DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS
JAPAN INTERNATIONAL COOPERATION AGENCY

TECHNICAL STANDARDS AND GUIDELINES FOR DESIGN OF FLOOD CONTROL STRUCTURES

#

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Project for the Strengthening of Flood Management Function of the DPWH

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Chapter 1 DIKE

1.1 Basic concept

A dike is an embankment or levee constructed along the banks of a stream, river, lake or other body of water for the purpose of protecting the landside from overflowing floodwater by confining the stream flow in the regular channel. River improvement should be planned with non-diked river if possible to have efficient drainage conveyance.

Flood level should be considered in choosing flood control measures. If the calculated design flood level is higher than the surrounding areas, dike has to be planned. As the dike prevents drainage water from the inland to flow naturally into the river; inland drainage improvement (non-dike system) is provided to address inland flooding. The height of the dike is designed based on the calculated design flood level, which is not fixed by the level of the surrounding areas. But the ground height should be considered in setting the flood level.

In most cases, non-diked river is preferable because of ease in maintenance and safety because breaching is unlikely compared to the diked river.

Dike is sometimes difficult to implement due to land acquisition problem (right of way). Moreover, the existence of important facilities, ports and harbors, etc. hampers the construction/implementation of said project. In such cases, concrete retaining wall type dike may be adopted.

In some cases, the dike is used as a roadway (cause way).

1.1.1. Materials

Dikes generally consist of soil and sand. The advantages of using earth materials are as follows:

1) Economical because of the availability of materials.
2) It will last for a long period of time.
3) It could be easily mixed with the ground materials.
4) It follows the ground deformation/settlement of foundation.
5) If the scale of flood control plan would be increased, it is easy to improve.
6) If the dike is damaged by flood, earthquake or other inevitable disasters, it is easy to restore.
7) For environmental consideration.

1.1.2 Causes of Dike Damages and Proposed Countermeasures.

Main causes of damage/breaching of dike and its countermeasures are as follows:

Causes of Damage	Countermeasures
Erosion (Scouring)	The surface of the dike on both sides shall be covered with vegetation for protection against erosion. The riverside should be protected with revetment, if necessary.
Overflow	Sand bagging for emergency measure. For long term measure, provide concrete and asphalt covering for the crest and the landside slope.
Seepage	To prevent the collapse of dike caused by seepage, embankment materials for the dike should consist of impervious materials (e.g. clay) in the riverside, and pervious materials in the inland side. Drainage structures and related facilities works should be provided at the inland side to drain accumulated water.
Earthquake	Immediately repair/restoration after the earthquake.

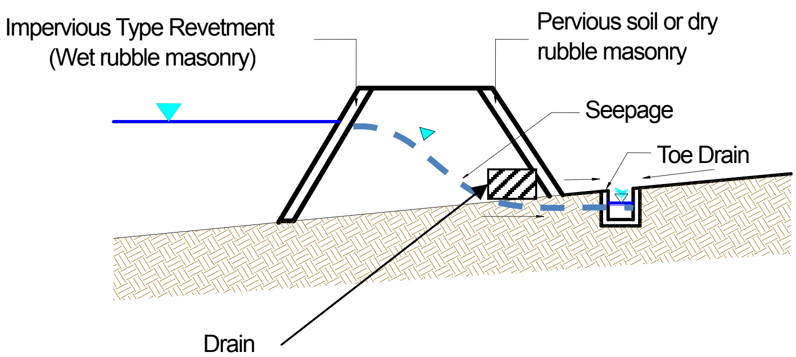


Figure 1.1.2 Example of Countermeasure Against Seepage

1.1.3 Design Consideration

Dike construction may include new dike and widening or raising existing dike.

1) Design of New Dike

The new dike shall be designed to protect the affected flood prone areas. In consideration of the stability of the structure, the dike alignment shall avoid unstable peat and muck, weak subsoil, and loose sand foundation to prevent settlement.

2) Heightening /Widening

Whenever there is a necessity to heighten/widen the dike on the landside or riverside, the position depends on the alignment; however, ideally the landside is preferred.

When there is a right of way problem or when there is adequate water way, widening may be applied on the riverside. However, when the toe of the dike is close to the low water channel in case of a compound cross section, it is suggested to avoid widening on the riverside even if there is sufficient river width.

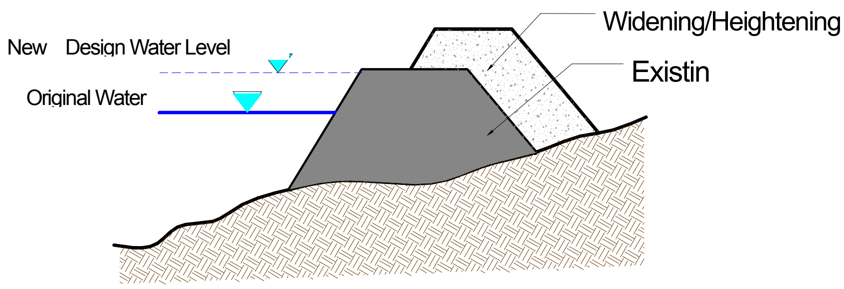


Figure 1.1.3 Widening and Heightening of Existing Dike

1.2 Alignment

The following shall be considered for the proposed alignment:

1) Reduction of the existing stream area shall be avoided as much as possible.
2) The alignment shall be as straight as possible. Sharp curves should be avoided since these portions are subject to direct attack of flow.
3) Where there is sufficient space, the dike should not be too close to the river channel or riverbanks to prevent undermining or scouring.
4) Valuable land, historical or religious structures, and weak/permeable foundation should be avoided.

1.3 Design Criteria

1.3.1 Parts of Dike

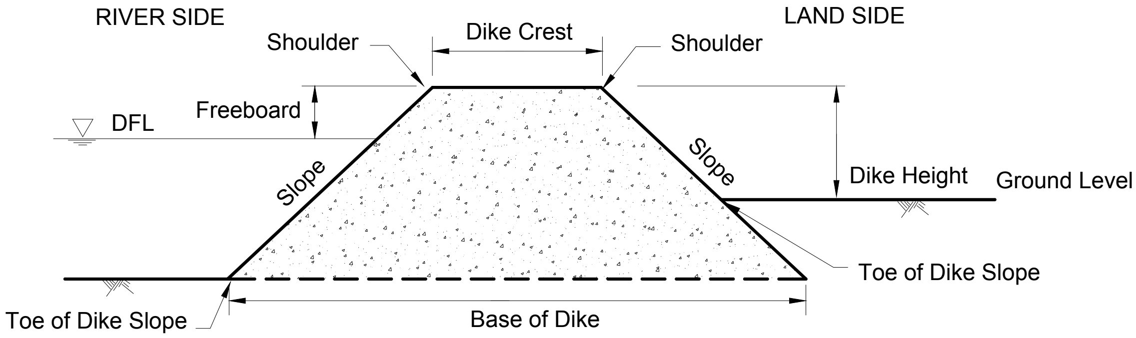
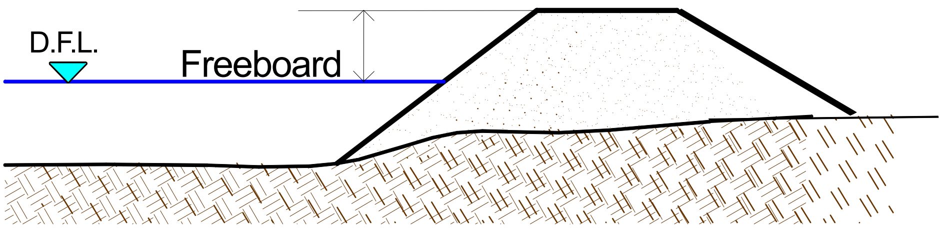


Figure 1.3.1 Parts of Dike

1.3.2 Height

The height of a dike shall be based on the design flood level plus the required freeboard. The calculated flow capacity shall be used as the Design Flood Discharge for establishing the freeboard.



Dike height = Design flood level + Freeboard

Figure 1.3.2 Dike Height

1.3.3 Freeboard

Freeboard is the margin from design flood level up to the elevation of the dike crest. It is the margin of the height which does not allow overflow. The freeboard shall be based on the design flood discharge which shall not be less than the value given in Table 1.3.3 .

Table 1.3.3 Minimum Required Freeboard

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

For the backwater effect in a tributary, the height of the dike in the transition stretch shall not be lower than that of the main river or even higher at the confluence in order to prevent inundation in the subject areas. In general, the dike's height of the main river at the confluence point is projected following its design flood level.

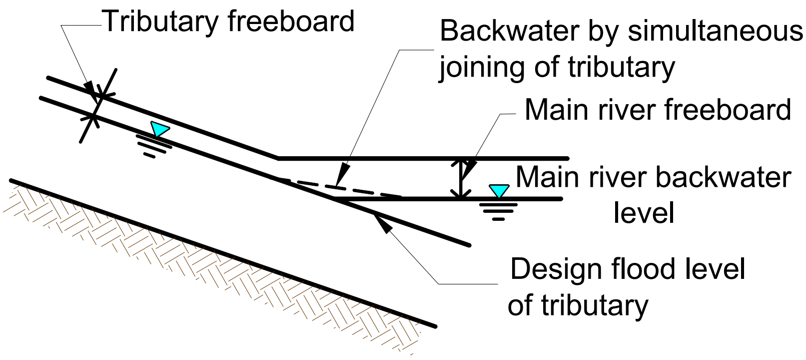


Figure 1.3.3a Free Board Due to Backwater Effect

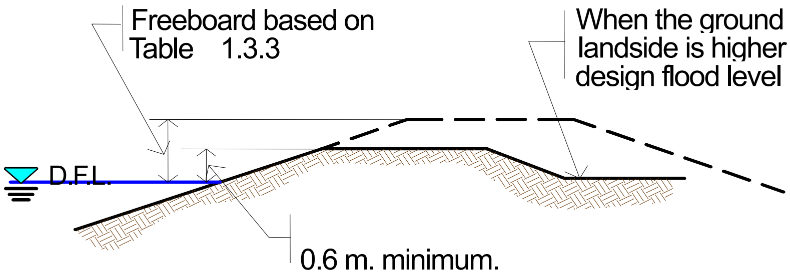


Figure 1.3.3b Freeboard when Landside is higher than design flood level

1.3.4 Crest Width

The crest width of the dike, especially for wide river, shall be based on the design flood discharge, and shall not be less than the values given in Table 1.3.4. When the landside ground level is higher than the design flood level, the crest width shall be 3 m or more regardless of the design flood discharge. Crest width shall be designed for multi-purpose use, such as for patrolling during floods and in the execution of emergency flood prevention works.

The base of the dike is fixed by the width of its crest and slope. Likewise, the dike shall be designed to prevent from possible collapse due to seepage which is also dependent on the width of the dike's crest.

Table 1.3.4 Crest Width of Dike

Design flood discharge, Q (m^3/sec)	Crest Width (m)
Less than 500	3
500 and up to 2,000	4
2,000 and up to 5,000	5
5,000 and up to 10,000	6
10,000 and over	7

For backwater effect on the affected tributary, the crest width of the dike shall be designed such that it is not narrower than the dike crest width of the main river.

1.3.5 Maintenance Road

The dike shall be provided with a maintenance road for patrolling the river during emergency flood prevention activity. When a permanent road is to be built and the difference in height between the dike crest and the landside is below 0.6 meter; maintenance road is no longer necessary. However, the dike's crest itself can be used as a maintenance road. The maintenance road shall be 3.0 meters or more.

1.3.6 Slope

In principle, the slope of the dike shall be as gentle as possible at least lesser than 2:1. When the crest height from riverbed is more than 6.00 meters, the slope of the dike shall be gentler than 3:1.

