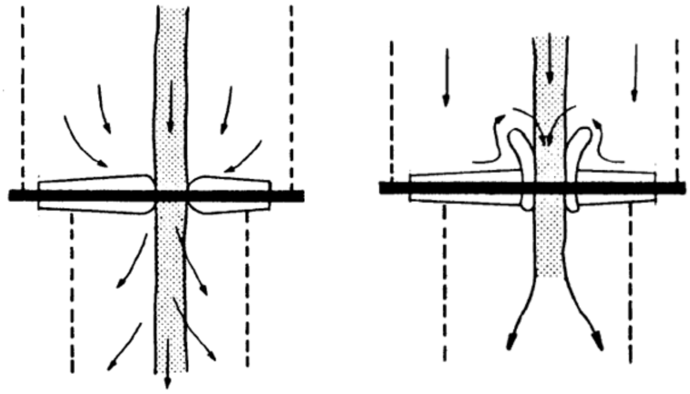


Figure 8.2.2m: Scouring at Bridge Abutment

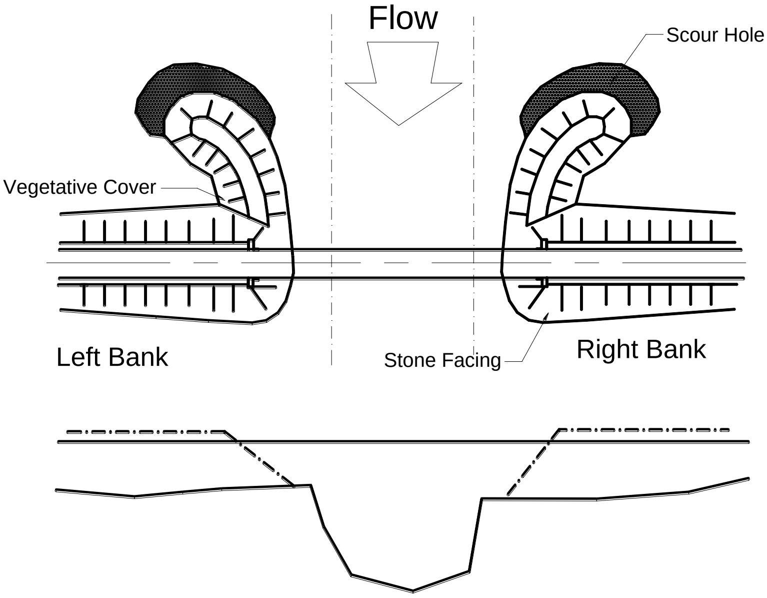


Figure 8.2.2n: Typical Guidebank

5) Response to River Channel Improvement

a) Change of Profiles

i Dredging with steeper slope

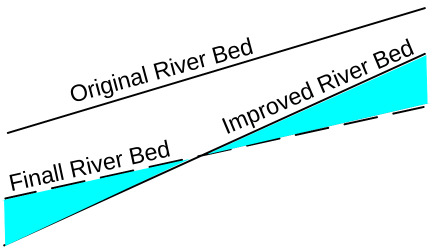


Figure 8.2.2o: Dredging with Steeper Slope

When dredging of channel is undertaken with steeper bed slope than the existing one, the bed slope tends to return to the original slope, when it reaches a state of equilibrium through sedimentation.

Ii Local Dredging

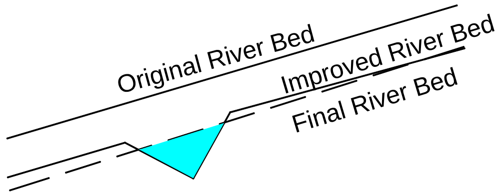


Figure 8.2.2p: River Bed Change after Dredging

When local dredging is made, deposition occurs in the dredged stretches and the bed returns to the original condition.

The same phenomenon occurs when the cut-off channel is constructed to correct/straighten the conspicuous meandering. The cut-off channel constructed with steeper slope in the existing channel causes river bed degradation due to increased velocity in the upper reaches and riverbed aggradation 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, (see Figure 8.2.2c, Scale of Typical Sand Bars**)**. However, the flow attack zones start to move causing modification of the sand bars due to the following river works.

i. Construction of ground sill

ii. Construction of spur dike and/or revetment

iii. Dredge/excavation of river bed including gravel extraction

Meanders are formed in alluvium (water-deposited material, usually unconsolidated), by a continuous process of erosion, transportation and deposition of the bed material. Usually, material is eroded from the concave portion (outer bend) of a meander, transported downstream and deposited on the convex portion (inner bend), or bar, of a meander. The material is often deposited on the same side of the stream from which it was eroded. Meanders will usually appear wherever the river traverses in gentle slope consisting of fine-grained material that is easily eroded and transported but has sufficient cohesiveness to provide firm banks.

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.

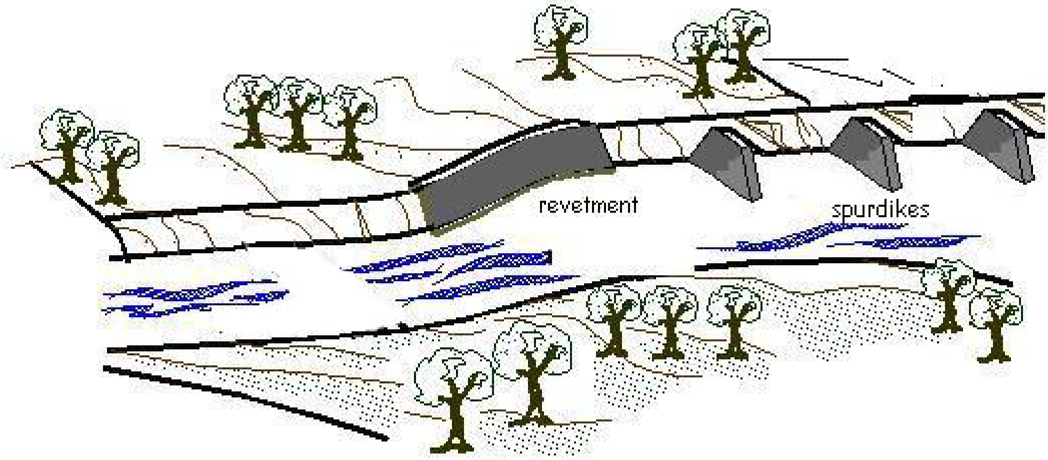


Figure 8.2.2q: Typical River Structures for Bank Erosion or Meandering

8.3 River Channel Characteristics

The channel improvement should attain not only the design flood discharge capacity but also the channel stability, which makes river course and bed changes less susceptible and maintenance works easy.

In the river improvement planning, the analysis of river characteristics is vital in order to determine the various factors that can affect the stability of the existing river channel.

The following parameters are taken into consideration in planning.

1) Typical scale of channel: channel width (B), water depth (H), mean bed slope (S_{BM}) and flow velocity (V).
2) Floodplain high-water channel characteristics of deposits and behavior during flood
3) Channel alignment: types of meander, relationship of sand bars, location and rate of bank erosion, and formation of islands
4) Channel cross section: scour depth, changes in cross section due to flood
5) Types of change in longitudinal profile of channel: rate of change, bed armoring
6) Others such as small-scale bed form, sediment discharge, ecosystem, types of human-induced change in river channel

The river parameters described above can be roughly classified and described in terms of mean annual maximum discharge Q_m, , river bed slope S_b and representative grain size diameter of bed material dr,

Using the bankfull discharge Q_m, bed slope S_b and representative grain diameter size of bed material dr, 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 8.2.1. 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 (Hm) 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 can be estimated from the representative riverbed material (dr) and the average riverbed slope (Sb) using Figure 8.2.2f.

dr = 1 cm or less; The velocity, V = 2m/sec or less
dr = 1 cm or more; V = 2m/sec or more
dr = 10 mm or more;   V = 3 m/sec or more.

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 (Hm) of the mean annual maximum flood (8.2.2b) 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 8.2.2 4) Erosion).

8.4 River Channel Planning

8.4.1 Planning Procedure

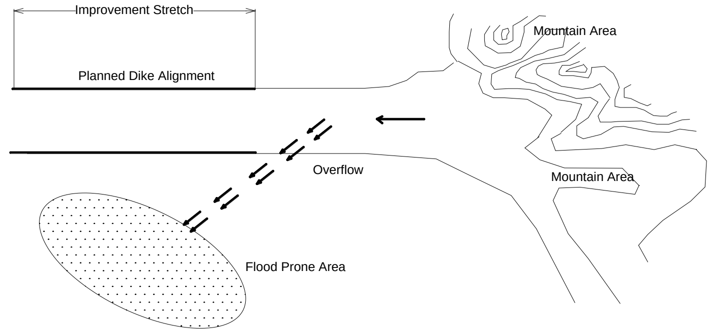
The procedure to formulate the river improvement plan includes the following:

1) Identify the improvement stretch of the project
2) Set the channel route/alignment of river
3) Establish the river cross section, longitudinal profile showing the design flood level

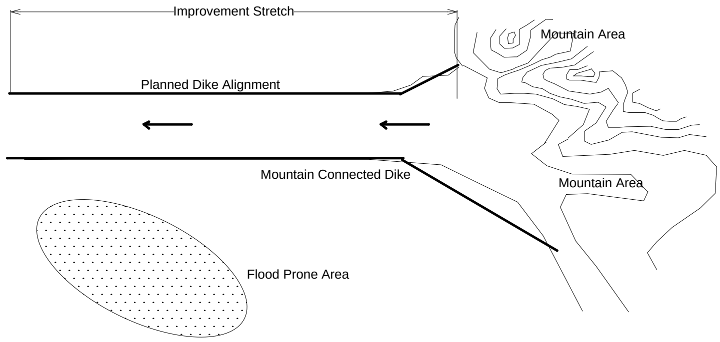
and riverbed slope

8.4.2 Improvement Stretch

The proposed improvement 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. The stretch with flow capacity less than the design discharge will cause inundation. Also the stretch for improvement should be connected/anchored to higher bank with flow capacity equal to or more than the design discharge, e.g., mountain connected dike to prevent overflowing as shown in Figure 8.4.2.



a. No Connection to Other Structure



b. Connected to Mountain Dike

Figure 8.4.2: Determination of Improvement Stretch

8.4.3 River Channel Route

River channel route should basically follow the existing one. If there is a flow constraint on the channel due to sharp meandering, then cut-off channel is recommended. Alternative routes should be set by combining the portions of existing and new excavated river. The following conditions shall be taken into consideration in the selection of the best route :

1) topography and geology,
2) present and future land uses,
3) administrative district/boundary,
4) irrigation and drainage systems,

5) influence to groundwater level,

6) structural measures for inland waters,

7) influence to the upper and lower reaches of the planned section (e.g., sediment balance, etc.)

8) project cost,

9) operation and maintenance.

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 8.4.2)

4) For wide river with steep slope and high velocity (e.g. alluvial fan), open dikes may be planned 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 8.4.3).

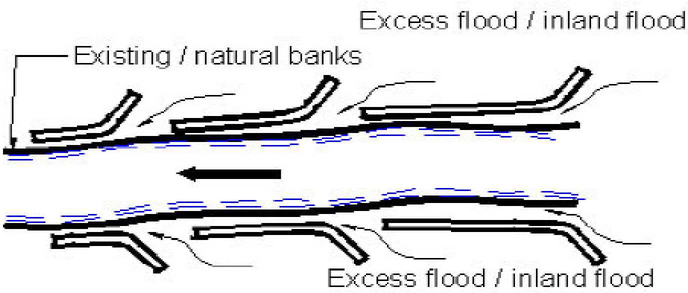


Figure 8.4.3: Open Dike

Chapter 9 SOCIO-ECONOMIC ANALYSIS

The objective of economic evaluation is not to assess the efficiency of the investment and the return of the project itself, but to evaluate the viability of the identified project alternatives.