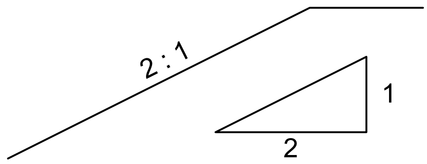


Figure 1.3.6 Minimum Slope of Dike

A slope of 4:1 is usually used for a dike consisting of sand and shall be protected by providing a cover of good soil sodded at least 300 mm thick. When the surface of a dike is covered by a revetment, the slope of dike could be steeper than 2:1.

1.3.7 Berm

If the height of the dike is more than 5 meters, a berm shall be provided along the slopes for stability, repair and maintenance purposes. The berm width shall be 3.00 meter or more.

1.3.8 Extra-embankment

Extra-embankment shall be planned due to consolidation of the dike. The standard for extra-embankment height is shown below:

Table 1.3.8 Standard Value of Extra-embankment Height

Dike Height (m)	Dike Foundation Materials			
	Ordinary Soil		Sand/ Sand & Gravel	
	Extra Embankment Materials			
	Ordinary Soil cm	Sand/ Sand & Gravel cm	Ordinary Soil cm	Sand/ Sand & Gravel cm
≤ 3 m	20	15	15	10
3 m – 5 m	30	25	25	20
5 m – 7 m	40	35	35	30
≥7 m	50	45	45	40

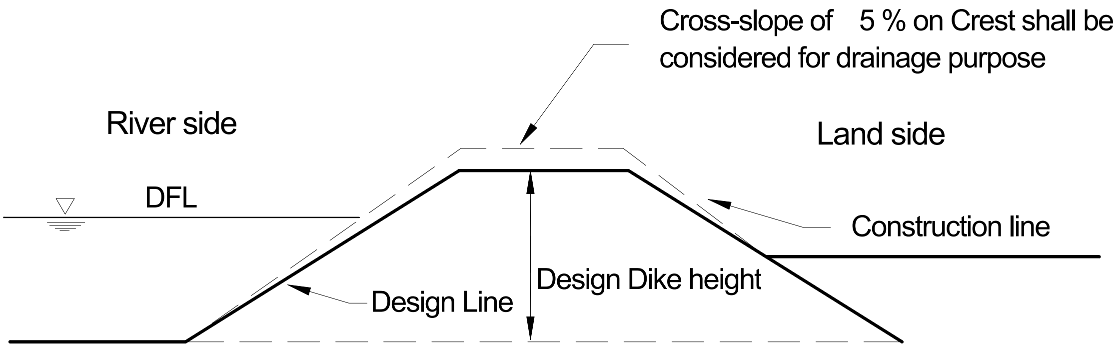


Figure 1.3.8 Extra-embankment Height

1.3.9 Selection of Materials for Dike

Suitable materials for the dike are selected for economic consideration, workability during construction, and stability of the dike. In case available materials on site are unsuitable, other alternatives shall be considered.

1) Evaluation of Materials for Dike

a) Economic aspect:

To secure the required quantities of materials near the construction site.

b) Engineering aspect: Materials shall be

With

- well-grained condition for high density and with high shearing strength for the structural stability.
- high impermeability for resistance to high water stage during flood.
- good workability during dike construction, especially during compaction.
- durability against environmental variation, such as reiteration of wet and dry conditions. Those conditions cause slope failure and cracks of dike body.

Without

- possibility of compressive deformation or expansion.
- toxic organic matter and water-soluble material.

The table below shows the general evaluation of materials for dike considering the above requirements.

Table 1.3.9 Evaluation of Materials for Dike

Soil Classification		Evaluation of Materials for Dike		Countermeasure
		Remarks		
Coarse grained soil	Gravel	o	Permeability of materials is high.	The seepage control measure and sodding will be required.
	Gravelly soil	o		
	Sand	o	Permeability of materials is high, and the slope is easy to fail.	The seepage control measure will be required.
	Sandy soil	o		
Fine grained soil	Silt	o	In case of high moisture content, it is difficult to compact the materials sufficiently.	It is required to lower the moisture content by drying or to improve the soil condition by use of additive materials
	Clay	o		
	Volcanic cohesive soil	o		
	Organic soil	Δ	Most of the materials are with high moisture content. Therefore, it is difficult to compact or mold the materials.	In addition to drying or improvement with additive, mixing with other good materials is also recommended.
Highly organic soil		x	It is difficult to compact or transform the materials due to high moisture content. The compressive deformation of the materials is high. In addition, the materials have the disadvantage for the environmental variation such as the reiteration of the dry and wet conditions.	

O : Applicable
Δ : Applicable with appropriate treatment
X : not applicable

Based on the requirements and general evaluation mentioned above, the materials for the dike are described in details as follows:

a) Suitable Materials for Dike

The following are suitable materials, which meet the basic requirements.

- Well-graded materials: Those materials can be compacted sufficiently. Coarse-grained fractions contribute to the strength of the materials, while the fine-grained fractions contribute to the increase of the impermeability of materials.

- Materials with maximum grain-size diameter of 10 to 15 cm. Maximum grain-size diameter is determined considering the limitation of the rolling thickness during construction. If the maximum diameter is too large, the materials could not be compacted sufficiently.

- Materials with fine-grained fractions (0.075mm or less) content ratio from 15% to 50%. Fine-grained fraction is a requirement to secure the impermeability of dike. However, if fine-grained fraction is more than 50%, there is a high risk of cracks during dry condition.

- Materials with less silt fraction: Silt fraction contributes to the erosion of slope surface and slope failure due to decrease of shear strength caused by high permeability and increase of water content ratio.

b) Unsuitable Materials for Dike

The following are considered unsuitable materials for dike:

i. Materials with fine-grained fractions less than 15%.

In case of sole application of the materials, it is difficult to ensure the impermeability of the dike. However, the materials have sufficient strength after compaction. Therefore, the materials can be used as dike materials in the following cases:

1 Flood with short duration and dike with a broad cross sectional area.

2 Dike with sealing works (revetment) on riverside or drain works in landside toe.

ii Materials with less bearing capacity for the construction equipment.

This is from the viewpoint of the limitation of construction equipment. Therefore, application of the materials depends on selection of construction equipment.

iii Highly organic materials.

The compressive strength of the materials is insufficient and the compressive deformation of the materials is high. In addition, the organic matter will decompose after construction.

2) Countermeasures for Unsuitable Materials for Dike

Materials that do not meet the basic requirements are not always unsuitable for the dike. These materials can be used by taking countermeasures to meet the requirements. There are three (3) countermeasures described as follows:

a) Mechanical stabilization

This method aims to stabilize the poorly graded materials by mixing with different graded materials. The following are mainly used for dike works:

i. Mixing the sandy soil with fine-grained soil in order to secure the impermeability of the materials.

ii. Mixing the cohesive soil with coarse-grained soil in order to prevent contraction of cracks caused by dry condition.

Concept of mechanical stabilization is presented in figure below:

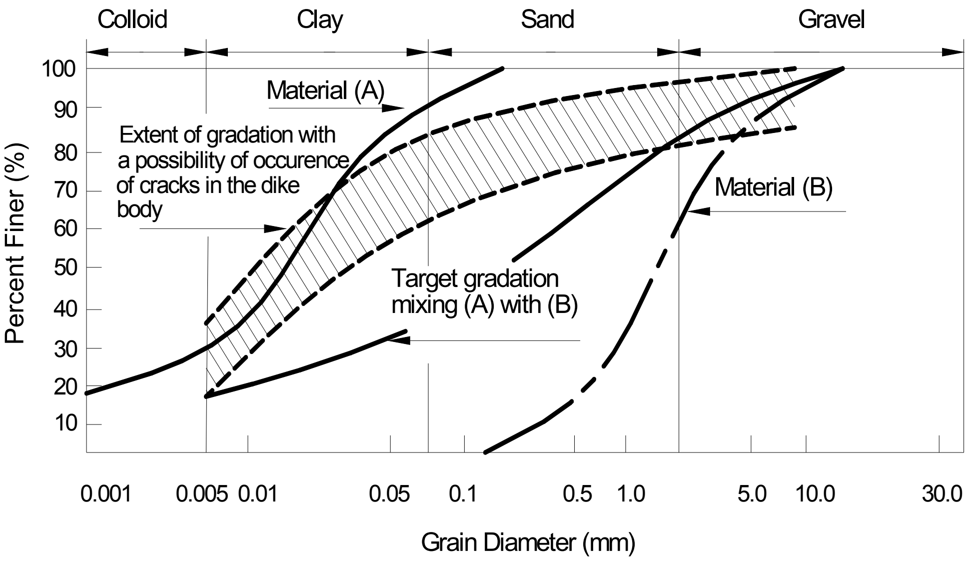


Figure 1.3.9.2.1 Concept of Mechanical Stabilization

b) Lowering the moisture content by drying

This method is applied when the natural moisture content of the materials is higher than the moisture content required for compaction. The natural moisture content of the materials is lowered to the extent described below by use of drying and trench excavation. This type of materials is suitable for dike with the exception of moisture content. When the moisture content is lowered to the desired limit, the soil materials can be considered suitable.

Sandy soil: The moisture content shall be lowered to a certain extent in which 90 % or more of the average degree of compaction ratio can be achieved.

Cohesive soil: The moisture content shall be lowered to allow workability of the compaction equipment. In general, lowering the moisture content of the cohesive soil is not effective. Therefore, the soil stabilization by the use of additives is also used at the same time.

c) Soil stabilization by use of additives

This method is applied to materials with insufficient bearing capacity (for construction equipment). The materials are stabilized by the use of additives, such as lime, cement, etc.

The solidification mechanism of the corresponding additives is as follows:

Lime: Lime reacts to the moisture of the materials by generating heat and absorbing the moisture from the materials. In addition, lime is chemically solidified by pozzolanic reaction in the long run.

Cement: Cement is hydrated with the moisture of the materials and solidified by the bonding with the soil particles. Solidification duration of cement is shorter than that of lime and is attained within 3 to 7 days after mixing with the materials.

1.3.10 Specification for Compaction

Basically, the engineering characteristics of the earth materials are improved by compaction. Dike, mostly of earth materials; indispensably requires compaction to enhance its durability. Therefore, compaction in the design specification must be required.

Compaction is specified into 1) Quality-Specified Type and 2) Construction Method-Specified Type. Their applications shall be determined by the characteristics, materials and site conditions. Basically, quality-specified type shall be applied in principle.

1) Quality-Specified Type

For this type, quality of dike is specified in the contract. Inspection during construction shall be based on the specified quality.

Table 1.3.10 Standard of Specification of Compaction

ITEM	Soil Classification			
	Coarse Grained Soil	**Sandy Soil** 15% $\leq$ FC < 25%	**Sandy Soil** 25% $\leq$ FC < 50%	**Cohesive Soil**
$D_{c.ave}$	$D_{c.ave}$ = 90%	$D_{c.ave}$ = 90%	-	-
w_n	-	-	Extent able to secure the trafficability of equipment	
V_a	-	-	$V_a \leq$ 15%	2% $\leq$ Va $\leq$ 10%
S_r	-	-	-	85% $\leq$ Sr $\leq$ 95%
Pass rate of quality	-	-	90%	90%
Lower limit of quality	D_c = 80%		-	-

- Fine grained fraction (Fc): the ratio of weight of materials smaller than 0.074 mm.
- Degree of compaction (D_c): the ratio of the dry unit weight of the compacted materials to the maximum dry unit weight of materials at the standard compaction test.
- $D_{c.ave}$ Average value of D_c during the dike compaction/construction
- Moisture content (w_n): the ratio of the weight of water to the weight of solid soil
- Air void ratio (V_a): the ratio of the air void to the apparent volume of the materials.
- Degree of saturation (S_r): the ratio of the volume of water to the void of the materials.

There are two (2) kinds of specifications:

a) Specification by use of dry unit weight

This specification of dike is mainly used for gravelly soil, sand, and sandy soil. The quality of dike is specified by the use of the lower limit and average value of the degrees of compaction and the moisture content of the materials.

Even if the degree of compaction is larger than the lower limit of 80%, if the moisture content is smaller than W_{opt}, the strength of the material could be smaller when it is submerged during flood time.

The above relation is presented in the figure below:

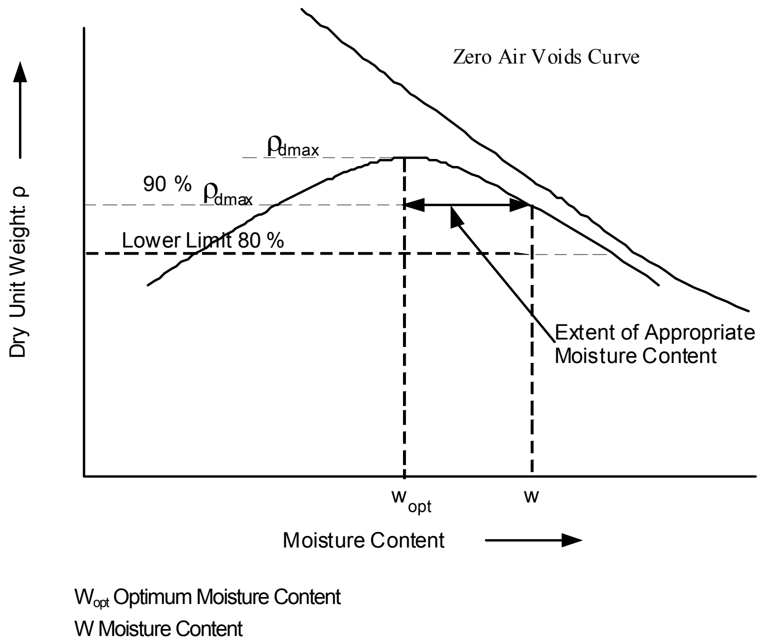


Figure 1.3.10 Relation between Dry Unit Weight and Moisture Content

b) Specification by Air Void Ratio or Degree of Saturation

The quality of dike is specified by air-void ratio and degree of saturation. This method is mainly applied to cohesive soils. Materials have high moisture content; thus, it is difficult to apply the specification by use of dry unit weight.

The values of specifications in Table 1.3.10 are checked during the dike construction by observing the wet unit weight of the compacted material (ρ_t) and its moisture content (w_n). The dry unit weight (ρ_d), the degree of compaction (D_c), the air void ratio (V_a) and the degree of saturation can be calculated as follows:

Dry unit weight: $$\rho_d = \frac{100\rho_t}{100 + w_n} \quad (g/cm^3)$$

Degree of compaction: $D_C = 100\left(\dfrac{\rho_d}{\rho_{dmax}}\right)$ (%)

Air Void ratio: $V_a = 100 - \dfrac{\rho_d}{\rho_w}(\dfrac{100}{\rho_s} + w_n)$ (%)

Degree of saturation: $S_r = \dfrac{w_n}{\dfrac{\rho_w}{\rho_d} - \dfrac{1}{\rho_s}}$ (%)

Where,

ρ_{dmax}: maximum dry unit weight of the materials in the laboratory test

ρ_s : unit weight of soil

ρ_w : unit weight of water (1 g/cm^3)

2) Construction Method-Specified Type

The dike construction method is specified in the contract, including types of compaction equipment and the degree of compaction.

The quality of the dike is managed in the construction method, such as construction equipment and compacted times. In case that the compaction method of the materials is empirically established or the materials are gravelly sand, it is preferable to apply this specification.

1.4 Other Design Consideration

1.4.1 Classification of Foundation

The ground condition of the proposed dike alignment shall be investigated in consideration of the foundation. Weak and permeable foundations are major issues which should be verified during the preparation and the survey and investigation stages. Weak and permeable foundations can be described as follows:

1) Weak Foundation

A weak foundation can cause damages in and around the dike, such as sliding failure and large subsidence of the dike during construction, decrease in function of dike due to the continuous subsidence after the construction, deformation of surrounding foundation and structures, etc.

The following are the possible location of weak foundation and shall be considered during field investigation:

a) Flat swamp/damp or paddy field area
b) Flat paddy field extending into plateau or mountainous area
c) Inland side of natural levees, sea sides, or sand dunes

Classification of weak foundation is summarized as follows:

Table 1.4.1a Classification of Weak Foundation

Composition of Foundation	Judgment
Clay	N ≤ 3 (Standard penetration test) qc ≤ 3 kgf/cm2 (Dutchcone penetration test) Subsidence ≤ 100 kgf (Swedish sounding test) qu ≤ 0.6 kgf/cm2 (qu: Unconfined compressive strength) Natural water content ≤ 40 % (alluvial clay)
Organic soil	Peat foundation Muck foundation consists of highly organic soil
Sand	N ≤ 10 (Standard penetration test) Foundation of poorly graded fine sand

2) Permeable Foundation

A permeable foundation can make dike prone to breach due to seepage at the foundation, boiling, and piping at the landside toe of the dike.

The following are the possible location of a permeable foundation and shall be considered during field investigation.

a) Area near a river, which may be a fan, a natural levee or a delta.

b) Traces of old rivers.

c) Landslide area with spring water or rise in groundwater level during floods.

The following can be considered as permeable foundations:

a) Surface layer consisting of sand, gravel or coarse sand.

b) Continuous sand gravel layer or coarse sand layer under a thin and impermeable surface layer.

The relationship between soil classifications and the coefficient of permeability is shown in table below. Generally, foundation with coefficient of permeability more than $k = 5 \times 10^{-3}$ cm/s can be judged as permeable foundation.

Table 1.4.1b Relationship between D_{20} and Coefficient of Permeability (k)

D_{20}	K (cm/s)	Soil Classification	D_{20}	k (cm/s)	Soil Classification
0.005	3.00×10^{-6}	Coarse clay	0.18	6.85×10^{-3}	Fine sand
0.01	1.05×10^{-5}	Fine silt	0.20	8.90×10^{-3}	
0.02	4.00×10^{-5}		0.25	1.40×10^{-2}	
0.03	8.50×10^{-5}		0.3	2.20×10^{-2}	Medium sand
0.04	1.75×10^{-4}		0.35	3.20×10^{-2}	
0.05	2.80×10^{-4}		0.4	4.50×10^{-2}	
0.06	4.60×10^{-4}	Minute sand	0.45	5.80×10^{-2}	
0.07	6.50×10^{-4}		0.5	7.50×10^{-2}	
0.08	9.00×10^{-4}		0.6	1.10×10^{-1}	Coarse sand
0.09	1.40×10^{-3}		0.7	1.60×10^{-1}	
0.10	1.75×10^{-3}		0.8	2.15×10^{-1}	
0.12	2.60×10^{-3}	Fine sand	0.9	2.80×10^{-1}	
0.14	3.80×10^{-3}		1.0	3.60×10^{-1}	
0.16	5.10×10^{-3}		2.0	1.80	Fine gravel

D_{20}: Diameter at 20% of grain-size distribution curve.

1.4.2 Floodwall

If land acquisition is a major consideration for dike in urban area or in areas close to important facilities, a floodwall may be an alternative. The floodwall height shall include freeboard, but for a large river or in the place with high wave length, the floodwall may be higher than a man's height, where river is not at sight. Also the height shall not impair the scenic views, etc. The desirable height shall be 80 cm or less. The height shall have stability from structural standpoint.

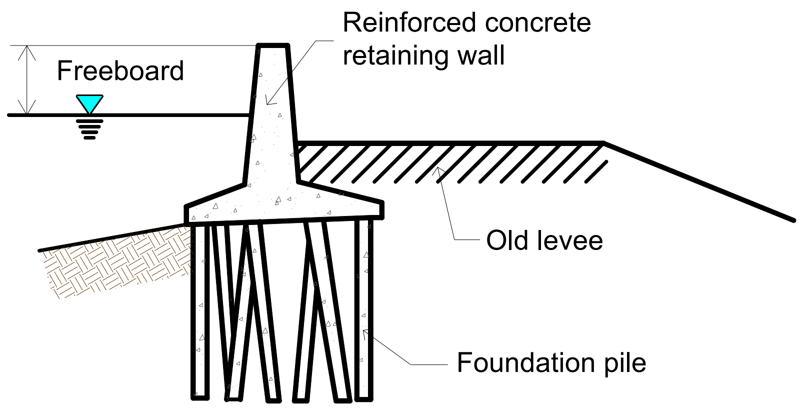


Figure 1.4..2a Self-Standing Retaining Wall (Example)

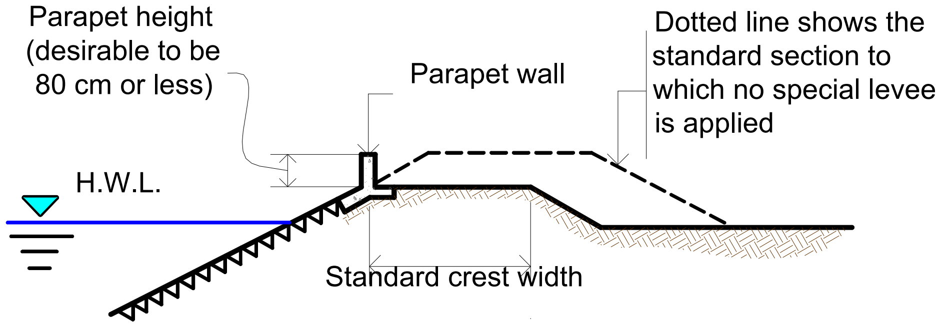


Figure 1.4.2b Parapet Wall (Example)

1.4.3 Dike Affected by Tidal Fluctuation

The dike height affected by high tide (section at which design high-tide level is higher than the design flood level) shall be designed in consideration of the high-tide level plus the surge height due to wave action.

The dike affected by high tide should be generally covered on the respective three faces by concrete or similar material, taking into account the wave overtopping action. It is necessary to provide drainage at the dike's heel in order to collect local runoff and the floodwaters resulting from the wave overtopping action.

1.4.4 Overflow Dike

The dike for special purpose, such as overflow levee, guide levee, separation levee, etc. shall be planned to allow sufficient demonstration of the functions.

The height, length, width, etc. (of overflow levee, guide levee, separation levee, etc.) depend on the place of construction, purpose, etc.; and therefore, must be thoroughly analyzed on a case to case basis. In some cases, hydraulic model tests, etc. must be conducted to confirm the appropriateness of the design of each structure.