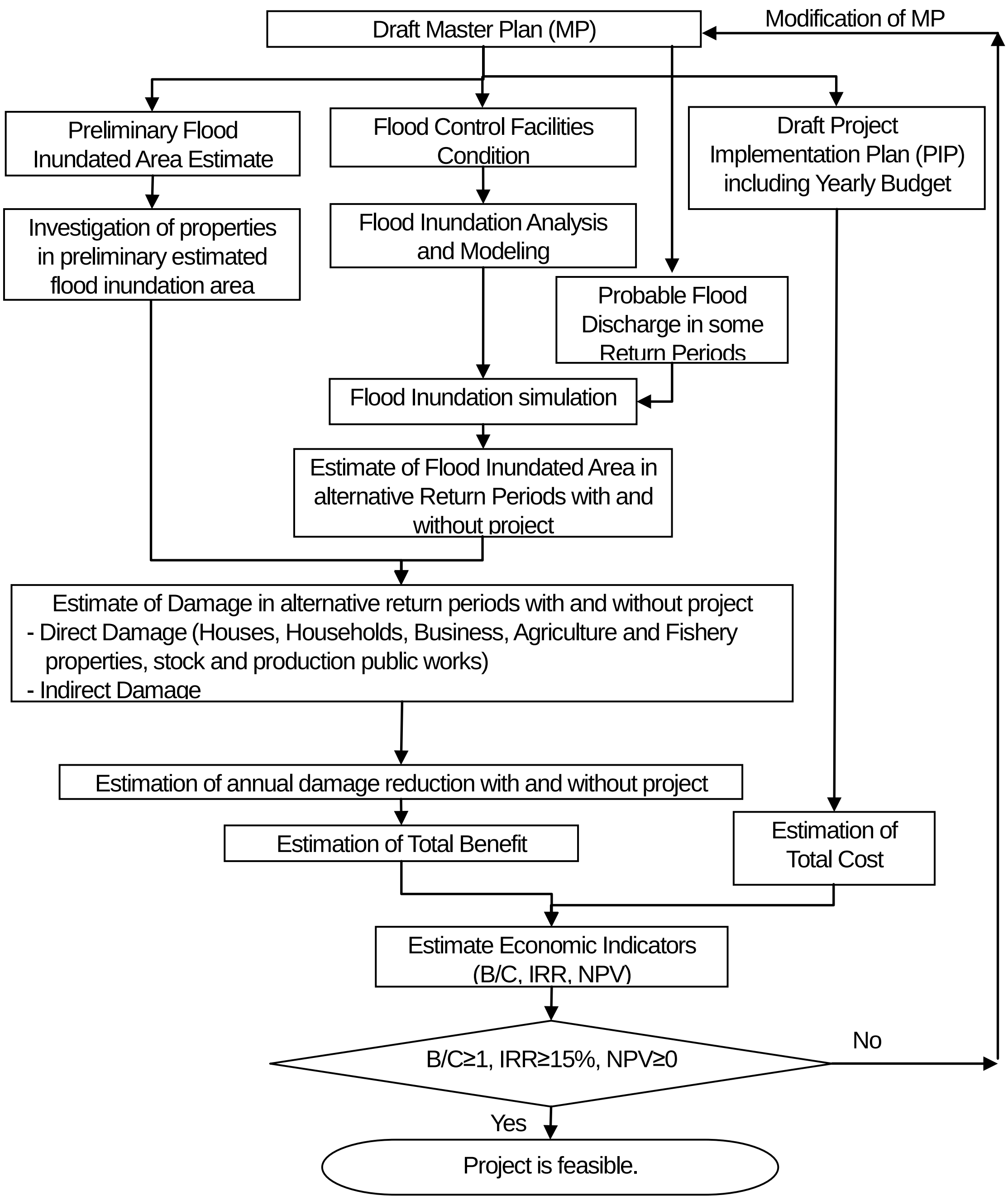


Figure 9.1: Procedure of Socio-economic Analysis

9.1 Economic Evaluation

The following procedures in the economic evaluation of the projects are:

1) Identify the most likely specific damages.
2) Estimate the basic unit value per unit and/or unit area (amount/unit, or amount/ha) for the specific damages.

3) Evaluate the damages due to the floods as the basis of evaluation.
4) Estimate the annual average flood damages by means of probability analysis for each return period under the "With-" and "Without-Project" concept.
5) Identify the economic benefit as differences of damages in the "With-" and "Without-Project" conditions.
6) Compare the economic benefit with the economic cost of project, and evaluate project feasibility by means of some indices such as the economic internal rate of return (EIRR), the net present value (NPV), and the B/C Ratio.

9.1.1 Economic Internal Rate of Return (EIRR)

The Asian Development Bank (the ADB) states that project is feasible if EIRR exceeds Opportunity Cost of Capital (the OCC). In developing countries, the EIRR ranges from 8 % to 12%. NEDA requires at least 15% or more of EIRR for foreign and locally funded projects

The Economic Internal Rate of Return (EIRR) is calculated using the cash flow of economic cost and benefit during the project life, defined by the following formula:

$$\sum_{t=1}^{t=T} \frac{C_t}{(1+R_e)^t} = \sum_{t=1}^{t=T} \frac{B_t}{(1+R_e)^t}$$

Equation 9.1.1

Where:

T	:	the last year of the project life;
Ct	:	an annual economic cost flow of the project in year *t*;
Bt	:	annual benefit flow derived from the project in year *t*; and
Re	:	the EIRR (a discount rate to be used for costs resulting in the same amount of benefit in terms of present value).

When the resulting EIRR is the same or higher than the discount rate applied for the present value of both the benefit and cost, the project is feasible.
Generally, the economic cost of a project is defined as the opportunity cost of capital. Goods and services invested in a project are no longer utilized for other projects. The benefits of other projects are sacrificed. This is called sacrificed benefits or opportunity cost of the project. The applied discount rate is generally considered as the same rate of the said opportunity cost of capital. Therefore, in case the resulting EIRR is higher than the applied discount rate, the economic viability of the project is higher than the rate of opportunity cost of capital or the sacrificed benefit of the other project.

9.1.2 Net Present Value

The NPV is expressed as "B-C" and defined by the following formula:

$$NPV = B - C = \sum_{t=1}^{t=T} \frac{B_t}{(1+R_e)^t} - \sum_{t=1}^{t=T} \frac{C_t}{(1+R_e)^t}$$

Equation 9.1.2

If "B-C" (difference between the present value of cost and the present value of the benefit) is positive (>0), , it means that the project under the Study will have a viability to execute.

9.1.3 Benefit Cost Ratio

The B/C Ratio is defined by the following formula:

$$B/C = \sum_{t=1}^{t=T} \frac{B_t}{(1+R_e)^t} \Bigg/ \sum_{t=1}^{t=T} \frac{C_t}{(1+R_e)^t} \qquad \text{Equation 9.1.3}$$

If the rate of the present value of the benefit divided by the present value of the cost is more than “1.00”, then the project is feasible. Project life is assumed at 50 years after the completion of the project. Cash flow of the economic cost and economic benefit should be made from the first year of the construction works up to the end of each project life.

Also, annual operation and maintenance cost (O&M Cost) should be taken into account; and the replacement cost, if any, should be considered since the initial works of the facilities are temporary in the project life.

1) Basic Conditions

   In the process of determining the different economic indicators such as the Net Present Value (NPV), the Benefit-Cost Ration (B/C) and the Internal Rate of Return (IRR) of the proposed priority projects that would address the flooding problems, the following basic conditions and assumptions are applied:

   a) A project life of 50 years after completion

   b) A detailed design period is 2 years and a construction period is 5 years. In each period the budget is equally distributed every year.

9.2 Classification of Flood Control Benefits

Benefit of flood control project is generally classified as follows;

1) Direct Benefit
2) Indirect Benefit
3) Intangible Benefit
4) Development Benefit

9.2.1 Direct Benefits

Direct Benefits are those that accrued directly from the reduction or prevention of flood damages (deterrent effects). The benefit is the value for reducing flood damage risk of productive land and assets in the flooded area. It is monetary value based on the flood damage amount without the project condition.

The direct benefits refer to the prevented or reduced direct damages that can be caused by floods. Direct damage is the damage directly inflicted on vulnerable assets that consists of the following:

- Agricultural damages include damages to crops, livestock and aquaculture; and
- Non-agricultural damages include damages to houses and household goods, buildings and facilities in the buildings, and infrastructures.

The calculation of damage value is based on the sociological investigation of the flood prone area which includes productivity, quantity of public and private buildings and social infrastructures. The damage rates for them are provided based on the flood inundation depth and duration for all the properties in the study area.

Other direct benefits arise from the interruption and/or suspension of economic activities, transport movement, and cost of rescue and relief activities. These benefits are accrued from the reduction of the opportunity to generate income and can be technically translated as the Opportunity Cost.

Estimation of Direct Benefit is the cost of flood damage under the without the project scenario called as "Damage Assessment", estimated based on the following procedures:

1) Flood Inundation Depth-Area(-Duration) Analysis
2) Estimation of Annual Flood Damage Amount (Annual Direct Benefit)

   The annual flood damage (direct benefit) is estimated based on the flood inundation depth-area (duration) analysis that can be quantified. Flood inundation analysis should be carried out prior to the estimation of direct benefit of flood control project and can be done by adopting the following process:

   a) Preparation of probable flood hydro-graph or peak discharge by hydrological analysis: (2-year, 5-year, 10-year, 25-year, 50-year, 100-year and so on);
   b) Flood inundation analysis to be done by adequate hydraulic analysis, for example; Uniform flow, Non-uniform flow & Non-steady flow calculation under the existing condition of the river channel (condition of without flood control project); and

   Estimation of Flood Inundation Depth-Area(-Duration) by the different scale of flood based

   a) Delineation of the Land-use classification of flood inundation area based on the sociological survey and the results of the "Flood Inundation Depth-Area (–Duration)";
   b) Estimation of unit damage value of assets /productions of land/ buildings in the flood inundation area by sociological survey;
   c) Establishment of the corresponding factor per inundation depth and Land-use class; and
   d) Estimation of Flood Damages corresponding to the given return-period of recurrence.

9.2.3 Indirect Benefits

Indirect Benefit is considered for the damage risk mitigation such as traffic disruptions, loss of retail and industrial output, interruptions in utility services, the cost of emergency operation and so on. The damage value is generally estimated based on the flood damage investigation through hearing and sociological sounding. This damage is quantifiable but difficult to convert to monetary value; hence, in such case the indirect benefit is seldom taken into account.

9.2.4 Intangible Benefit

Intangible benefits of flood control project are non-quantifiable monetary value; hence, estimation is difficult. These include losses of lives, productivity, quality of life, additional stress/anxiety/sickness and other traumatic experiences, which are major issues in project assessment.

Most flood control projects do not consider the intangible benefits; even though, significant flood protection / mitigation can improve the welfare of the people in the flood prone area.

9.2.4 Development Benefits

Development benefits promote the acceleration or enhancement of economic growth and development of the area due to flood protection. The accompanying positive impact due to development may include the following:

- Land enhancement
- Increase agricultural production
- Improve agri-aquacultural activity
- Change in economic structure
- Employment opportunities
- Enhance investment potentials; and
- Improvement in quality of life.

Development benefit of the flood control project can be estimated based on the development of the project area after the completion of flood control project. The difference of land use condition with and without project can be considered based on the existing regional development plan and so on.

Development benefit for flood control project is indispensable in economic evaluation because it not only protects the existing properties in the flood prone area but also triggers development in accordance with the national/ regional development plan.

Project components that contribute development should be noted and taken into account, such as: heightening of lands using river excavation / dredging materials, road on dike's crest, river side park, residential and industrial area, and so on.

9.2.5 Estimation of Total Benefits

The total benefit is estimated based on the annual benefit covering the entire project life. In this estimate, the future benefits are converted with the discount rate to calculate the present values, which become the total benefit. The total benefit is estimated for river basin or project based on the implementation period, project life, and the discount rate.

9.3 Economic Cost

The Economic Cost is usually applied to EIRR calculation for the cost stream. Economic cost is estimated from the market price (Project cost), from which (a) Tax, duty and interest during the construction are to be deducted, and (b) Exchange rate, labor cost and land acquisition cost are to be adjusted.

Market Price

The market price (Project cost) is the same amount as the investment cost. The Project is assessed in terms of expense and income balance; where, the expense is the amount of investment and the income as the cash return from the Project. The expense is called "Cost" and income as "Benefit". The financial viability of the Project is assessed through the market price and the real cash income.

The expenses and income should be assessed in view of the national economy. The economic cost should be reflected as real resources of the project. For example, the cement is necessary to construct the project but tax for cement is not really needed for construction which is transferred to national treasury. It means that the tax is paid by the Project. (National Government) to the national treasury, which is no expenses by the owner of the Project.

Land Acquisition Cost

Land acquisition cost is usually not considered as economic cost because the market price of the land is speculative investment for which price increase is expected. The market price of land is therefore usually not real price of resources. Instead of the cost stream, the land price is considered as the "Negative Benefit" for the economic evaluation. To cite an example, the acquired agricultural land used for river by the Project cannot be expected be productive, which is loss of the benefit. For the financial evaluation however, the market price of land acquisition cost should be considered as the cost stream.

Labor Cost

The economic cost for skilled labors such as equipment operator, foreman, is usually the same as the actual payment because such skilled labors will continue the same work for the different projects, and the number of skilled labor is limited and their salary is generally stable as the actual value of labor force, which means that the payment to skilled labor is reflected to their real value of labor resources.

For the cost of non-skilled labor is however rather different. Non-skilled labor is usually temporary workers during the construction stage, and the salary for the labors is generally determined based on the national standard such as minimum wages of the workers in the country. The minimum wage is usually determined based on the minimum required living cost of the country, but it is not reflected real value of labor resources. Generally the real value of the non-skilled labor resources is less than the salary to the non-skilled labors, which should be adjusted for economic evaluation.

Shadow Price Rate

In addition to the above adjustment, there are many cost items which do not reflect real value of resources, such as cost of imported material and equipment, which are usually added from the real value. To simplify the conversion of market price of the Project cost to economic cost, shadow price is applied in many projects, which is estimated through detail assessment of the cost breakdowns of material, equipment and labor. Generally, the shadow price rate is applied about 80% to 95% of the market price in developing countries.

9.4 Cost Estimation

1) Unit Cost

   Unit costs are basically derived from related projects by the DPWH of similar price index.

2) Cost Estimates Items

Project Cost

a) Constitution of Project Cost

Financial project cost consists of construction cost, administration cost, engineering services, compensation cost and physical contingency.

i Construction Cost

The construction cost consists of preparatory works and main construction cost.

Preparatory works is estimated at 15% of the main construction cost.

- Mobilization Works (10% of total amount of civil works)
- Unmeasured Factors (5% of total amount of civil works)

Main Construction Cost/ The cost for main works is computed by multiplying the unit cost with the work quantity.

ii Administrative Cost

The administrative cost is computed at 5% of the construction cost.

iii Engineering Services

The cost of engineering services is to cover detailed design and construction supervision. 15% of the construction cost is adopted to the engineering services cost, which consists of 10% for the detailed design and 5% for the construction supervision.

iv . Compensation Cost

The compensation cost is divided into land acquisition and house evacuation. These costs are estimated for the respective river basins.

v. Physical Contingency

The physical contingency is estimated at 10% of the sum of the construction cost, administration cost, engineering services cost and compensation cost.

vi Price Contingency

The price contingency is estimated at 3% of total amount of direct cost.

2) O/M and Replacement Cost

The operation and maintenance (O&M) cost of the structural measures is practically estimated at 1.5% annually of the construction cost

A replacement cost of gate is estimated at 70% of their construction costs with replacement, at least every 15 years.

9.5 Estimation of Annual Average Benefit (Annual Damage Reduction)

The flood control benefit is defined as the damage reduction by the designed works. The annual average benefit is calculated as the sum of the product of an annual average damage reduction caused by floods and discharges and an occurrence probability between those floods.

9.6 Example

The example is taken from the Master Plan of Kinanliman River Basin with return of 25 years.

Table 9.6a: Estimated Flood Damage (1)

Cross Section: 119.48m, Left Bank Land Elevation (EL.m): 6.9, Right Bank Land Elevation (EL.m): 6.97

Return Period (years)	Discharge(m3/s)	Water Surface Level (EL.m)	Water Depth in Left Bank Land (m)	Water Depth in Right Bank Land (m)
2	201	6.83	0	0
5	281	7.63	0.73	0.66
10	323	8.01	1.11	1.04
25	380	8.49	1.59	1.52
50	413	8.78	1.88	1.81
100	459	9.16	2.26	2.19

Table 9.6b: Estimated Flood Damage (2)

Return Period (years)	Numbers of Houses by Ground Elevation													
	Left Bank h<5	Left Bank 5≤h<6	Left Bank 6≤h<7	Left Bank 7≤h<8	Left Bank 8≤h<9	Left Bank 9≤h<10	Left Bank 10≤h<20	Right Bank h<5	Right Bank 5≤h<6	Right Bank 6≤h<7	Right Bank 7≤h<8	Right Bank 8≤h<9	Right Bank 9≤h<10	Right Bank 10≤h<20
2	90	126	75	54	29	43	34	0	2	0	0	0	0	2
5	90	126	75	54	29	43	34	0	2	0	0	0	0	2
10	90	126	75	54	29	43	34	0	2	0	0	0	0	2
25	90	126	75	54	29	43	34	0	2	0	0	0	0	2
50	90	126	75	54	29	43	34	0	2	0	0	0	0	2
100	90	126	75	54	29	43	34	0	2	0	0	0	0	2

Table 9.6c: Estimated Flood Damage (3)

Return Period (years)	Inundated or not inundated													
	Left Bank h<5	Left Bank 5≤h<6	Left Bank 6≤h<7	Left Bank 7≤h<8	Left Bank 8≤h<9	Left Bank 9≤h<10	Left Bank 10≤h<20	Right Bank h<5	Right Bank 5≤h<6	Right Bank 6≤h<7	Right Bank 7≤h<8	Right Bank 8≤h<9	Right Bank 9≤h<10	Right Bank 10≤h<20
2	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5	1	1	1	1	0	0	0	1	1	1	1	0	0	0
10	1	1	1	1	1	0	0	1	1	1	1	1	0	0
25	1	1	1	1	1	0	0	1	1	1	1	1	0	0
50	1	1	1	1	1	0	0	1	1	1	1	1	0	0
100	1	1	1	1	1	1	0	1	1	1	1	1	1	0

if inundated: 1, if no-inundated: 0

Table 9.6d: Estimated Flood Damage (4)

Return Period (years)	Number of inundated houses													
	Left Bank h<5	Left Bank 5≤h <6	Left Bank 6≤h <7	Left Bank 7≤h <8	Left Bank 8≤h <9	Left Bank 9≤h <10	Left Bank 10≤h< 20	Right Bank h<5	Right Bank 5≤h <6	Right Bank 6≤h <7	Right Bank 7≤h <8	Right Bank 8≤h <9	Right Bank 9≤h <10	Right Bank 10≤h< 20
2	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5	90	126	75	54	0	0	0	0	2	0	0	0	0	0
10	90	126	75	54	29	0	0	0	2	0	0	0	0	0
25	90	126	75	54	29	0	0	0	2	0	0	0	0	0
50	90	126	75	54	29	0	0	0	2	0	0	0	0	0
100	90	126	75	54	29	43	0	0	2	0	0	0	0	0

Table 9.6e: Estimated Flood Damage (5)

Return Period (years)	Number of houses by water depth in left bank				Number of houses by water depth in right bank				Ave. Assess Value	Damage Ratio by Inundation Depth				Damage by Inundation Depth (Left Bank)				Damage by Inundation Depth (Right Bank)				Total Damage
	0.10≤h < 0.30	0.30≤h < 0.60	0.60≤h < 1.00	1.00≤h	0.10≤h < 0.30	0.30≤h < 0.60	0.60≤h < 1.00	1.00≤h	peso/unit	0.10≤h< 0.30	0.30≤h< 0.60	0.60≤h< 1.00	1.00≤h	0.10≤h< 0.30	0.30≤h< 0.60	0.60≤h< 1.00	1.00≤h	0.10≤h < 0.30	0.30≤h < 0.60	0.60≤h< 1.00	1.00≤h	
2	0	0	0	0	0	0	0	0	750,000	0.025	0.030	0.035	0.040	0	0	0	0	0	0	0	0	0
5	0	0	345	0	0	0	2	0	750,000	0.025	0.030	0.035	0.040	0	0	9,056,250	0	0	0	52,500	0	9,108,750
10	0	0	0	374	0	0	0	2	750,000	0.025	0.030	0.035	0.040	0	0	0	11,220,000	0	0	0	60,000	11,280,000
25	0	0	0	374	0	0	0	2	750,000	0.025	0.030	0.035	0.040	0	0	0	11,220,000	0	0	0	60,000	11,280,000
50	0	0	0	374	0	0	0	2	750,000	0.025	0.030	0.035	0.040	0	0	0	11,220,000	0	0	0	60,000	11,280,000
100	0	0	0	417	0	0	0	2	750,000	0.025	0.030	0.035	0.040	0	0	0	12,510,000	0	0	0	60,000	12,570,000

Table 9.6f: Annual Damage Reduction

River Discharge	Return Period		Amount of Damage			Average damage reduction of interval	Probability of interval	Annual Damage reduction	Accumulation of annual damage reduction
			Without Project	With Project	Reduction of Damage				
201	2	0.5	0	0	0				0
281	5	0.2	9,108,750	0	9,108,750	4,554,375	0.300	1,366,313	1,366,313
323	10	0.1	11,280,000	0	11,280,000	10,194,375	0.100	1,019,438	2,385,750
380	25	0.04	11,280,000	0	11,280,000	11,280,000	0.060	676,800	3,062,550
413	50	0.02	11,280,000						
459	100	0.01	12,570,000						