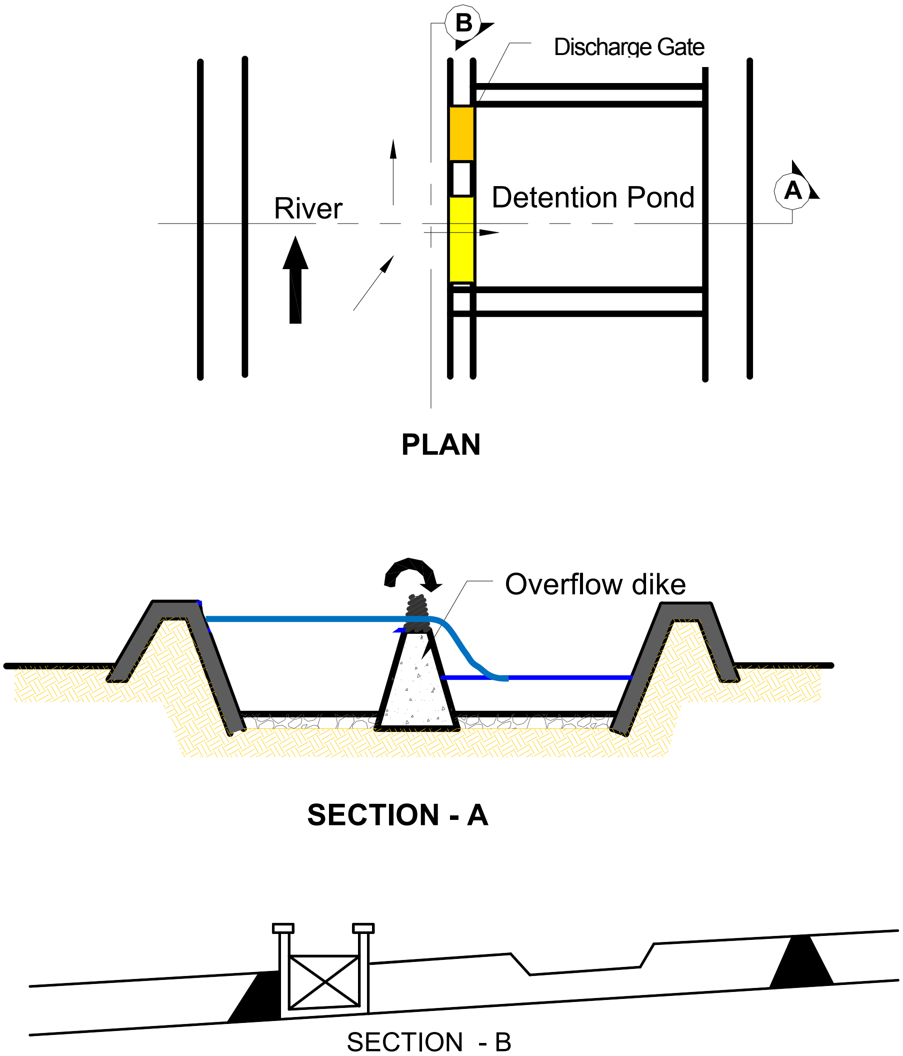
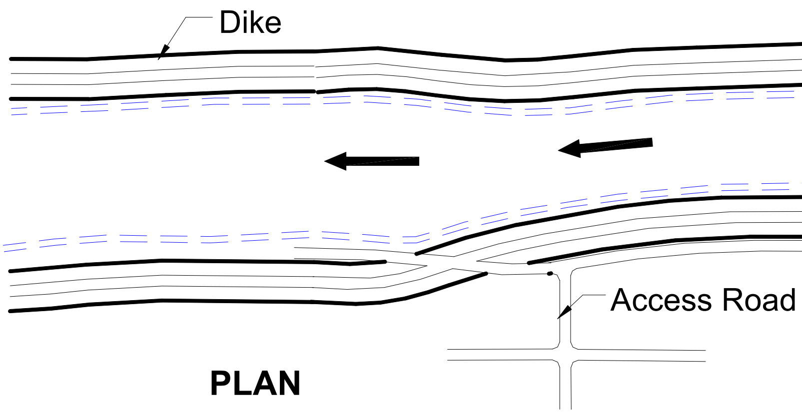


Figure 1.4.5 Illustrative Example of Overflow Dike

1.4.5 Provision of Access Road and Stairs

Access road shall be provided in portions of dike where there are human activities (i.e., quarry, fishing, agriculture, etc.). Access road shall be built properly in consideration to flood control function of a dike. Whenever possible, access road shall be constructed near the existing peripheral and/or riverside road with its entrance facing downstream side. For maintenance and other purposes, a built-in stair is also necessary. Stairway shall be strong enough to withstand the expected external forces acting on it.



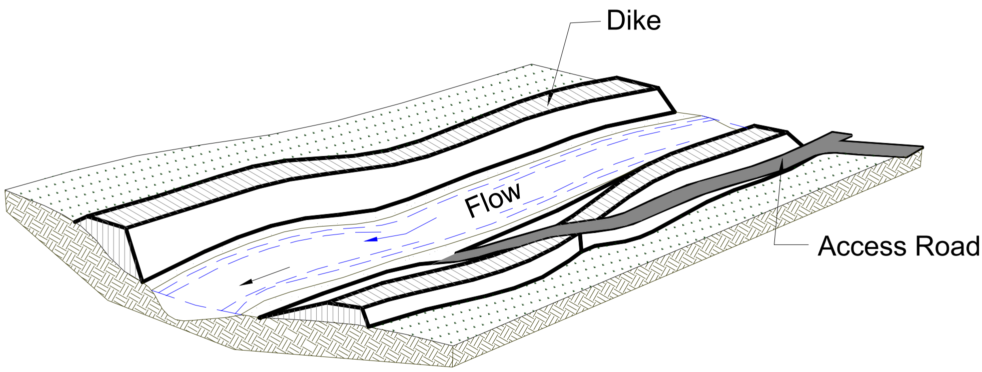


Figure 1.4.6 Access Road

1.4.6 Toe Protection Work (Landside)

When the dike is constructed along the road or the drain, provide toe protection work, which shall have a height of 0.5 – 1.0 m and shall be made of dry stone masonry to secure the drainage in the dike body.

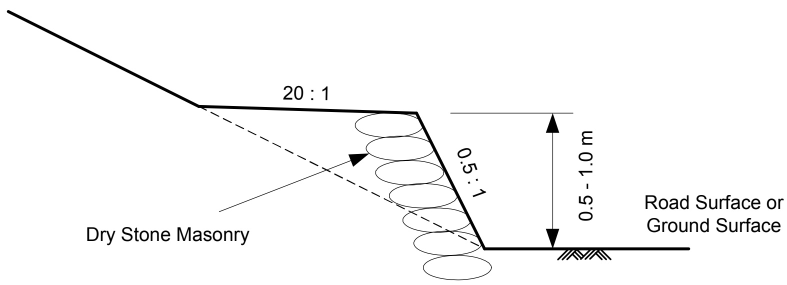


Figure 1.14 Toe Protection work

1.4.7 Safety against Seepage

During flood, the pore pressures of the dike will increase due to the seepage of the floodwater, which eventually decrease the shearing strength of dike. As a result, the safety of the dike will be decreased. Verification of the dike safety shall be carried out by the following items:

1) Safety against slope failure during the flood by the slip-circle method.

2) Safety against piping of the foundation by seepage analysis.

Chapter 2 REVETMENT

2.1 Basic Concept

The external force which contributes to erosion depends on the river flow velocity. The revetment protects the collapse of riverbank due to erosion, scouring and/or riverbed degradation.

Revetment shall be designed based on the existing site conditions, such as river flow velocity and direction, embankment material, topographical, morphological, and geological conditions of the riverbank, etc. Further, revetment shall be designed to withstand the lateral forces due to high velocity flow, when located in flow attack zone, on a weak geological condition of riverbank, and with poor embankment materials.

The revetment structure shall consist of slope covering works, foundation works and foot protection works. The components of revetment are illustrated below.

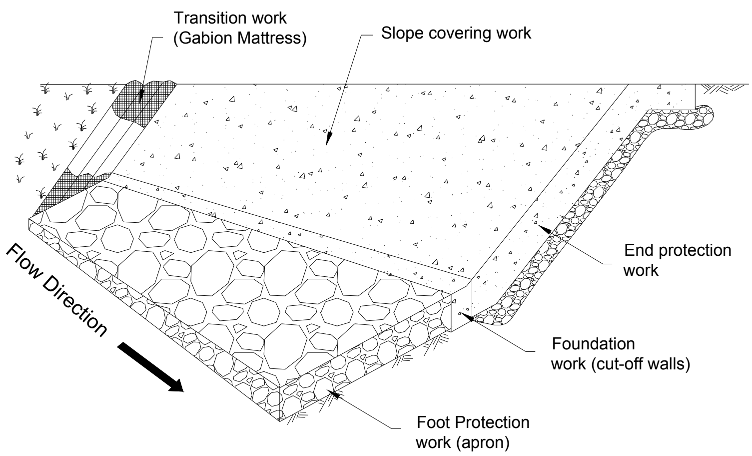


Figure 2.1a Components of Revetment

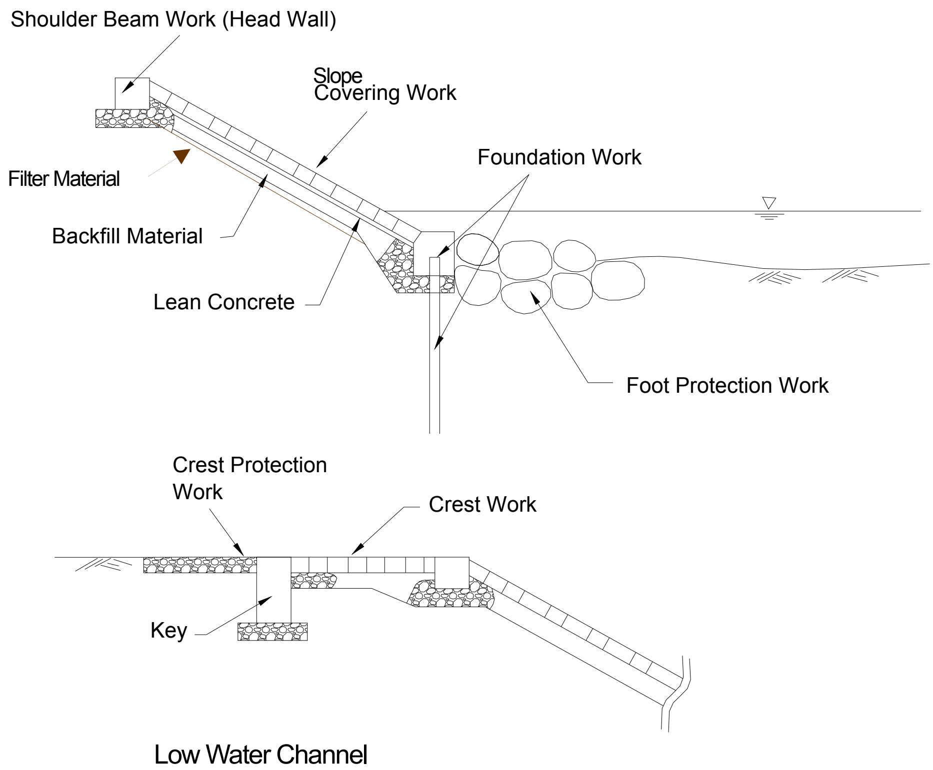


Figure 2.1b Components of Revetment

1) Slope covering work: directly covers and protects the bank slope from direct attack from flood water, boulders and floating debris.
2) Foundation work: constructed at the toe of the slope that supports the slope covering works.
3) Foot protection work: constructed to prevent scouring in front of the foundation work and outflow of material from the back of the slope covering work.
4) Shoulder beam work: headwall installed at the shoulder of the revetment to prevent damage.
5) Backfilling material: materials which are backfilled to the slope covering work to prevent residual water pressure underneath the slope covering work.
6) Filter material/cloth: installed behind the backfilling material to prevent the coming out of fine materials underneath the revetment due to flow forces or the residual water pressure.
7) Crest work: protect the crest of the slope covering work.
8) Key: installed at the end portion of the crest work to protect it against erosion at the back of the revetment.
9) Crest protection work: installed at the end portion of the key to join the crest and the original ground in order to protect against erosion at the back of the revetment.

10) End protection work: installed at the upstream and downstream end of the slope covering work to prevent undermining of materials behind the slope covering work.

11) Transition work: installed between the upstream and downstream sides of the end protection work and the natural banks to connect the revetment and natural banks smoothly.