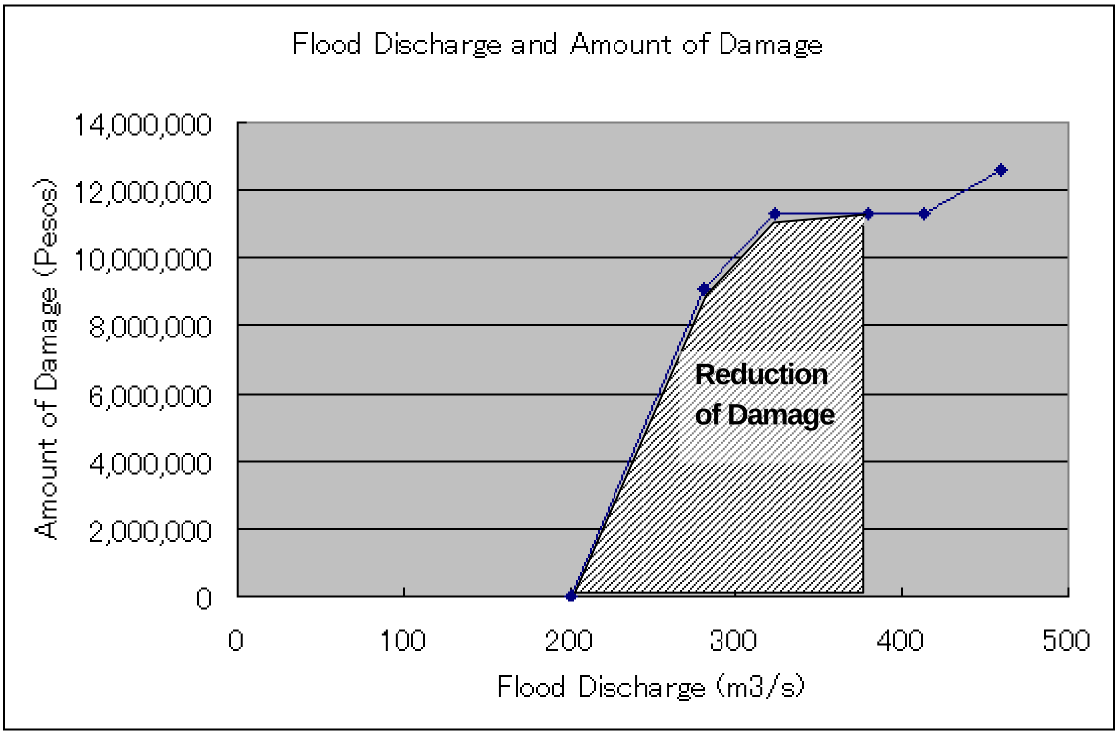


Figure 9.6a: Flood Discharge and Amount of Damage

Table 9.5g: Economic Cost and Benefit Stream of the Kinanliman River Basin

No of Year	Year	Cost (Mil. Pesos)				Present Value of Cost (Mil. Pesos)	Benefit (Mil. Pesos)			Net Benefit (Mil. Pesos)	Net Present Value (Mil. Pesos)
		D/D & Consruction	O/M	Replacement	Total		Benefit	Inflationary Benefit	Present Benefit		
t		a	b	c	d=a+b+c	e=d/(1+r)^t	f	g=f*s^(t+2)	h=g/(1+r)^t	i=g-d	j=i/(1+r)^t
1	2009	1.1			1.1	1.0	0.0	0.0	0.0	-1.1	-1.0
2	2010	1.1			1.1	0.8	0.0	0.0	0.0	-1.1	-0.8
3	2011	5.4			5.4	3.6	0.0	0.0	0.0	-5.4	-3.6
4	2012	5.4			5.4	3.1	0.6	0.7	0.4	-4.7	-2.7
5	2013	5.4			5.4	2.7	1.2	1.5	0.7	-3.9	-1.9
6	2014	5.4			5.4	2.3	1.9	2.5	1.1	-2.9	-1.3
7	2015	5.4			5.4	2.0	2.5	3.4	1.3	-2.0	-0.8
8	2016		0.3		0.3	0.1	3.1	4.3	1.4	4.0	1.3
9	2017		0.3		0.3	0.1	3.1	4.5	1.3	4.2	1.2
10	2018		0.3		0.3	0.1	3.1	4.6	1.1	4.3	1.1
11	2019		0.3		0.3	0.1	3.1	4.8	1.0	4.5	1.0
12	2020		0.3		0.3	0.1	3.1	4.9	0.9	4.6	0.9
13	2021		0.3		0.3	0.0	3.1	5.1	0.8	4.8	0.8
14	2022		0.3		0.3	0.0	3.1	5.3	0.7	5.0	0.7
15	2023		0.3		0.3	0.0	3.1	5.4	0.7	5.1	0.6
16	2024		0.3		0.3	0.0	3.1	5.6	0.6	5.3	0.6
17	2025		0.3		0.3	0.0	3.1	5.8	0.5	5.5	0.5
18	2026		0.3		0.3	0.0	3.1	6.0	0.5	5.7	0.5
19	2027		0.3		0.3	0.0	3.1	6.2	0.4	5.9	0.4
20	2028		0.3		0.3	0.0	3.1	6.4	0.4	6.1	0.4
21	2029		0.3		0.3	0.0	3.1	6.6	0.4	6.3	0.3
22	2030		0.3	0.6	0.9	0.0	3.1	6.8	0.3	5.9	0.3
23	2031		0.3		0.3	0.0	3.1	7.1	0.3	6.8	0.3
24	2032		0.3		0.3	0.0	3.1	7.3	0.3	7.0	0.2
25	2033		0.3		0.3	0.0	3.1	7.5	0.2	7.2	0.2
26	2034		0.3		0.3	0.0	3.1	7.8	0.2	7.5	0.2
27	2035		0.3		0.3	0.0	3.1	7.8	0.2	7.5	0.2
28	2036		0.3		0.3	0.0	3.1	7.8	0.2	7.5	0.1
29	2037		0.3		0.3	0.0	3.1	7.8	0.1	7.5	0.1
30	2038		0.3		0.3	0.0	3.1	7.8	0.1	7.5	0.1
31	2039		0.3		0.3	0.0	3.1	7.8	0.1	7.5	0.1
32	2040		0.3		0.3	0.0	3.1	7.8	0.1	7.5	0.1
33	2041		0.3		0.3	0.0	3.1	7.8	0.1	7.5	0.1
34	2042		0.3		0.3	0.0	3.1	7.8	0.1	7.5	0.1
35	2043		0.3		0.3	0.0	3.1	7.8	0.1	7.5	0.1
36	2044		0.3		0.3	0.0	3.1	7.8	0.1	7.5	0.0
37	2045		0.3	0.6	0.9	0.0	3.1	7.8	0.0	6.9	0.0
38	2046		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
39	2047		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
40	2048		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
41	2049		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
42	2050		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
43	2051		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
44	2052		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
45	2053		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
46	2054		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
47	2055		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
48	2056		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
49	2057		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
50	2058		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
51	2059		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
52	2060		0.3	0.6	0.9	0.0	3.1	7.8	0.0	6.9	0.0
53	2061		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
54	2062		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
55	2063		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
56	2064		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
57	2065		0.3		0.3	0.0	3.1	7.8	0.0	7.5	0.0
Total		29.2				16.0			16.7		0.4

29.4

B/C= 16.7 ÷ 16.0

1.04

NPV= 0.7 Million Pesos

EIRR= 15.5%

r (Discount Rate) = 15%

s (Growth rate of GNP per Capita) = 3.35% (until 2034)

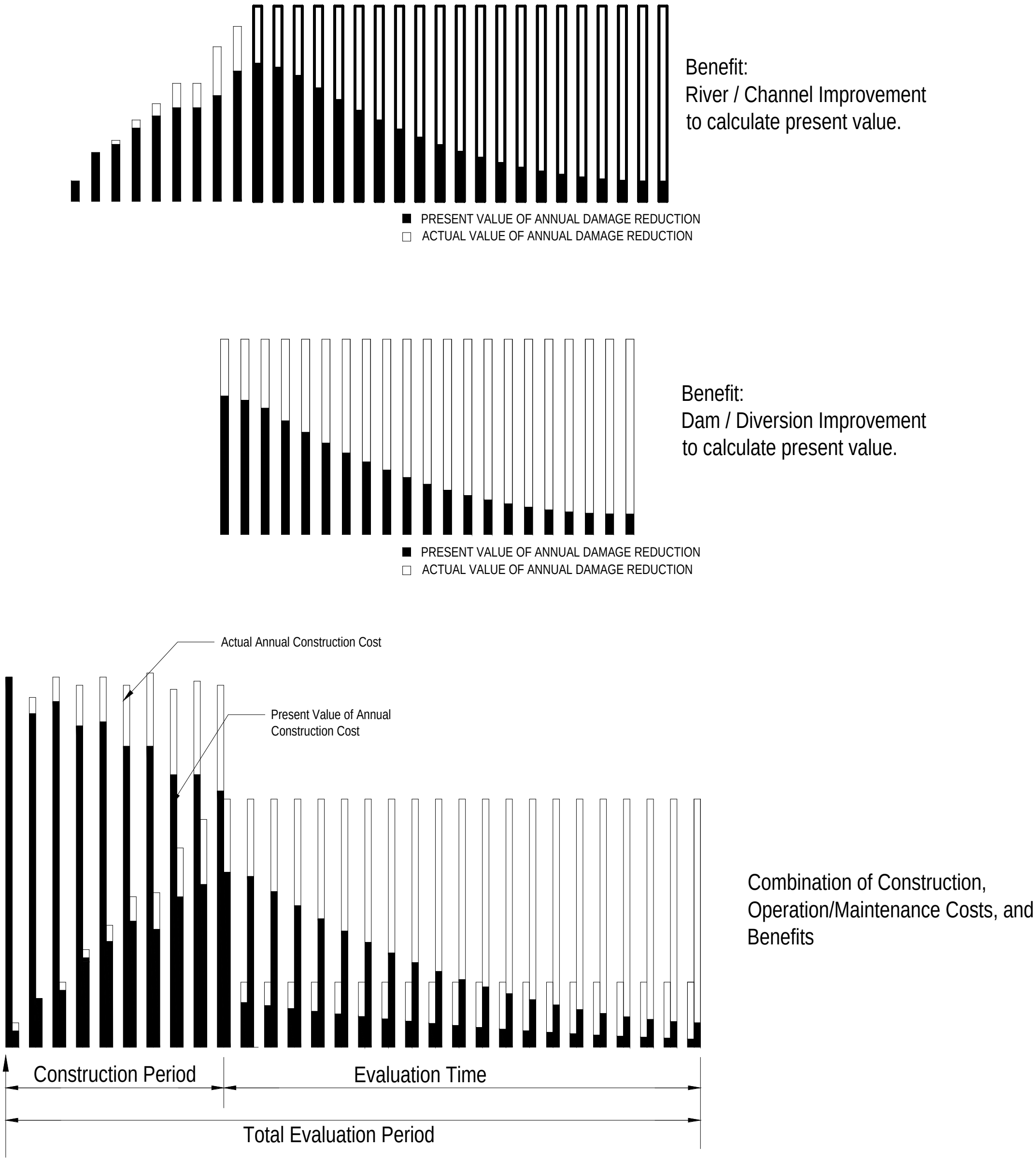


Figure 9.6b: Image of Calculated Present Value of Costs and Benefits of Flood Control Projects

3) Damage Cost Due Traffic Interruption

Mitigating traffic interruption caused by flood and/or sediment disaster, which results to bridge collapse, road inundation and washed out road, is considered as indirect benefit. Indirect benefits can be assessed from the following:

- Roads or railroads damaged by flood water and/or sediment flow will disturb traffic flow in the nearby and surrounding areas.
- Additional cost to make a detour shall be considered as damage cost.
- Damage cost is calculated by summing up the losses incurred due to time,

running and fixed costs.

$D = LT + LD$ Equation 9.5a

Where:

D	:	Damage cost
LT	:	Loss due to additional time
LD	:	Loss due to Distance

Loss of time is calculated as below.

$$LT = \sum_j \sum_l \left(Q_{jl} \times TTF_{jl} \times \alpha_j - Q_{jl} \times TTO_{jl} \times \alpha_j\right) \quad \text{Equation 9.5b}$$

Where:

Q_{jl}	:	Traffic flow of type j vehicle in linkage l (numbers / day)
TTF_{jl}	:	Traffic time in flooding condition (min)
TTO_{jl}	:	Traffic time in ordinary condition (min)
α_j	:	Unit cost for time (php / min * number of car)

Loss of distance is calculated as below.

$$LD = \sum_j \sum_l \left(Q_{jl} \times TDF_{jl} \times \beta_j - Q_{jl} \times TDO_{jl} \times \beta_j\right) \quad \text{Equation 9.5c}$$

Where:

Q_{jl}	:	Traffic flow of type j vehicle in linkage l (numbers / day)
TDF_{jl}	:	Traffic distance in flooding condition (min)
TTO_{jl}	:	Traffic distance in ordinary condition (min)
β_j	:	Unit cost for travel (php / min * number of car)

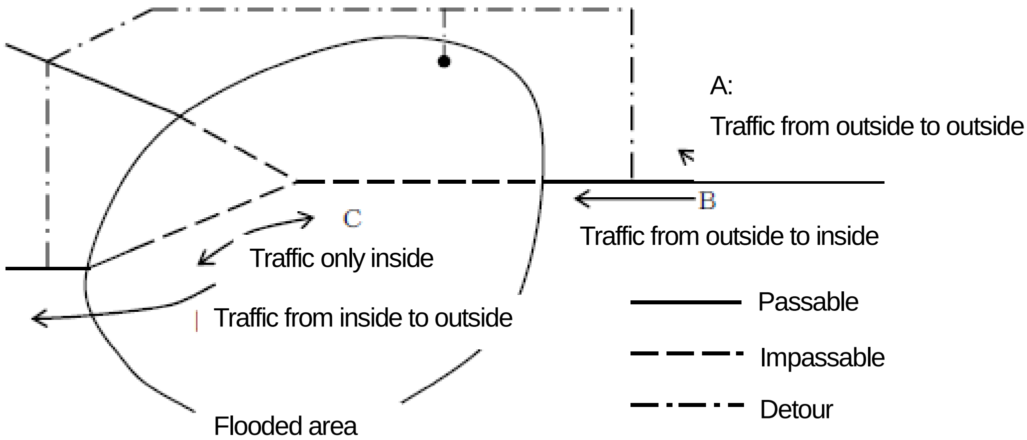


Figure 9.5b: Traffic Rerouting Due to Flood

Or, we can use the equation below.

$$TC = PD \times AADT \times L \times VOC \times N \qquad \text{Equation 9.5d}$$

Where:

TC	:	Traffic Cost
PD	:	Percentage of traffic diversion
AADT	:	Traffic from NTCP
L	:	Length
VOC	:	Vehicle operation cost
N	:	Period of traffic interruption

Chapter 10 ENVIRONMENTAL IMPACT ASSESSMENT

10.1 General

Philippine EIA System

Presidential Decree PD 1156 (1977), requires EIA preparation for projects affecting environmental quality and PD 1586 (1978) defines the EIS System and its scope for Environmental Critical Projects and Environmental Critical Areas. Projects are scrutinized through Environmental Impact Assessment, which is an important tool in all aspects of project cycle. The Department of Environment and Natural Resources (DENR) Department Administrative Order No. 30 Series of 2003 describes various documentary requirements for environmental certificates. The certificates are Environmental Compliance Certificate ECC and/or Certificate of Non-Coverage CNC, where issuance depends on the significance of the social and environmental impacts due to project activities.

EIA helps to determine the impacts of the proposed project and to formulate countermeasures to prevent, minimize, mitigate, or compensate the adverse impacts caused by the project. Environmental consequences should be recognized early in the project cycle to account for the project selection, siting, planning, and design Integration of environmental assessment in the early stage of planning is essential for the following reasons.

1) To address environmental issues in a timely and cost-effective fashion,

2) To incorporate alternatives to the proposed project,

3) To avoid costs and delays in the implementation due to unanticipated environmental problems.

10.2 General EIA for Master Plan

At initial stage of planning, details of the project are not available, but the basic nature of the project alternatives are proposed for example, whether it is a dam, dike, revetment, cut-off channel or combinations of these structures; and areas which are likely to be inundated; and the proposed project sites.

During Master Plan, it is the aim that environmental disastrous alternatives are identified and eliminated, and new alternatives are introduced. Each alternative will be subjected to analysis commensurate to the expected impacts, which may cover the following contents:

1) Existing environmental baseline conditions,
2) Potential environmental impacts, direct and indirect, including environmental enhancement,
3) Systematic environmental comparison of alternative investments, sites, technologies and design,

4) Preventive, mitigatory, and compensatory measures, generally in the form of an environmental mitigation, or management plan,
5) Environmental management and training,
6) Environmental monitoring.

10.3 Detailed EIA for Feasibility Study

In feasibility study, a chapter or a volume is allotted for project environmental impact assessment. Since, the projects are already identified, detailed EIA study should be carried out in conjunction with economic, technical and design work. At this stage, the project may be subject to "screening" to decide whether a full and comprehensive EIA report must be prepared. Late preparation of EIA is cumbersome, time consuming, and expensive to incorporate recommendations in the project construction.

10.4 Brief Description of EIA Process

Under the Philippine Environmental Impact Assessment System, the table below gives the general procedure.

Table 10-4: Overview of Stages of the Philippine EIA Process[1]

SCREENING	**Screening** determines if a project is covered or not covered by the PEISS. If a project is covered, screening further determines what document type the project should prepare to secure the needed approval, and what the rest of the requirements are in terms of EMB office of application, endorsing and decision authorities, duration of processing.
SCOPING	**Scoping** is a Proponent-driven multi-sectoral formal process of determining the focused Terms of Reference of the EIA Study. Scoping identifies the most significant issues/impacts of a proposed project, and then, delimits the extent of baseline information to those necessary to evaluate and mitigate the impacts. The need for and scope of an Environmental Risk Assessment (ERA) is also done during the scoping session. Scoping is done with the local community through Public Scoping and with a third party EIA Review Committee (EIARC) through Technical Scoping, both with the participation of the DENR-EMB. The process results in a signed Formal Scoping Checklist by the review team, with final approval by the EMB Chief.
EIA STUDY and REPORT PREPARATION	The **EIA Study** involves a description of the proposed project and its alternatives, characterization of the project environment, impact identification and prediction, evaluation of impact significance, impact mitigation, formulation of Environmental Management and Monitoring Plan, with corresponding cost estimates and institutional support commitment. The study results are presented in an **EIA Report** for which an outline is prescribed by EMB for every major document type.
EIA REPORT REVIEW and EVALUATION	**Review of EIA Reports** normally entails an EMB procedural screening for compliance to minimum requirements specified during Scoping, followed by a substantive review of either composed third party experts commissioned by EMB as the EIA Review Committee for PEIS/EIS-based applications, or DENR/EMB internal specialists, the Technical Committee, for IEE-based applications. EMB evaluates the EIARC recommendations and the public's inputs during public consultations/hearings in the process of recommending a decision on the application. The EIARC Chair signs EIARC recommendations including issues outside the mandate of the EMB**.** The entire EIA review and evaluation process is summarized in the Review Process Report (RPR) of the EMB, which includes a draft decision document.
DECISION MAKING	**Decision Making** involves evaluation of EIA recommendations and the draft decision document, resulting to the issuance of an ECC, CNC or Denial Letter. When approved, a covered project is issued its certificate of Environmental Compliance Commitment (ECC) while an application of a non-covered project is issued a Certificate of Non-Coverage (CNC). Endorsing and deciding authorities are designated by AO 42, and further detailed in this Manual for every report type. Moreover, the Proponent signs a sworn statement of full responsibility on implementation of its commitments prior to the release of the ECC. The ECC is then transmitted to concerned LGUs and other GAs for integration into their decision making process. **The regulated part of EIA Review is limited to the processes within EMB control. The timelines for the issuance of decision documents provided for in AO 42 and DAO 2003-30 are applicable only from the time the EIA Report is accepted for substantive review to the time a decision is issued on the application**
MONITORING, VALIDATION, and EVALUATION/ AUDIT	**Monitoring, Validation and Evaluation/Audit** stage assesses performance of the Proponent against the ECC and its commitments in the Environmental Management and Monitoring Plans to ensure actual impacts of the project are adequately prevented or mitigated.

[1] page 15, Revised Procedural Manual for DENR Administrative Order No. 30-03 (DAO 03-30), (Implementing Rules and Regulations Of Presidential Decree No. 1586, Establishing the Philippine Environmental Impact Statement System) EMB- Environmental Impact Assessment and Management Division (EIAMD) www.emb.gov.ph/eia August 2007 (2nd Printing: January 2008)

The following discussions further explain the EIA System.

10.4.1 Screening

Screening is the assessment of the potential magnitude of project impacts and depth of study required. Projects are categorized generally as projects that will not have significant impacts, that could have significant impacts, and that would definitely have significant impacts. Screening is based on the scale and type of project, location and sensitivity of site, and the nature and magnitude of potential impacts.

Screening is the first step in the EIA process. The EIA Coverage and Requirements Screening Checklist (ESRSC) Annex 2-1a of the Revised Procedural Manual for DENR Administrative Order No. 30-03 (DAO 30-03) is filled up and submitted by the proponent to the Environmental Management Bureau (EMB). The form is downloadable from the website of the EMB or can be obtained from its office. The screening results in project groupings according to DAO 30-03 which may fall in Environmental Impact Assessment, Initial Environmental Examination, simple Project Description for the issuance of either Environmental Compliance Certificate (ECC or Certificate of Non-Coverage (CNC).

Although, most flood control projects, as they are on a piecemeal budget, fall under the Certificate of Non-Coverage. In general, Flood Control Structures addressed specific deleterious fluvial and flood plain problems, in single or in combination of type of structure; hence, mostly mitigating and enhancement measures.

The following discussion elucidates identification of project categories based on the Revised Procedural Manual of DENR DAO 30-03.

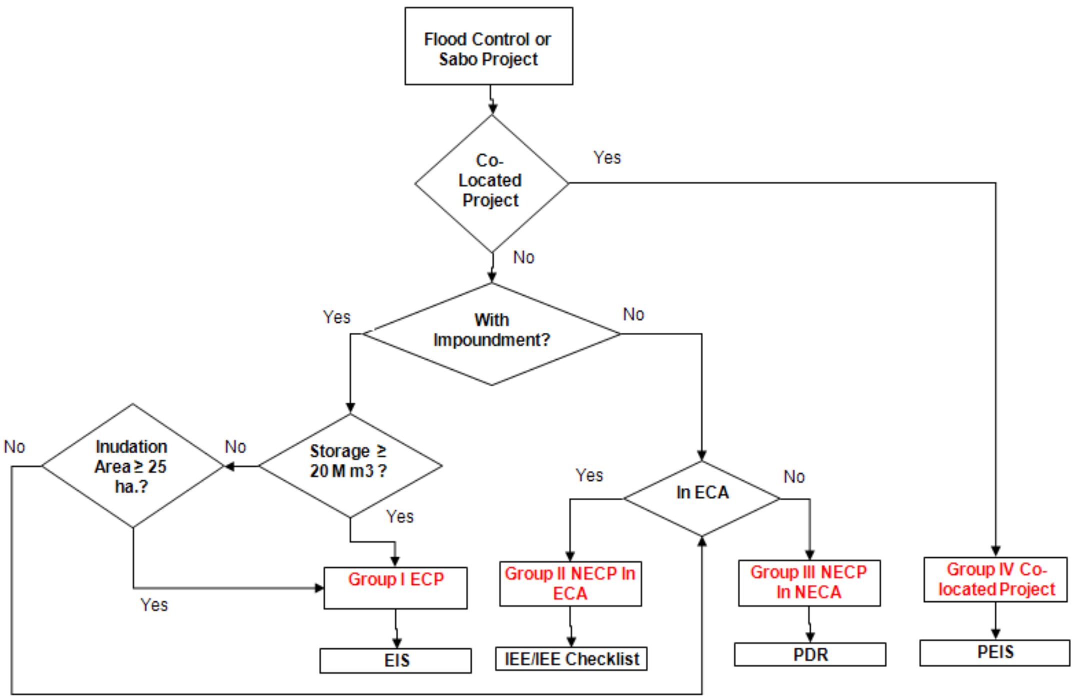


Figure 10.4: Screening Process

The result of screening would categorize flood control project in any of the following:

Group I Environmentally Critical Projects (ECP) in either Environmentally Critical Areas (ECA) or Non-Environmentally Critical Areas (NECA). For Flood Control Projects, major dam is included in this category with storage equal to or greater than 20 M m^3 or inundation area of equal to or more than 25 ha.

An Environmental Impact Assessment Report is the documentary requirement to secure Environmental Compliance Certificate.

Group II Non-Environmentally Critical Project (NECP) in Environmentally Critical Areas (ECA). Flood Control Projects are listed in this category including minor dams with storage capacity of less than 20 M m^3 and/or inundated area of less than or equal to 25 ha.

Projects not listed in the Group I or ECPs but are located in the ECA require Initial Environmental Examination, as the least documentary requirement.

Group III Non-Environmentally Critical Project (NECP) in Non-Environmentally Critical Areas (NECA). Most flood control projects of the DPWH especially the District Engineering office may fall in this project group.

Group IV Co-located projects in either ECA or NECA. Flood control projects can be classified in this category if the project is basin wide, catchment areas, with a scale transcending political boundaries or ECA or NECA.

Group V Unclassified Projects. These projects are not listed in any project groups listed above but will be evaluated by EMB for its classification later.

Environmental Critical Areas (ECA)

Environmental Critical Areas (ECA)s are delineated by the DENR and are listed in Annex 2-1a of the Procedural Manual of DAO 03-30. ECA includes any of the areas declared as:

1) National Parks, watershed reserves, wildlife preserves, sanctuaries.
2) Aesthetic potential tourist spots
3) Habitat of any endangered or threatened species of Philippine wildlife (flora and fauna)
4) Of unique historic, archeological, or scientific interests
5) Traditionally occupied by cultural communities or tribes
6) Frequently visited and/or hard-hit by natural calamities (geologic hazards, floods typhoons, volcanic activity, etc.)
7) Critical slopes
8) Prime agricultural lands
9) Recharged areas of aquifer

10) Water bodies by one or any combination of the following conditions: tapped for domestic purposes, within the controlled and/or protected areas declared by appropriate authorities, which support wildlife and fishery activities).

11) Mangrove areas characterized by one or any combination of the following conditions: with any pristine and dense young growth, adjoining mouth of major river system near or adjacent to traditional productive fry or fishing grounds, areas which act as natural buffers against shore erosion, strong winds and storm floods, areas on which people are dependent for their livelihood.

12) Coral reefs characterized by one or any combination of the following conditions: with 50% and above live coralline cover, spawning and nursery grounds for fish, act as natural breakwater of coastlines.

The DENR has authority to include the project in the Environmental Impact Assessment System additional non-environmentally critical project (NECP), Group II types which may be located in Environmentally Critical Areas (ECA) having significant impacts in the environment. On the other hand, certain projects, regardless of location or project size due to negligible impacts arising from the method or technology and due to the mitigation of environmental issues and enhancement of the environment, may be excluded.

The technical standards and guidelines are confined mostly with flood control projects that generally consider lesser impoundment of water or Sabo dams of height less than 15 m and not involving 20 million m^3 or with inundation area of less than 25 ha. Thus, most of flood control and Sabo projects fall under Group II and Group III.

In the Philippines, flood control projects are implemented through foreign assistance or/and local funds. The former usually is a large scale project where impacts involve higher magnitude and significance which may require ECC as deemed by DENR. For locally funded projects which are of meager budget usually cover small area for emergency works to counter eventual disaster due to damages wrought by previous floods, collapse of banks or bank over flow where CNC is usually issued.

10.4.2 Scoping

If the project requires EIS or IEE at the least, the next step is scoping, which aims to identify the issues and impacts that are likely to be important and establish the terms of reference.

Scoping meetings are of many levels. First level or initial scoping is a meeting with the EMB and a member of a review committee. Succeeding meetings are done with the stakeholders, these are the affected people group or those in some way or another are affected by the project activities. Technical scoping follows after the stakeholder meeting. The result of the meetings sets the tone of the EIA where the Terms of Reference of the EIA or IEE are drawn out and determined.

Five sets of pro-forma letter of request for scoping are required with attached pro-forma project description, both downloadable from the EMB website.

A scoping meeting is optional for IEE; however, the proponent may decide to proceed with this meeting initially with the members of the Review Committee, the EMB and later with the representatives from different stakeholders directly and indirectly affected by the project. In case the proponent or the preparer decides not to have a scoping meeting, they may make a reasonable and sensible scope of the study.