During planning and design stage, the following shall be considered:

1) Alignment of revetment shall be as smooth as possible.
2) Structural type of the revetment shall be determined based on the estimated external forces (velocity of flood flow) and the characteristics of river.
3) Foot protection works shall be considered based on external forces.
4) Transition structure (end protection works) of the revetment to the original bank shall be provided.

2.2 Location and Alignment

Revetment should be planned at riverbanks in high velocity areas considering the site conditions (river flow direction, topography, geology, and embankment material). Construction should be prioritized on river bend or at stream attack part or drift stream part as shown in Figure 2.2.1. The possibility of scour is very high on these locations compared to the other parts along the river system.

Although alignment of revetment depends on channel plan or existing bank alignment, bank alignment should be improved with revetment as smooth as possible particularly at bend areas.

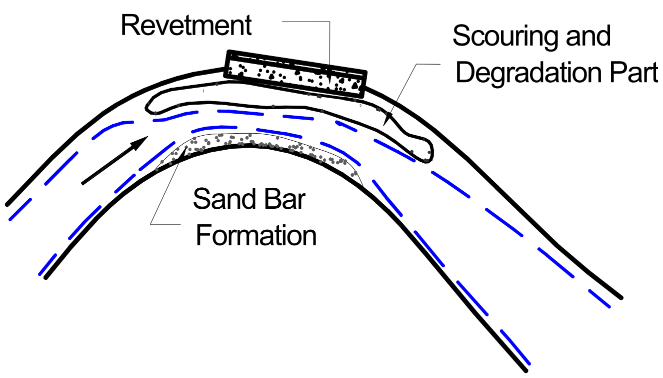


Fig. 2.2.1 Construction of Revetment at River Bend

2.3 Type Selection

2.3.1 Calculation of Design Velocity for Revetment

1) Design velocity for revetment

The flow velocity is an indispensable factor in the selection of the type of slope covering work. The mean velocity derived in the uniform flow calculation is not equal to the velocity of flow in front of the revetment, which is influenced by the effects of sand bar, bend and foot protection work, etc. To design the revetment, calculate the design velocity of the revetment using the average mean velocity as detailed below.

Apply correction to the mean velocity derived from the uniform flow calculation to the design velocity of the revetment. If there is no correction, it is recommended that the maximum value in the mean velocities of the representative cross sections at the design flood is adopted as the design velocity of the revetment at the proposed site.

2) Correction of mean velocity for design

The design velocity (V_D) is estimated based on the average value of the mean velocities of the representative cross sections (V_{mave}) at the design flood, shown as follows:

$V_D = \alpha V_{mave}$ ....(Eq. 1)

Where:

V_D : Design velocity (m/s)
V_{mave} : Average value of the mean velocities of the representative cross sections at the design flood.
α : Correction coefficient

Considering the effects of the bend or scouring and the installation of effective foot protection work, the correction coefficient is estimated as follows:

Table 2.3.1 Application of Equation of Correction Coefficient

Stretch	Foot protection work	Applied Equation
Straight	w/o	Eq. 2
	w/	Eq. 2 + Eq. 5 or 6
Bend	w/o	Eq. 3 or 4
	w/	Eq. 3 or 4 + Eq. 5 or 6

a) Correction for straight stretch and without foot protection work

Considering the decrease of the stream area due to sand bar, the correction coefficient is as follows:

$$\alpha = 1 + \frac{\Delta Z}{2H_d} \quad ....(\alpha \leq 2.0) \quad ....(\text{Eq. 2})$$

Where

ΔZ: Maximum scouring depth (m) (refer to 2.4.3)
H_d: Average design water depth (m)

b) Correction for bend stretch and without foot protection work

Inner bank of the bend: $$\alpha = 1 + \frac{B}{2r} \quad .....(\text{Eq. 3})$$

Outer bank of the bend: $$\alpha = 1 + \frac{B}{2r} + \frac{\Delta Z}{2H_d} \quad .....(\text{Eq. 4})$$

Where

α: Correction coefficient
(Segment 1: $\alpha \leq 2$, Segment 2 and 3: $\alpha \leq 1.6$)
B: River width (m)
r: Radius of the bend (m)
ΔZ: Maximum scouring depth (m)
H_d: Average design water depth (m)

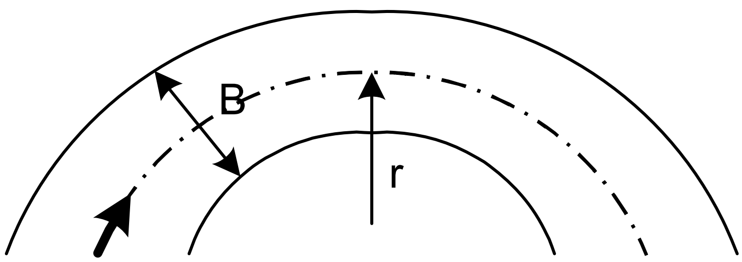


Figure 2.3.1.1 Bend Stretch of River

c) In case with foot protection work

In case structure with the adequate foot protection works (crest width of 2 m or more), the correction coefficient of above a) or b) (α_1) is revised as follows:

$b_w / H_l > 1.0$: $\alpha = 0.9\,\alpha_1$ .... (Eq. 5)

$b_w / H_l \leq 1.0$: $\alpha = 1.0\alpha_1$ .... (Eq. 6)

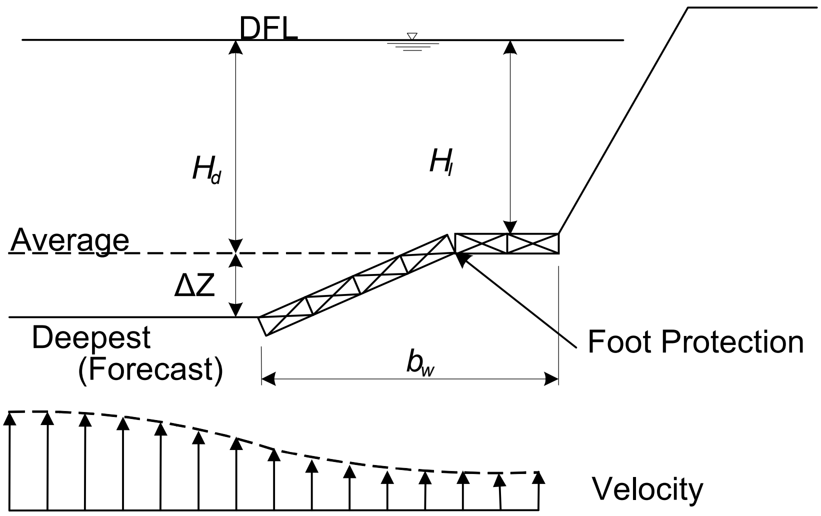


Figure 2.3.1.2 Cross Sectional Distribution of Velocity

2.3.2 Slope Covering Works

There are many types of slope covering work shown in the table below. The type of slope covering work at the site shall be selected based on the design velocity, slope, availability of construction materials near the site, ease of construction works and economy, etc. When there are constraints due to the required boulder stones during flood and the slope of the bank, combination of the slope covering works shall be considered.

Table 2.3.2.1 Criteria of Slope Covering Work

Type of Revetment *	Allowable Design Velocity (m/s)	Slope (H:V)	Remarks
1) Sodded Riverbank with Pile Fence	2.0	Milder than 2 : 1	a. Not applicable for places near roads and houses. b. Diameter and length of wooden pile shall be determined considering past construction records. Diameter of fill boulder shall be determined using Table 2.5.3a.
2) Dry Boulder Riprap	3.0	Milder than 2 :1	a. Diameter of boulder shall be determined using Table 2.3.2.2. b. Height shall not exceed 3 meters.
3) Grouted Riprap (Spread Type)	5.0	Milder than 1.5 : 1	Use Class "A" boulders for grouted riprap and loose boulder apron.
4) Grouted Riprap (Wall Type)	5.0	1.5 : 1 to 0.5 : 1	Use class "A" boulder for grouted riprap.
5) Gabion (Spread Type)	5.0	Milder than 1.5 : 1	a. Not advisable in rivers affected by saline water intrusion. Not applicable in rivers where diameter of boulders present is greater than 20 cm.
6) Gabion (Pile-up type)	6.5	1.5 : 1 to 0.5 : 1	a. Not advisable in rivers affected by saline water intrusion. Not applicable in rivers where

Type of Revetment *	Allowable Design Velocity (m/s)	Slope (H:V)	Remarks
			diameter of boulders present is greater than 20 cm.
7) Rubble Concrete (Spread Type)		Milder than1.5 : 1	Chapter 3
8) Rubble Concrete (Wall Type)			
9) Reinforced Concrete			Minimum thickness of 20 cm.
10) Gravity Wall			
11) Sheet Pile		vertical	In cases where ordinary water level is very high.

* Refer to Typical Design Drawings

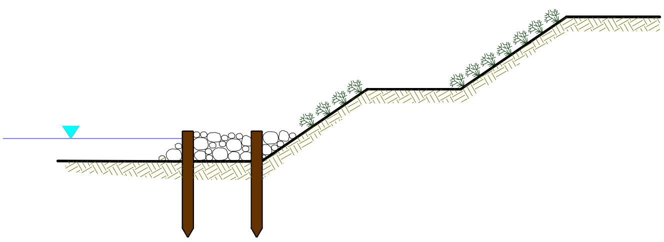


Figure 2.3.2.1a Sodding with Grass or Some Other Plants (Natural Type)