10.4.3 EIA Study and Report Preparation

Under the procedural manual of DAO 30-03, the following governs the preparation of the documents required for the necessary permit:

For new projects: EIA-covered projects in Groups I, II and IV are required either an ***(1)*** *Environmental Impact Statement (EIS),* ***(2)*** *Programmatic EIS (PEIS),* ***(3)*** *Initial Environmental Examination Report (IEER) or* ***(4)*** *IEE Checklist (IEEC), depending on project type, location, magnitude of potential impacts and project threshold. For noncovered projects in Groups II and III, a* ***(5)*** *Project Description Report (PDR) is the appropriate document to secure a decision from DENR/EMB. The PDR is a "must" requirement for environmental enhancement and mitigation projects in both ECAs (Group II) and NECAs (Group III) to allow EMB to confirm the benign nature of proposed operations for eventual issuance of a Certificate of Non-Coverage (CNC). All other Group II projects with PDR-threshold level and all other Group III projects are both noncovered, thus, do not need to submit any EIA report or secure any decision document from DENR EMB. However, a PDR may be submitted at the option of the Proponent should the Proponent need a CNC for its own purposes, e.g. financing pre-requisite. For Group V projects, a PDR is required to ensure new processes/technologies or any new unlisted project does not pose harm to the environment. The Group V PDR is a basis for either issuance of a CNC or classification of the project into its proper project group*

For operating projects with previous ECCs but planning or applying for clearance to modify/expand or re-start operations, or for projects operating without an ECC but applying to secure one to comply with PD 1586 regulations, the appropriate document is not an EIS but an EIA Report incorporating the project's environmental performance and its current Environmental Management Plan. This report is either an ***(6)*** *Environmental Performance Report and Management Plan (EPRMP) for single project applications or a* ***(7)*** *Programmatic EPRMP (PEPRMP) for co-located project applications. However, for small project modifications, an updating of the project description or the Environmental Management Plan with the use of the proponent's historical performance and monitoring records may suffice.*[2]

<u>EIA Report</u>

Primary data are necessary for the conduct of EIA and supplemented by secondary data. Mandatory primary baseline information is required on the locality and influenced area of the project. Preparation may take 6 months or more depending on the extent of the study.

Common contents are the following:

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[2] Page 7, Revised Procedural Manual for DENR Administrative Procedural Manual for DENR Administrative Order No. 30 Series of 2003 (DAO 03-30)

The EIA Study covers the proposed project and its alternatives, the project environment, impact identification and prediction, evaluation of impact significance, impact mitigation, Environmental Management and Monitoring Plan.

1) Executive Summary

   Brief description of Project Description, Methodology (scope and duration), Project Setting, Summary of Scoping Report, Summary of Matrix of Major Impacts, Mitigation Measures and Environmental Management Plan, Summary Matrix of Environmental Monitoring Plan, Summary Presentation of the EIA Processes and Process Documentation

2) Introduction (1/2 page) - the purpose of the report, extent of the EIA study and brief description of any special techniques or methods used.

3) Project Description - the type of and need for project, location, size or magnitude of operation and proposed schedule for implementation.

   Basic Project Information, Project Location, Project Rationale, scale and duration, Alternatives, Description of Project Phases and components (Pre-Construction/Operational Phase, Construction, Operational and Abandonment Phases), resource requirements, manpower complement, estimate of waste generation from the most critical project activities and environmental aspects, project cost.

4) Description of the Environment - the physical and ecological resources, human and economic development and quality of life values in the area affected by the project. Environmental standards will be used as the baseline for comparative purposes.

   Baseline Environmental Description (land, water, air and people), focused on the sectors and resources most significantly affected by the proposed action

5) Alternatives - For each alternative, a summary of the probable adverse impacts in relation to the project, and other alternatives in comparison with the project whether it minimizes the environmental impact over all other alternatives and is within acceptable environmental impact limits. In most cases, environmental impacts "with" and "without" project alternatives should be examined.

   Impact Identification, Impact Prediction and Evaluation, Future Environment with and without the project.

6) Anticipated Environmental Impacts and Mitigation Measures - both direct and indirect, on different environmental resources or values due to project location, as related to design, during construction and regular operation will be discussed and mitigation, offsetting or enhancement measures will be recommended.

Impact Assessment, focused on significant environmental impacts (in relation to pre-construction, construction/development, operation and decommissioning stages), taking into account cumulative, unavoidable and residual impacts;

7) Economic Assessment -: (a) costs and benefits of environmental impacts; (b) costs, benefits and cost effectiveness of mitigation measures; and (c) for environmental impacts that have not been expressed in monetary values, a discussion of such impacts, if possible, in quantitative terms (e.g. weight or volume estimates of pollutants). This information should be integrated into the overall economic analysis of the project.

8) Environmental Management Plan - describe the impacts to be mitigated, and activities to implement the mitigation measures, including how, when, and where they will be implemented. The environmental monitoring plan will describe the impacts to be monitored, and when and where monitoring activities will be carried out, and who will carry them out.

   Mitigation/Enhancement Measures/Plan (Impacts/Risks)> (Construction Contractor's Environmental Program, Social Development Program – resettlement/relocation/livelihood- Contingency Response Plan, Abandonment Plan) Environmental Monitoring Action Plan, Institutional Plan, Information Education Communication Plan, Cost Estimate/Viability.

9) Public Consultation and Disclosure - describe the process undertaken to involve the public in project design and recommended measures for continuing public participation; summarize major comments received from beneficiaries, local officials, community leaders, NGOs, and others, and describe how these comments were addressed; list milestones in public involvement such as dates, attendance, and topics of public meetings; list recipients of this document and other project related documents; describe compliance with relevant regulatory requirements for public participation; and summarize other related materials or activities, such as press releases and notifications. This section will provide of summary of information disclosed to date and procedures for future disclosure.

10) Proposal for an Environmental Monitoring and Guarantee Fund (if required)

11) Conclusions - describe the gains which justify implementation of the project; explain how significant adverse environmental impacts will be mitigated or offset and compensated for; explain/justify use of any irreplaceable resources and; describe follow-up surveillance and monitoring.

Initial Environmental Examination (IEE)

Group III projects and Group II projects in Environmentally Critical Area (ECA) require Initial Environmental Examination (IEE). Data or information may be taken from secondary sources or previous studies relevant to the project; otherwise, the preparer may be required to have primary data. Preparation of the IEE may take three to six months depending on the extent of the study. Accountability statement by the preparer and the proponent shall be duly signed and included in the documents. The format shall follow the existing procedural manual of DAO 03-30 as shown below.

ANNEX 2-15 INITIAL ENVIRONMENTAL EXAMINATION REPORT (IEER) OUTLINE

IEER OUTLINE

(maximum of about 75 pages)

NOTE: REFER TO ANNEX 2-7a (EIS SCOPING AND PROCEDURAL SCREENING CHECKLIST) AS BASIS FOR DETERMINING SIMILAR OR EQUIVALENT SPECIFIC CONTENTS/REQUIREMENTS OF EACH SECTION

Project Fact Sheet

Table of Contents

Executive Summary

1) Brief Project Description
2) Brief Summary of Project's IEE Process
3) Summary of Baseline Characterization
4) Summary of Impact Assessment and Environmental Management Plan
5) Summary of Environmental Monitoring Plan

DRAFT MAIN IEER

1. BASIC PROJECT INFORMATION
2. DESCRIPTION OF THE PROJECT'S IEE PROCESS -
2.1. Terms of Reference of the IEE Study (if scoping was done with EMB)
2.2. IEE Team
2.3. IEE Study Schedule
2.4. IEE Study Area
2.5. IEE Methodology
3. PROJECT DESCRIPTION
3.1. Project Location, Area, ECA category (if applicable)
3.2. Project Rationale
3.3. Project Development Plan, Process/Technology and Project Components
3.4 Description of Project Phases (Activities/Environmental Aspects, Associated Wastes and Built-in Pollution Control Measures)
3.4.1. Pre-construction/ Pre-operational phase
3.4.2. Construction/Development phase
3.4.3. Operational phase
3.4.4. Abandonment phase
3.5. Manpower Requirements
3.6. Project Cost
3.7. Project Duration and Schedule
4. BASELINE ENVIRONMENTAL CONDITIONS, IMPACT ASSESSMENT AND MITIGATION
4.1. The Land (Discuss only relevant modules or modules likely to be affected by project impacts) - Land use, Petrology, Geology, Geomorphology, Terrestrial biology
4.2. The Water (Discuss only relevant modules or modules likely to be affected by project impacts – Hydrology, Oceanography, Water Quality, Freshwater Biology, Marine Biology
4.3. The Air - ((Discuss only relevant modules or modules likely to be affected by project impacts - Meteorology, Air Quality, Noise
4.4. The People
5. ENVIRONMENTAL MANAGEMENT PLAN
5.1. Impacts Management Plan
5.2. Emergency Response Policy and Generic Guidelines (if applicable)
5.3. Environmental Monitoring Plan

Note: Attach under this section the filled out Project Environmental Monitoring and Audit Prioritization Scheme (PEMAPS) Questionnaire in Annex 2-7d of the RPM

5.4 Institutional Plan for EMP Implementation

6. BIBLIOGRAPHY/REFERENCES
7. ANNEXES
7.1. Scoping Checklist (optional, since Scoping is not required for IEEs)
7.2. Original Sworn Accountability Statement of Proponent
7.3. Original Sworn Accountability Statement of IEE Preparer
7.4. Baseline Study Support Information

Project Description Report

The project description report is the simplest form required to obtain a Non-Certificate of Coverage (CNC) for projects deemed as Group II, classified under enhancement /mitigation projects, Group III projects and Group V, unclassified projects, require Project Description by EMB to evaluate the appropriate group they belong. Content of Project Description includes location, scale and duration, rationale, alternatives, phases and components, resource requirements, manpower complement, estimate of waste generation from the most critical activities and environmental aspects, and project cost. The format shall follow the existing procedural manual of DAO 03-30 as shown below.

ANNEX 2-16 of the Procedural Manual of DAO-03-30

PROJECT DESCRIPTION REPORT (PDR) OUTLINE(maximum of about 30 pages)

PURPOSES: GRP II and III ENHANCEMENT/MITIGATION PROJECTS – PD REPORT REQUIRED; ALL OTHER GRP II and III PROJECTS – PD REPORT OPTIONAL; GRP V UNCLASSIFIED/ UNLISTED/NEW TECHNOLOGY PROJECT –PD REPORT REQUIRED

Table of Contents (1 page)

1. BASIC INFORMATION ON PROJECT and PROPONENT (1 page)
2. PROJECT DESCRIPTION (15 pages)
2.1. Project Location and Area (at the minimum, shown in an official NAMRIA topographic or nautical map (whichever type is applicable and of appropriate scale); Show title, legend, scale, project location and political boundaries (from site/barangay to region); indicate any known ECA category encompassing the project area
2.2. Project Rationale – state need for & purpose of the project., particularly environmental enhancement or mitigation purpose of the project
2.3. Project Development Plan, Process and Components - Attach tentative/option of Physical Plan/Site Development Map being considered at the FS stage; briefly describe process/technology; list/describe and indicate project components (facilities/infrastructures, other single projects supporting the main project) on the topographic map
2.2 Description of Project Phases - For Group II and III non-covered projects: focus on activities and processes which may cause residual impacts; For Unclassified/Unlisted/New Technology Projects: focus on critical activities and processes per phase which place a demand on local resource uses and which generate emissions, effluent, hazardous waste, solid waste, other wastes)
2.3.1. Pre-Construction/ Pre-Development phase
2.3.2. Construction/Development phase
2.3.3. Operational phase – For Unclassified/Unlisted/New Technology Projects: Specifically present if processes and substances to be used are listed and fall within the limits covered by Environmental Risk Assessment as enumerated in Section C of Annex 2-7a of the Revised Procedural Manual)
2.3.4. Abandonment phase
2.4. Project Emissions/Effluent/ Hazardous Waste/Solid Waste/Other Wastes – Present integrated summary of types of wastes (residual for Group III non-covered projects) ; estimate waste generation rate; identify built-in waste management measures and facilities planned or committed to be built into the project design
2.5. Manpower - Present manpower requirements per project phase; specify expertise needed; nature & estimated number of jobs available for men; nature and number of jobs available for women; specify strategy and tentative scheme for sourcing locally from host and neighboring LGUs and those from outside
2.6. Project Cost
2.7. Project Duration and Schedule

3. OVERVIEW/GENERIC DESCRIPTION OF THE BASELINE ENVIRONMENT (4 pages – on land, water, air, people) – focus on the environmental components and factors likely to be affected by the project's impacts; only secondary data or qualitative environmental description is necessary
4. ENVIRONMENTAL MANAGEMENT PLAN (3 pages - focused only on the residual management scheme on the relevant land, water air & people module)
5. ANNEXES (3-6 pages)
5.1 Original Sworn Accountability Statement of Proponent (Use Annex 2-21 of RPM)
5.2 Photos or plates of proposed project site, cumulative/residual impact areas and surrounding communities (N, S, E, W of the project; key sectoral features - land, water, air, people)

Programmatic Environmental Impact Statement (PEIS)

Programmatic Environmental Impact Statement (PEIS) is required for project under Group IV, co-located projects. The report details comprehensive studies of environmental impacts of co-located project in contiguous area and its carrying capacity to absorb the impacts.

Programmatic Environmental Performance Report and Management Plan (PEPRMP)

This document is required for small project modifications. Modification and updating of project description and the environmental management plan based on the historical performance and monitoring records will suffice.

10.5 EIA Report Review and Evaluation

The EMB designates EIA review committee, which are third party experts for environmentally critical projects, the Technical Committee from EMB internal specialists for environmentally non-critical projects. The requirements specified during scoping after a substantive review with the EMB and the review committee will be evaluated by EMBs through the EIARC recommendations and the public's inputs during public consultations/hearings before recommending a decision on the application. EIARC recommendations, including those on issues outside the mandate of the EMB, are signed by the EIARC Chair.

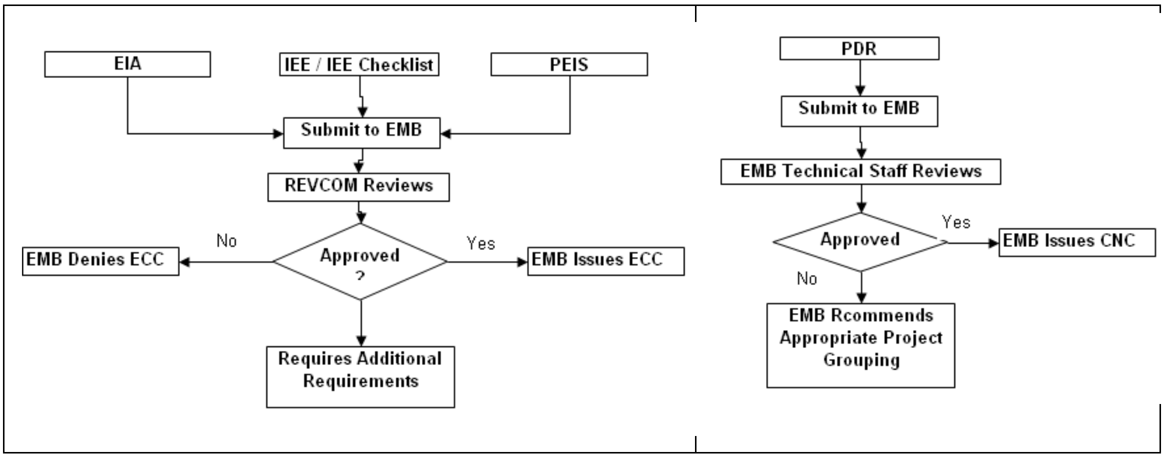

Figure 10.5: Submission to Approval Procedure

10.6 Decision Making

After the review of the documents, EIA recommendations and the draft decision document, result to the issuance of an ECC, CNC or Denial Letter. When approved, a covered project is issued Environmental Compliance Commitment (ECC) while a non-covered project is issued a Certificate of Non-Coverage (CNC). Endorsing and deciding authorities are designated by AO 42, and detailed in the DAO 03-30 procedural manual for every report type. The Proponent signs a sworn statement of full responsibility on implementation of its commitments prior to the release of the ECC. The ECC is then transmitted to concerned Local Government Units and other Government Agencies for integration into their decision-making process.

10.7 Monitoring, Validation and Evaluation/Audit

Monitoring, Validation and Evaluation/Audit stage assesses performance of the Proponent against the ECC and its commitments in the Environmental Management and Monitoring Plans to ensure actual impacts of the project are adequately prevented or mitigated.

For detailed discussion on the Philippine EIA System, please refer to DAO 03-30 procedural manual, available at the Environmental Management Bureau or from its web site.

Chapter 11 RESETTLEMENT ACTION PLAN

Projects dislocate or affect people directly or indirectly due to right or way, land acquisition, mobilization, and other activities during construction, operation, maintenance or decommissioning or abandonment. The policy framework for the Resettlement Action Plans for structures and land are derived Constitution, laws, policies and guidelines related to operation and implementation of resettlement and indigenous peoples, which are discussed below.

11.1 Basic National Policy

Article III, Section 9: "Private property shall not be taken for public use without just compensation."

Article XII, Section 5: "The state shall protect the rights of indigenous cultural communities to their ancestral lands to ensure their economic, social, and cultural well-being.

10.2 Republic Act (RA) 8974 –

An Act to Facilitate the Acquisition of Right-Of-Way (ROW), Site or Location for National Government Infrastructure Projects discuss the following:

1) Bases for land valuation for the modes of acquisition, negotiated sale and expropriation.

2) The law states that the implementing agency shall negotiate with the owner for the purchase of the property by offering first the current zonal value issued by the Bureau of Internal Revenue for the area where the private property is located.

3) The valuation of the improvement and/or structures on the land to be acquired shall be based on the replacement cost which is defined as the amount necessary to replace the structure or improvement according to the current market price for materials, equipment, labor, contractor's profit and overhead, and all other attendant costs associated with the acquisition and installation in place of the affected improvement/installation.

4) Acquiring rights, title or ownership of private property particularly real estate, to use for another purpose is through, donation, quit claim, exchange barter, negotiated sale or purchase, expropriation and other modes authorized by law.

5) Zonal Value. In case the mode of acquisition is through a negotiated sale, the first offer shall be the zonal value, issued by the Bureau of Internal Revenue, of the particular land where the property is located. In case the owner rejects the first offer, the Department shall renegotiate using the values recommended by the Appraisal Committee or Independent Land Appraiser.

6) Standards to determine market value. Negotiated sale between the DPWH and the People Affected Family (PAF) based on the following:

a) The classification and use for which the property is suited;

b) The development costs for improving the land;

c) The value declared by the owners;

d) The current selling price of similar lands in the vicinity;

e) The reasonable disturbance compensation for the removal and/or demolition of certain improvement on the land and for the value for improvements thereon;

f) The size, shape and location, tax declaration and zonal valuations of the land;

g) The price of the land as manifested in the ocular findings, oral as well as documentary evidence presented, and

h) Facts and events to enable the affected property owners to have sufficient funds to acquire similarly-situated lands of approximate areas to rehabilitate themselves as early as possible

7) Quit Claim. This is required to be executed by owners of lands acquired under the Public Land Act because of the reservation made in issuance of parents of titles thereto. In other words, even if the title or free patent describes the whole area as owned by the patentee or title holders, by operation of the law, a strip of twenty or sixty meters, as the case maybe, of that area as described is not absolutely owned by him, because it is reserved by the government for public use.

8) In case the Project Affected Persons (PAPs)/ People Affected Families (FAPs) are qualified for compensation but with arrears on land tax. To facilitate the processing of payment on land acquired from the PAPs with tax arrears the DHPW will pay the arrears and deduct the amount to the total compensation cost.

9) In case the PAPs/PAFs are qualified but already dead and the heirs have not undergone extra-judicial partition, the PAPs/PAFs will be given a grace period to meet the requirement within the validity period of allotment for two (2) years. If beyond the two years that PAPs cannot comply with the requirement, they have to settle the case in court.

11.2 Expropriation

1) For Structures: In the event that the PAF rejects the compensation for structures at replacement cost offered by DPWH, the DPWH or the PAF may take the matter to court. When court cases are resorted to either by DPWH through expropriation or by the PAFs through legal complaints, the DPWH will deposit with the court in escrow the whole amount of the replacement cost (100%) it is offering the owner for his/her assets as compensation to allow DPWH to proceed with the works. The PAF will receive the replacement cost of the assets within one (1) month following the receipt of the decision of the court.

2) For Land: If the owner contests the DPWH's second offered value for compensation

for land, the PAF or the DPWH may take the matter to court. DPWH shall immediately pay the owner: a) 100% of the value of the property based on the BIR zonal valuation, and b) the value of improvement and structures. However, if the owner rejects the full payment, the DPWH will deposit 100% of the BIR zonal value in an escrow account. The court shall determine the just compensation within sixty (60) days, taking into account the standards for the assessment of the value of the land (Sec. 5, RA 8974).

11.3 Indigenous Peoples' Rights Act (IPRA) of 1997

The IPRA sets the conditions, requirements, and safeguards for plans, programs, and projects affecting Indigenous Peoples. It spells out and protects the rights of Indigenous Peoples. The important provisions of the IPRA are:

1) The right of their ancestral domains.
2) The right to an informed and intelligent participation in the formulation and implementation of any project, government or private will impact upon their ancestral domains;
3) The right to participate fully, if they so choose, at all levels of decision-making in matters which may affect their rights, lives and destinies through procedures determined by them;
4) The right to receive just and fair compensation for any damages inflicted by or as a result of any project, government or private;
5) The right to stay in their territory and not to be removed from that territory through any means other than eminent domain. If relocation is necessary as an exceptional measure, it can only take place with the free and prior informed consent of the IPs and ICCs concerned,
6) The right to be secure in the lands to which they have been resettled;
7) The right to determine and decide their own priorities for the lands they own occupy, or use,
8) The right to maintain, protect, and have access to their religious and cultural sites;
9) The IPRA also created the National Commission on Indigenous Peoples (NCIP) to carry out the policies set forth in the IPRA. The NCIP has issued a number of orders that puts into operation the provisions of the IPRA; the most important for the purpose of this policy in NCIAP Administrative Order No. 1 or the Free and Prior Informed Consent (FPIC) Guidelines of 2006.

11.4 Other Applicable Laws and Policies (Executive Orders, Administrative Orders, and Department Orders.)

1) PD 1067 Philippine Water Code,

Utilization of Water, Article 51

The banks of rivers and streams and the shores f the seas and lakes throughout their entire length and within a zone of three (3) meters in urban areas, twenty (20) meters in agricultural areas and forty (40) meters in forest areas, along their margins, are subject o the easement of public use in the interest of recreation, navigation, flotage, fishing and salvage. No person is allowed to stay in this zone longer than what is necessary for recreation, navigation, flotage, fishing or salvage or to build structures of any kind.

Control of Waters, Article, 53 to 58

a) To promote the best interest and coordinated protection of flood plain lands, the Secretary of Public Works… may declare flood control areas and promulgate guidelines for governing flood plain management plan in these areas

b) In declared flood control areas, rules and regulations may be promulgated to prohibit or control activities that may damage or cause deterioration of lakes and dikes, obstruct the flow of water, change the natural flow of the river, increase flood losses or aggravate flood problems

c) The government may construct the necessary flood control structures in the declared flood control areas, and for this purpose it shall have a legal easement as wide as may be needed along and adjacent to the river bank and outside the bed or channel of the river.

d) River beds, sand bars and tidal flats may not be cultivated except upon prior permission from the Secretary of Department of Public Works, Transportation and Communication and such permission shall not be granted where such cultivation obstructs the flow of water or increase flood levels so as to cause damage to other areas.

e) Any person may erect levees or revetments to protect his property from flood, encroachment by the river or change in the course of the river, provided that such construction does not cause damage to the property of another.

f) When a river or stream suddenly changes its course to traverse private lands, the owners or the affected lands may not compel the government to restore the river to its former bed; nor can they restrain the government from taking steps to revert the river or stream to its former course. The owners of the land thus affected are not entitled to compensation for any damage sustained thereby. However, the former owners of the new bed shall be the owners of abandoned bed proportion to the area lost by each.

2) PD. 813 October 17, 1975, Section 29**,**Laguna Lake or Lake. Whenever Laguna Lake or lake is used in this Act, the same shall refer to Laguna de Bay which is that area covered by the lake waters, when it is at the average annual maximum lake level

of elevation 12.50 meters, as referred to a datum 10.00 meters below mean lower low water (M.L.L.W.). Lands located at and below such elevation are public lands which form part of the bed of said lake."

3) Commonwealth Act 141 Section 112 or Public Land Act – prescribes a twenty (20) meter strip of land reserved by the government for public use, with damages paid for improvements only.

4) Presidential Decree 635 amended Section 112 or CA 141 increasing the width of reserved strip of twenty (20) meters to sixty (60) meters.

5) EO 113 (1995) and EO 621 (1980)

   a) National Roads shall have a ROW width of at least 20 meters in rural areas, which may be reduced to 15 meters in highly urbanized areas.

   b) ROW shall be at least 60 meters in unpatented public land,

   c) ROW shall be at least 120 meters through natural forested areas of aesthetic or scientific value.

6) EO 1035

   a) Financial assistance to displaced tenants, cultural minorities and settlers equivalent to the average annual gross harvest for the last 3 years and not less than PhP 15,000 per ha.

   b) Disturbance compensation to agricultural lessees equivalent to 5 times the average gross harvest during the last 5 years.

   c) Compensation for improvements on land acquired under Commonwealth Act 141.

   d) Government has the power to expropriate in case agreement is not reached.