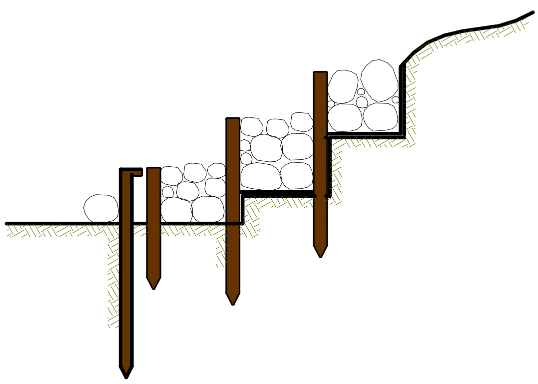


Figure 2.3.2.1b Wooden Pile Fence

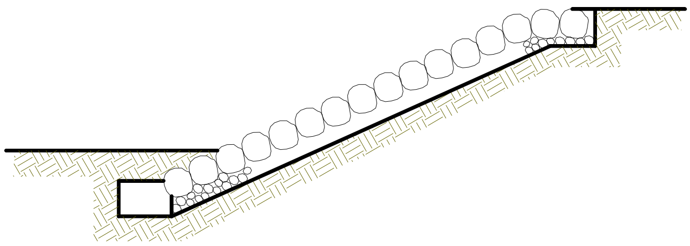


Figure 2.3.2.2 Dry Boulder Riprap

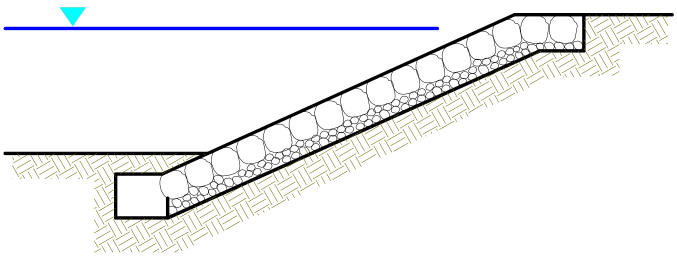


Figure 2.3.2.3 Grouted Riprap, Spread Type

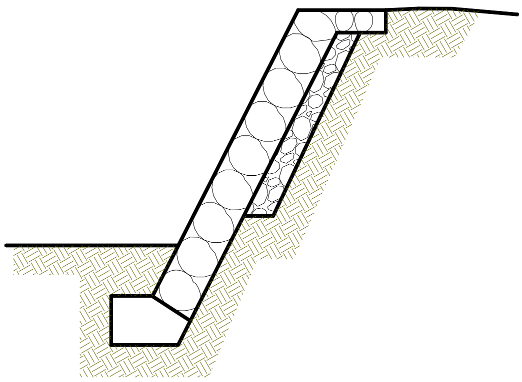


Figure 2.3.2.4 Grouted Riprap, Wall Type

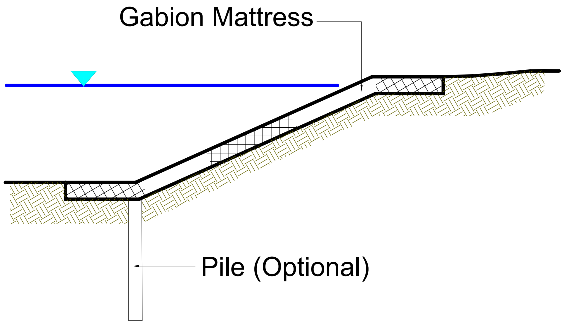


Figure 2.3.2.5 Gabion Mattress, Spread Type

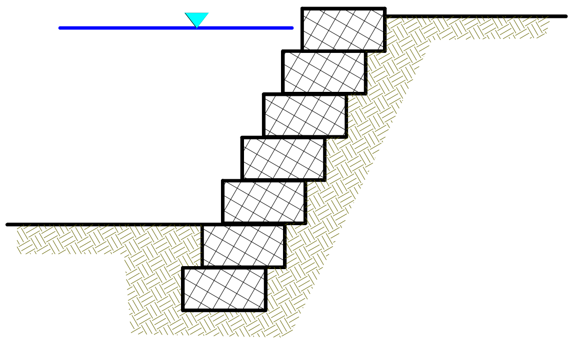


Figure 2.3.2.6 Gabion Mattress, Pile-up Type

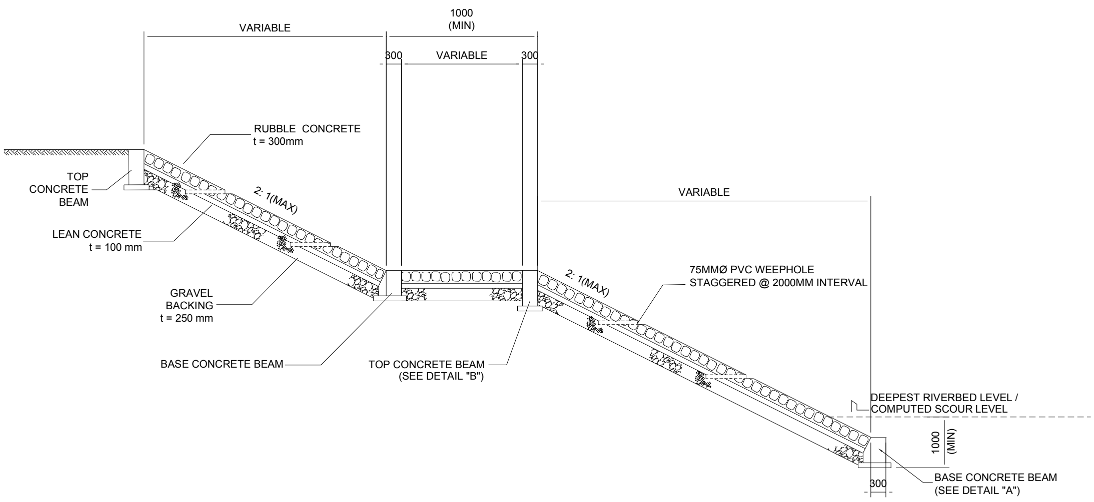


Figure 2.3.2.7 Rubble Concrete, Spread Type

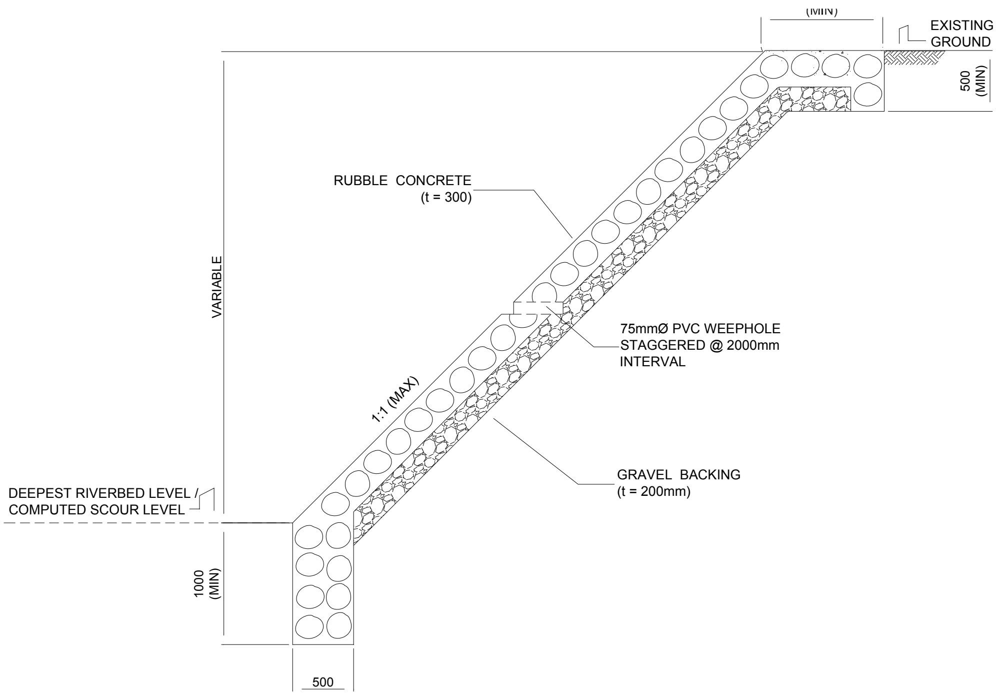


Figure 2.3.2.8 Rubble Concrete, Wall Type

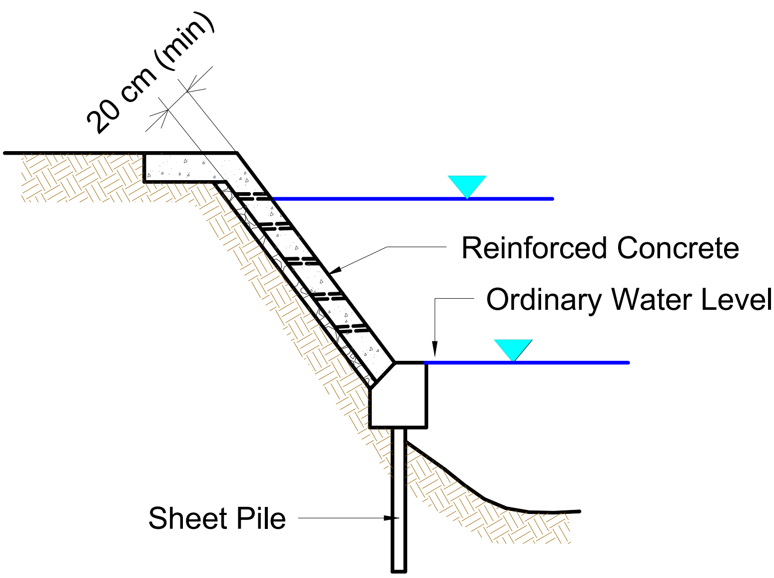


Figure 2.3.2.9 Reinforced Concrete

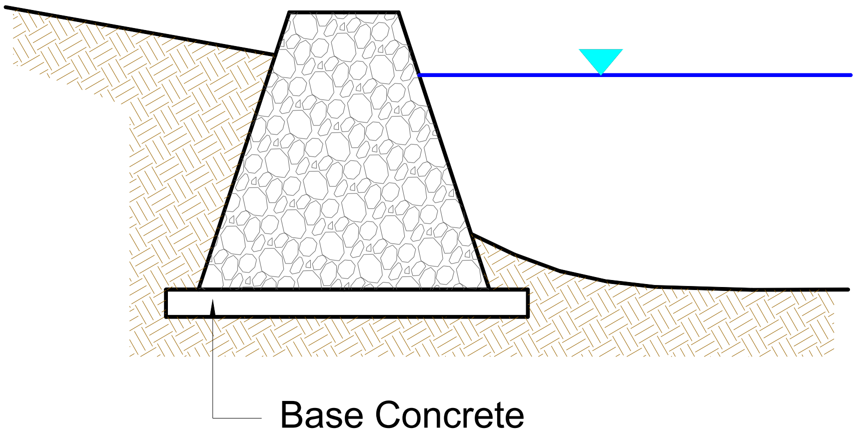


Figure 2.3.2.10 Gravity Wall

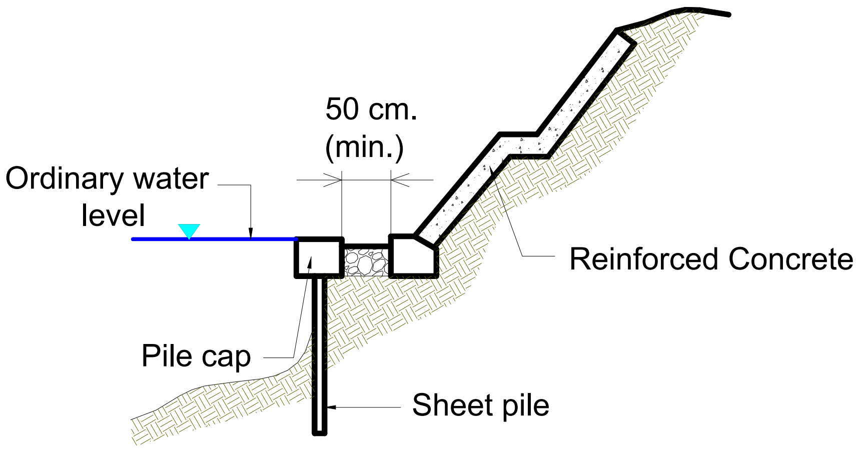


Figure 2.3.2.11 Steel Sheet Pile and Reinforced Concrete (Segment Combination)

Table 2.3.2.2 Diameter of Boulder for Dry Boulder Riprap (Unit: cm)

Water Depth (m)	Design Velocity (m/s)				
	1.0	2.0	3.0	4.0	5.0
1.0	20	20	20	60	-
2.0	20	20	20	30	70
3.0	20	20	20	30	50
4.0	20	20	20	20	40
5.0	20	20	20	20	40

(In case of slope 2:1)

2.4 Design Criteria

2.4.1 Height (Crest elevation of revetment)

Basically, the height of revetment is determined by setting the Design Flood Level (DFL). The revetment height shall be designed up to the top of riverbank or crest of embankment because floodwaters may exceed the DFL or top of the bank.

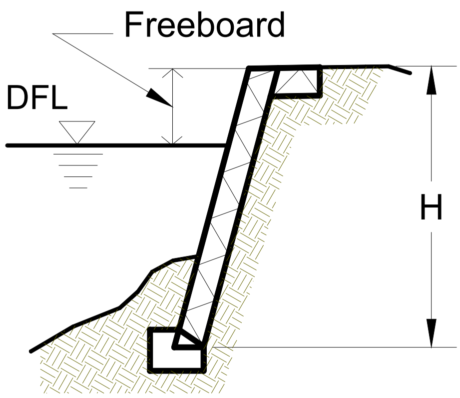


Figure 2.4.1 Height of Revetment

2.4.2 Depth (Depth of top of foundation)

The depth of the foundation shall be deeper than 1.0 m from the maximum scouring depth. If there is a difficulty to calculate the maximum scouring depth, it should be 1.0 m below from the deepest riverbed.