7) MO 65, Series of 1983

   a) Easement of ROW where the owner is paid the land value for the Government to use the land but the owner still retains ownership over the land.

   b) Quit claim where the Government has the right to acquire a 20 to 60 m width of the land acquired through CA 141. Only improvements will be compensated.

8) Republic Act 6389

   Provides for disturbance compensation to agricultural lessees equivalent to 5 times the average gross harvest in the last 5 years.

9) Article 141, Civil Code

   Real actions over immovable prescribe after (30 years. This provision is without prejudice to what is established for the acquisition of ownership and other real rights by prescription (1963).

10) NCIP Administrative Order No. 1, Series of 2006 or the Free and Prior Informed Consent Guidelines of 2006.

The Free and Prior Informed Consent Guidelines of 2006 spells out the procedure for obtaining the Free and Prior Informed Consent for affected communities. It details the process for conducting Field Based Investigation (FBI) and obtaining Certification Precondition from the NCIP attesting that the applicant has complied with the requirements for securing the affected ICC/IP's FPIC. It also provides the procedure for validating projects solicited/initiated by Indigenous Peoples.

The Lina Law, otherwise known as Republic Act No. 7279 or the Urban Development Housing Act of 1992 (UDHA), provides that certain lands owned by the government may be disposed of or utilized for socialized housing purposes. It was signed into law to address the housing shortage of the country.

The following diagrams summarize the procedure for right of way acquisition during feasibility study and detail design.

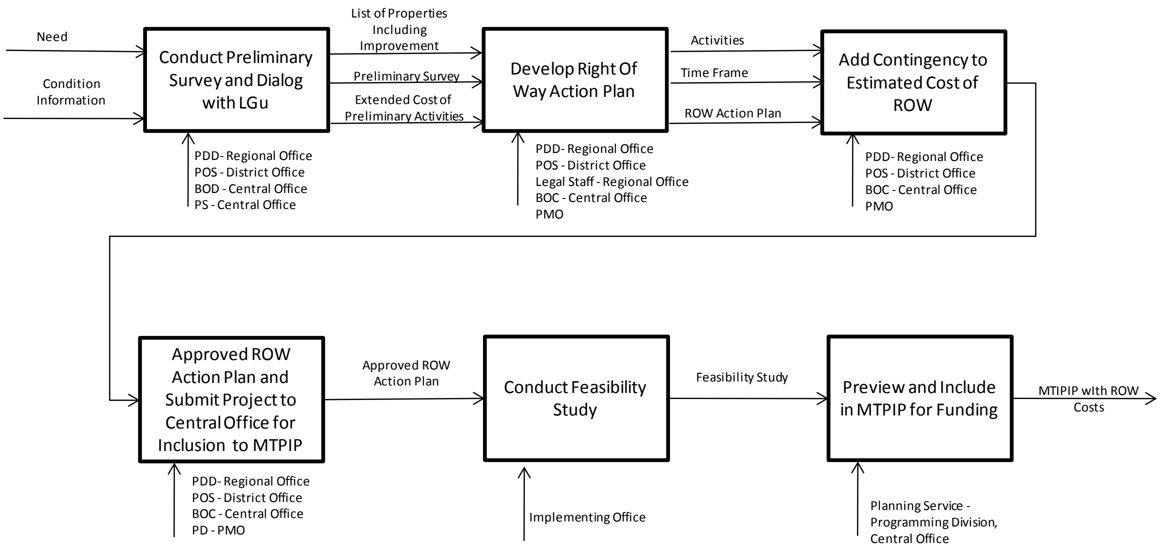

Figure 11.4a: Land Acquisition And Infrastructure Right of Way, Project Identification, Feasibility Study.

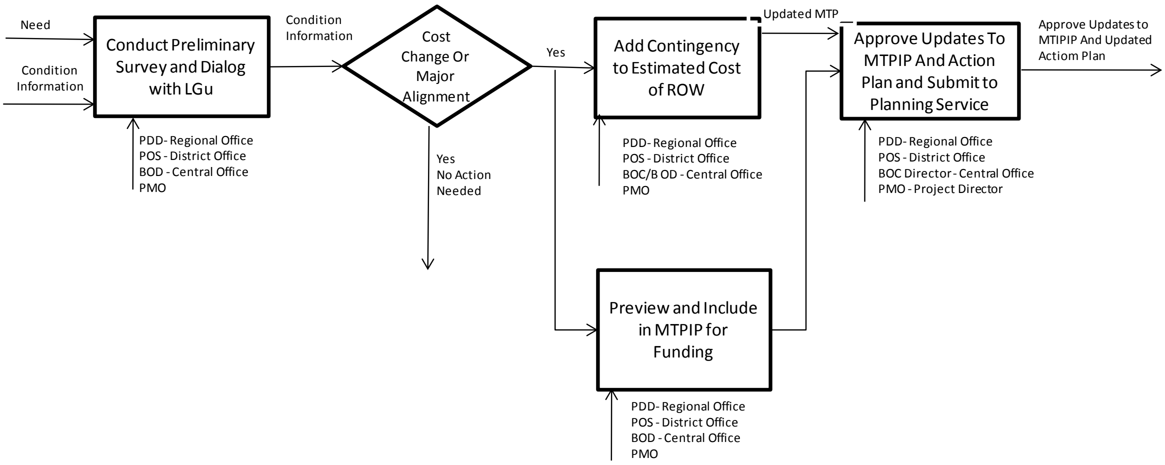

Figure 11.4b: Land Acquisition And Infrastructure Right of Way, Detail Design.

Chapter 12 Gender Consideration

Flood management projects shall consider gender or universal design, which take care of provisions for the people. Gender concerns are recognized in policies and strategies in infrastructural development.

It has been recognized that understanding the different living conditions and needs of women and men at project planning stage and taking them into account in the design and implementation of projects is important.

Gender and policies include the following;

1) Women and men must equally participate in climate change, disaster risk reduction, decision-making processes and other government programs at community, regional and nationwide levels;

2) Integration of gender-sensitive criteria into planning, design implementation, monitoring and evaluation of programs, projects and initiative; and

3) Allocation of adequate resources to address the needs of women, for example funding appropriate and environmentally sound technologies and supporting women's grassroots initiative in sustainable use of natural resources.

The implementation of such-policies, the study shall include the activities but not be limited to the following;

1) Undertake environmental planning through public consultation or multi-stakeholders forum and identify gender issues and concerns in the involvement of women, youth, senior citizens and disabled persons in infrastructure development. Women should constitute at least 30%of the total participants.

2) Develop gender-based information within the influence area of the proposed project.

3) Conduct social gender analysis such as trend of employment of women at all levels (actual construction, technical and management) in infrastructure projects or services, capacity of women to influence decisions about the planning design, operation and maintenance of infrastructure facilities; resettlement of women and their families as a result of the construction of infrastructure; access of women to water, health and transport services, etc. It is noted that the involvement of women in infrastructure development is very limited.

4) Identify appropriate sites for public restrooms along the whole stretch of the flood control project and recommend Operation and Maintenance (O/M) measures for these restrooms.

5) Identify appropriate sites for children's access, guardrails, footbridges, and other safety facilities and structures.

6) Prepare standard gender-sensitive design of infrastructure and facilities that caters the needs of women, aged people and children, such as wider space on restrooms for women, provision of ladders in the abutments of bridges and dikes, etc.

7) Incorporate in the plan of such gender-sensitive structure/facilities in the study and the cost in the economic evaluation.

Chapter 13 Climate Change Consideration

Uncertainties in the weather pattern and rise in temperature are attributed to climate change Trends in temperature and rainfall frequency and intensity are changing, however precise prediction or forecast cannot be established at present as there are many uncertainties. According to PAGASA, the annual mean temperature in the Philippines is expected to increase by about 0.90 °C to 1.1 °C for 2020 and 1.9 °C to 2.2 °C by 2050, respectively. Rainfall and discharge data throughout the country are not sufficient and some storm events exceed the return period commonly used for planning and design.

Related phenomena include extreme weather conditions and sea level rise. These changes affect the planning of different infrastructure including flood control projects.

Design safety level of flood control structures are lowered as a result of climate change scenario. For instance, a 10-year design scale would decline to 6-year return period in 2050. Sea level rise requires configuration of river walls and other flood control structures influenced by tidal fluctuations and storm surge. For more information, see the Study on Comprehensive Flood Mitigation For Cavite Lowland Area in the Republic of the Philippines, 2009.

To compensate for the impacts, adaptation requires structural and non-structural measures, which are considered as basin wide approach.

For the structural measures the following can be considered.

1) Adaptation against flood overflow from rivers
2) Reservation of Flood Detention for Flood
3) Adaptation against sea level rise and storm surge
4) Adaptation against inland flood

For non-structural measures the following can be considered:

1) Control of Excessive Land Development
2) Control of Encroachment into river area
3) Establishment of flood warning and evacuation system.

In the future, a separate manual on climate change for flood control will be formulated under the on-going Disaster Risk Management Study for Flood.

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.

ALLUVIAL PLAIN - General name for a plain produced by the deposition of alluvium from the action of rivers; e.g. flood plain, delta plain, alluvial fan, etc.

ALLUVIAL SEGMENT - A river segment wherein the bed materials are generally made up of sand or clay washed down together by the flowing of water from the upstream portion (mountain area) where a river system originates.

ANNUAL MAXIMUM FLOOD DISCHARGE - The highest momentary peak discharge in a year.

APRON - A floor or lining of concrete, gabion, or other resistant materials provided at the toe of the dam, spillway, groundsill, etc. to protect from local scouring caused by falling water or turbulent flow.

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.

BARRAGE - A weir equipped with series of sluice gates to regulate the water elevation at its upstream side.

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

BED ARMORING –refers to the coarse surface layers on a bed layer.

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.

BERM - A horizontal step or landing in a revetment/dike to cut the continuity of an otherwise long slope for stabilizing the structure itself and for maintenance purposes.

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.

CONCAVE BEND (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.

CONTROL POINT - In a river, the place or location of observation point where the planned discharge is observed and fixed.

CONVEX BEND (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.

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.

DRIFT STREAM PART - A portion of a river, usually at outer bends where the riverbed has become deep.

DUNE - A ridge of piled up sand.

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

EXISTING DISCHARGE CAPACITY - Full discharge capacity of a waterway, usually a river before any improvement works take place.

FLOOD LOSS, ANNUAL - Is the average of damages caused by flood over a considerable period of time. It is taken to be the actual cost of the flood risk. Since flood losses are intermittent and uncertain, the annual evaluation of a loss must be based upon some expression of the probabilities of occurrence.

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.

FRESHET - The sudden rise or overflow of water in a stream, brought on by melting snow or a heavy rain; the flow of fresh water into the sea.

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.

GULLY EROSION - Erosion on the dike slopes caused by heavy rainfall.

HOLE, WEEP - An opening provided in the revetment, retaining walls, catch walls, etc., to permit drainage of water collected behind such structures to eliminate and/or reduce residual hydraulic pressure.

IMPLEMENTING PLAN - A specific plan for project execution. This plan includes the channel plan, its cross section and longitudinal profile, structural drawings, cost estimates and implementing schedule.

INNER BEND - SEE ***CONVEX BEND***

LEVEL OF PLANNED DISCHARGE - The planned discharge capacity (of a river) to be finally adopted based on the viability of the project.

LEVEL, MAXIMUM FLOOD - The highest recorded flood level.

LONG-TERM TARGET DISCHARGE - The ideal maximum discharge capacity of a river system corresponding to the flood frequency (50-years, 100-years return period, etc.) used in calculation.

MAINTENANCE PATHWAY - A service and/or maintenance road for maintenance activities of a dike or revetment.

MASTER PLAN – The overall description of the project area. Sometimes 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.

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.

NON-DIKED RIVER - SEE ***RIVER, NON-DIKED***

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 – SEE ***CONCAVE BEND***

POLYGONAL FORM - One of the plane forms of the Groundsill structure wherein its vertex meets at the center of the river.

RESIDUAL HYDRAULIC PRESSURE - (1) An overturning action of water behind the revetment. This is caused by the sudden drop of water elevation at the riverside due to tidal variation;(2) Pressure exerted by the ground water behind the revetment. This pressure is developed when there is a big disparity in elevation between the subsiding floodwater in the river and the groundwater stages, due to the absence of drainage pipes/weep holes, like revetment structure.

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 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.

RIPPLE - A small wave spreading outward from a point where the surface of water is disturbed.

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 - River of natural bank wherein no improvement (like dike) has been introduced.

RUN-OFF ANALYSIS - Calculation of discharge.

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 underground, 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

WATER DEPTH OF ANNUAL MAXIMUM FLOOD – Floodwater depth of a river in an average 1 to 2 year return period.

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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