The top elevation of the foundation work is determined as follows:

1) Plot the 1 meter elevations below the maximum scouring level/deepest riverbed level from the cross-sections and project in the longitudinal profile.
2) Draw the line of the lowest elevation from 1) with the same longitudinal gradient of the top of slope covering work.

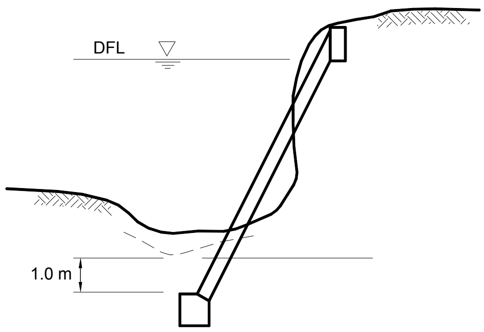


Figure 2.4.2a Depth of Foundation

In case there is difficulty in attaining the depth of the top of the foundation due to extreme scouring or riverbed degradation, the depth from the top of the foundation can be achieved by sheet pile foundation or foot protection work. The following four (4) cases can be considered for the top elevation of the foundation work:

i. The top elevation of the foundation work is set at the maximum scouring depth, and the minimum foot protection work shall be installed.

ii. The top elevation of the foundation is set above the maximum scouring depth, and the flexible foot protection shall be installed to cope with the scouring.

iii. The top elevation of the foundation is set above the maximum scouring depth, and the foundation work by sheet pile and the foot protection shall be applied in order to cope with scouring.

iv. In cases it is difficult to have adequate depth of embedment for the foundation work, such as high ordinary water level, tidal river, etc; cantilever sheet pile shall be installed as foundation work.

For cases ii and iii, the top elevation of the foundation work shall be set at 0.5-1.5 m deeper than the average riverbed level.

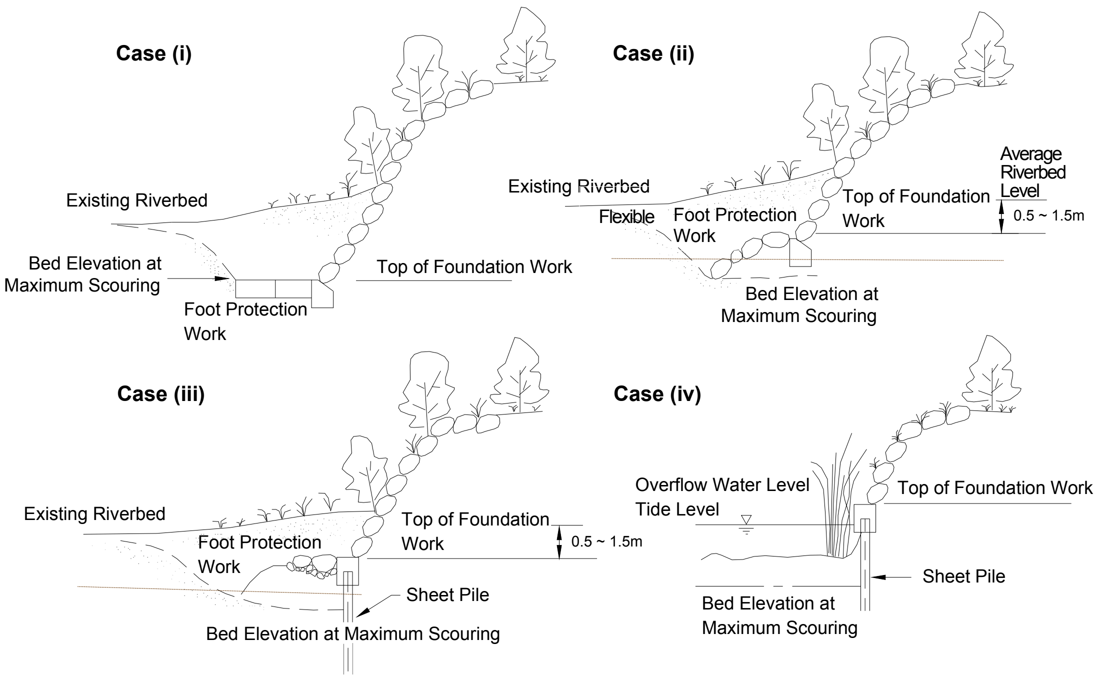
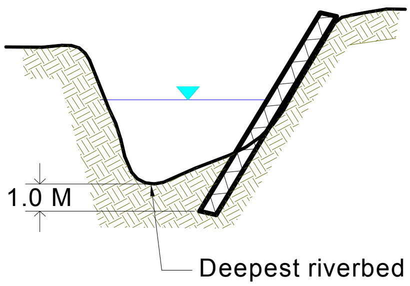


Figure2.4.2b Foundation Work

For a narrow river (less than 50 meters in width) the minimum depth of revetment foundation shall be 1.0 meter below the deepest riverbed elevation of the original riverbed or design riverbed, because riverbed materials are subjected to erosion during flood times

In wide river (more than 50 meters in width) with generally mild velocity and with fix mainstream course flowing very far from the bank where the proposed revetment will be constructed, (more than 20 meters away) the foundation may be 1.0 meter below the existing toe of the bank (Figure 2..4..2c). However, if there is tendency of riverbed degradation or change in mainstream course, the foundation depth should be more than 1.0 meter below the deepest riverbed of the original or designed river bed.



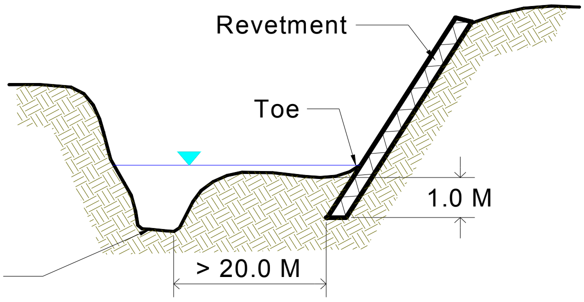


Figure 2.4.2b **Figure 2.4.2c**

2.4.3 Maximum Scouring Depth/Deepest Riverbed Level

1) Factors Contributing to Scouring

Bank erosion is attributed primarily to near-bank scouring of the riverbed during flood periods. Damage to existing revetments is mostly caused by scouring in front of the bank. Therefore, predicting the deepest riverbed level in the future is important to determine the foundation work of the revetment. The future deepest riverbed level is calculated by the scouring depth from the average riverbed level.

Various factors which contribute to scouring include the following:

a) Changes in average riverbed elevation

Channel excavation/dredging lower the average bed elevation, and the scoured areas become lower accordingly. When there is reduction in sediment transport upstream, sediment balance is affected resulting in a lower bed elevation.

b) River cross-section (change of river width and bend of river)

River cross-section directly influences scouring in two ways. One is that a change in river width from wide to narrow causes the water depth to increase; and the other is that a curved or meandering river causes the flow to move toward one side of the channel, resulting in bank scouring.

c) Structures

A structure located in the path of flowing water increases the velocity of flow around the structure and causes local scouring.

d) Sand bar - induced scouring

Sand bar deposits in the river cause obstruction to flow. When the scale of the height of sand bars is roughly equal to the scale of the water depth, the scale of the scouring caused by sand bar is relatively large. The amount of bar-induced scouring becomes larger if the influence of the bars occurred at the bend or the meandering spot.

2) Estimation method for maximum scouring depth, ΔZ

The scouring depth is measured from the average riverbed level. Basically, maximum scouring depth (ΔZ) at the proposed structure site is estimated as the larger value between the computed maximum scouring depth (ΔZc) and surveyed maximum scouring depth (ΔZs).

- Calculated maximum scouring depth (ΔZc) is an empirical value that considers the relationship among the width of a waterway, depth, the riverbed material, and the radius of curve, etc.

- Surveyed maximum scouring depth (ΔZs) is the deepest riverbed determined from actual field survey (cross sectional survey).

Scouring phenomena occur along the entire river stretch with different effects for straight line and bend or curve waterway. The primary factors that contribute to scouring based on the alignment of river are:

Straight–line waterway	:	sand bar height
Curve waterway	:	bend of river alignment

The maximum scouring depth is estimated as follows:

a) Maximum scouring depth for straight line waterway:

Maximum scouring depth (ΔZ) is influenced by the height of sand bar. The maximum scouring depth (ΔZ) is calculated according to the conditions of development of sand bar at the site.

In case of b/Hd < 10 or dr < 0.2mm

In case the riverbed is formed by fine sands (0.2mmΦ or less) and the ratio of river width (b) and average water depth (Hd) is 10 or less (b/Hd $\leq$ 10), the sand bar is not developed. Therefore, the surveyed maximum scouring depth (ΔZs) is the maximum scouring depth (ΔZ).

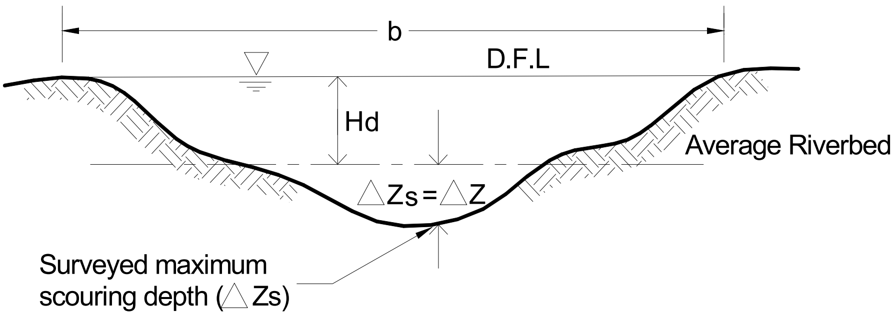
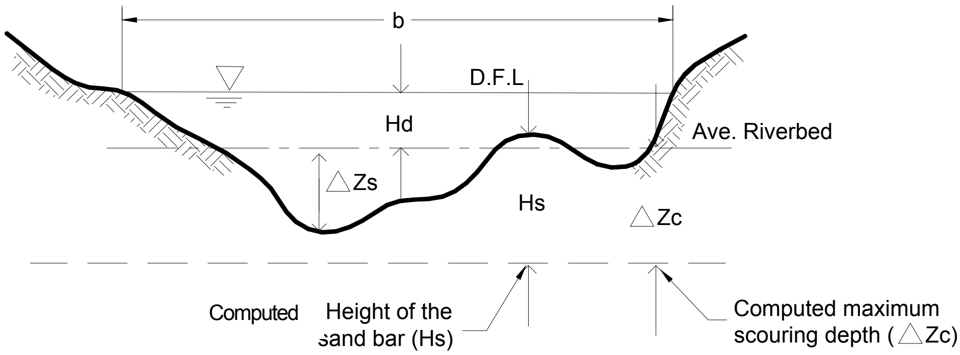


Figure 2.4.3a Deepest Riverbed and Maximum Scouring Depth (ΔZ)

In case of b/Hd > 10 and dr > 2 cm

When the ratio b/Hd exceeds 10 (b/Hd > 10) and the riverbed is formed by gravel, sand bar is generally formed. In this case, computed maximum scouring depth (ΔZc) should be determined. Afterwards, compare with the surveyed maximum scouring depth (ΔZs). The maximum scouring depth (ΔZ) is the larger value.

'dr' is mentioned in Part 1, Planning at section 3.3.4, River Bed Material Survey.



Where:

b: River Width
H_d: Average Water Depth
H_s: Height of Sandbar

Figure 2.4.3b In Case Maximum Scouring Depth is Influenced by the Height of Sand Bar

The maximum scouring depth is estimated, as follow:

i. The ratio of width of the waterway and average water depth is calculated as follows:

b / H_d -- **I**

ii. The ratio of average water depth and diameter of typical riverbed material is defined as:

H_d / d_r -- **II**

iii. The ratio of H_s / H_d is decided from figure based on I and II. Using Figure 2.4.3c below, value of H_s/H_d is obtained:

H_s / H_d -- **III**

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

$H_{max} = \{1+0.8(\mathbf{III})\}(H_d)$ ----------------- **IV**

v. Calculated maximum scouring depth (ΔZc) is determined by using the following formula:

$\Delta Z_c = (\mathbf{IV}) - H_d$ ---------------------------- **V**

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

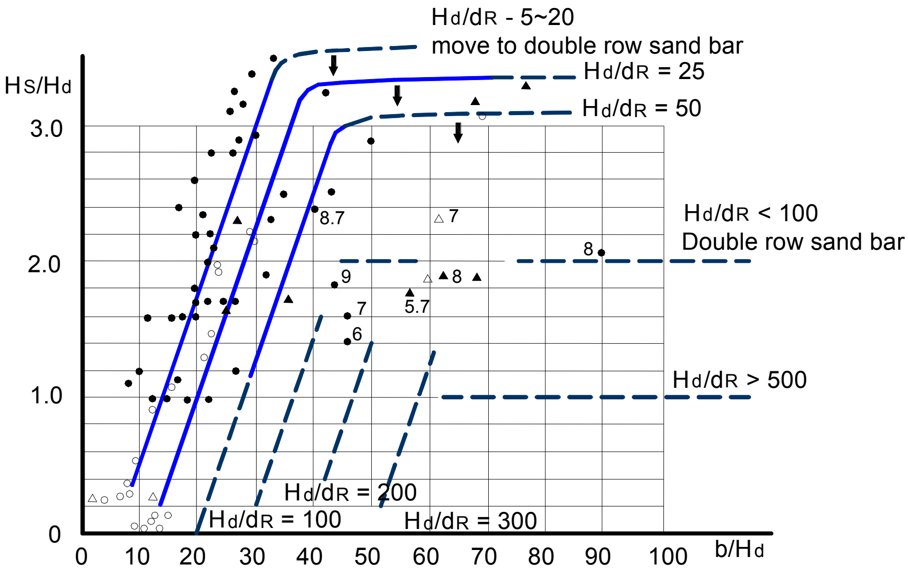


Figure 2.4.3c Relationship of H_s/H_d ~ b/h_d

Reference : _Guidelines for Disaster Restoration Works for Conservation of Precious natural surroundings_, Japan Oct. 1998.

In case of b/Hd > 10 and 0.2mm < d_r < 2cm

When the ratio b/Hd exceeds 10 (b/Hd > 10) and the riverbed is formed by coarse sand and medium sand, fish scale sand bars are generally developed. In this case, height of bar becomes higher due to integration of sand bars.

i. The calculated maximum scouring depth in the gravel riverbed shall be calculated according to the procedures mentioned above.

ii. The computed maximum scouring depth in coarse sand and medium sand, ΔZc should be multiplied by **1.5**, considering integration of sand bars, that is:

$\Delta Zc = 1.5\ (\mathbf{V})$ ---------------------------- **VI**

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

b) Maximum scouring depth of the curved waterway

Calculated maximum scouring depth (ΔZc) of the curve waterway is calculated from the ratio of width of the waterway (b) and waterway curve radius (r).

i. The ratio of water depth of the maximum scouring portion (H_{max}) and average water depth (H_d) is decided from figure below by using the ratio of waterway width (b) and waterway curve radius (r).

H_{max} / H_d ---------------------------- **I**

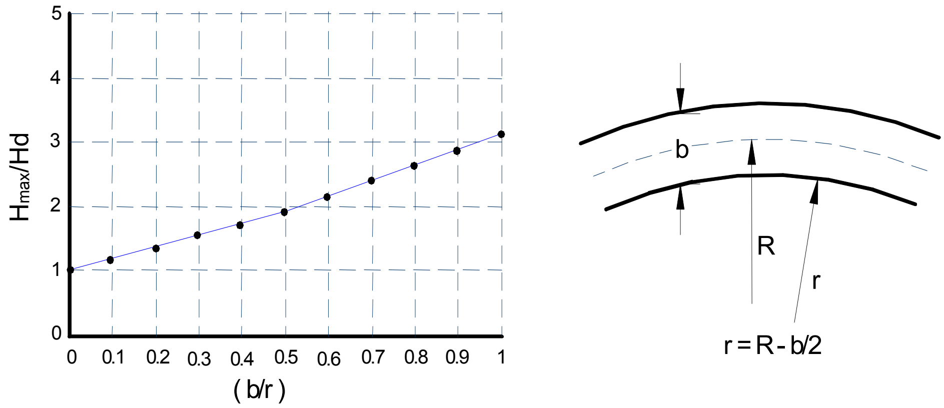


Figure 2.4.3d Relationship of H_{max}/H_d ~ b/r

Reference : <u>*Guidelines for Disaster Restoration Works for Conservation of Precious natural surroundings*</u>, Japan Oct. 1998.

ii. Water depth of the maximum scouring portion (H_{max}) is obtained by :

H_{max} = (**I**) (H_d) ---------------------------- **II**

iii. Calculated maximum scouring depth (ΔZc) is calculated by :

ΔZc = (**II**) – H_d ---------------------------- **III**

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

2.4.4 Segment Length

The length of one segment of revetment in the longitudinal direction should be less than 50 meters in order to prevent the extension of damage once one section of revetment collapses. Edge of the segment end shall be adequately filled with joint material (mortar) to connect with the adjoining revetment.

Structure of the partition works shall be the same as the end protection work.

2.4.5 Slope

After the determination of height of the slope covering work, the slope shall be planned based on the following:

1) The slope of the revetment shall be the same as the dike at 2:1 (horizontal and vertical, respectively) or milder. In case when the slope of revetment should be steeper than the dike, it shall be gentle as much as possible for stability purposes and shall be based on the natural slope of the adjacent bank.

2) In case of rapid flow stretches wherein floodwater contains a large quantity of boulders or gravels, the slope shall not be necessarily gentle but shall be milder than 0.5:1.

3) In case of joint portion with a rock-strewn slope, the slope of revetment shall be gradually changed to smoothly connect with the natural slope.

4) For the retaining wall type revetment, a maximum slope of 0.3:1 shall be observed considering stability and the resulting residual hydraulic pressure.

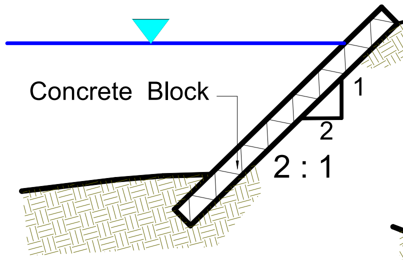


Fig. 2.2.4.5.1 a Revetment with -2.0 : 1 slope

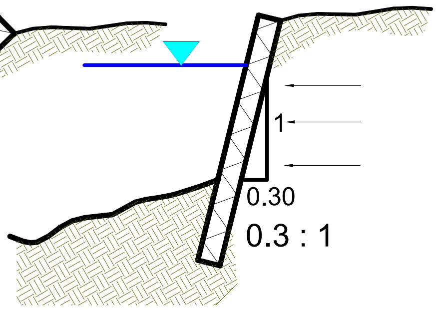


Fig. 2.2.4.5.1 b Revetment with 0.3:1 slope is for rubble concrete, reinforced concrete, and gravity wall type.

2.4.6 Berm Arrangement:

1) If the height of revetment is more than 5.0 meters, berm (banquette) must be provided, it shall be designed to separate the revetment into segments. Site condition must be considered also.

2) Berm is provided for stability and construction convenience of the revetment. Berm shall be at least 1.0 meter in width. It is also provided for a dike with height of more than 5 meters with a width usually more than 3.0 meters. Therefore, the width of the berm for revetment becomes more than 3.0 meters.

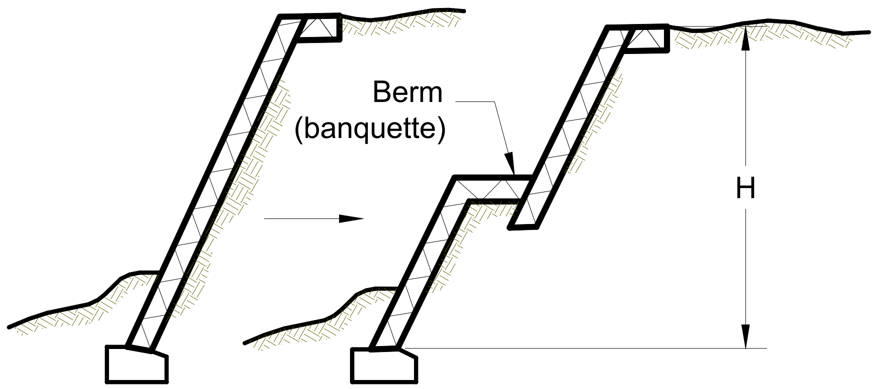


Figure 2.4.6.2b Case when H > 5 meters

2.4.7 Thickness

The thickness of revetment is generally based on the flow velocity, sediment runoff likely to occur in the proposed improvement stretch, soil and groundwater pressure at the back of revetment and other associated factors. Minimum overall thickness shall be 300 mm for all types of revetment, except for reinforced concrete type.

2.5 Foot Protection Works

2.5.1 Basic Concept

Foot protection works shall be adequately placed in front of the revetment foundation to prevent scouring. The foot protection shall have a minimum width of 2.0 meters towards the centerline of stream. In some cases, it is very difficult to set the foundation if the ordinary water level area is so deep and is influenced by high tide. However, if scouring is likely to occur down to the level of the deepest riverbed, the foundation of revetment should be placed deeper. In such case, steel sheet pile or concrete sheet pile should be provided with provision of adequate foot protection works in front of the sheet pile foundation to prevent scouring.

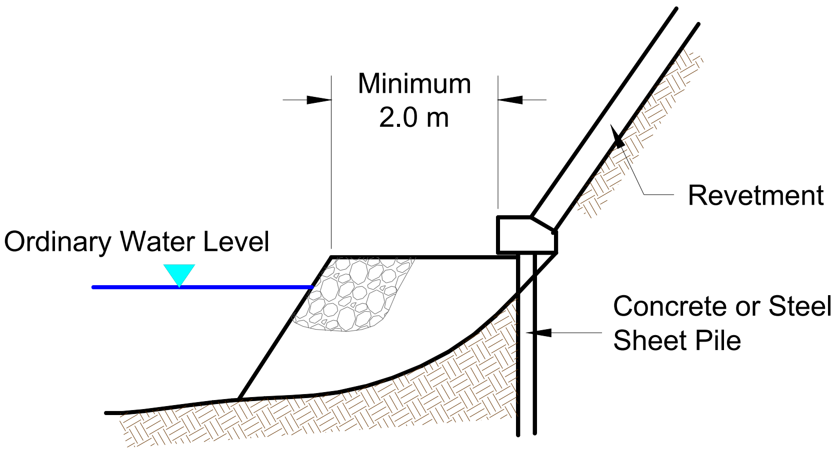


Fig. 2.5.1 Protection Against Scour

2.5.2 Type of Foot Protection Works

The type of foot protection work shall be determined based on river conditions, convenience in construction, economy, etc. The basic requirements for the foot protection work are as follows:

- Sufficient weight against the flow forces.
- Sufficient width to prevent scouring in front of the revetment.
- Durability
- Flexibility for the fluctuation of riverbed.

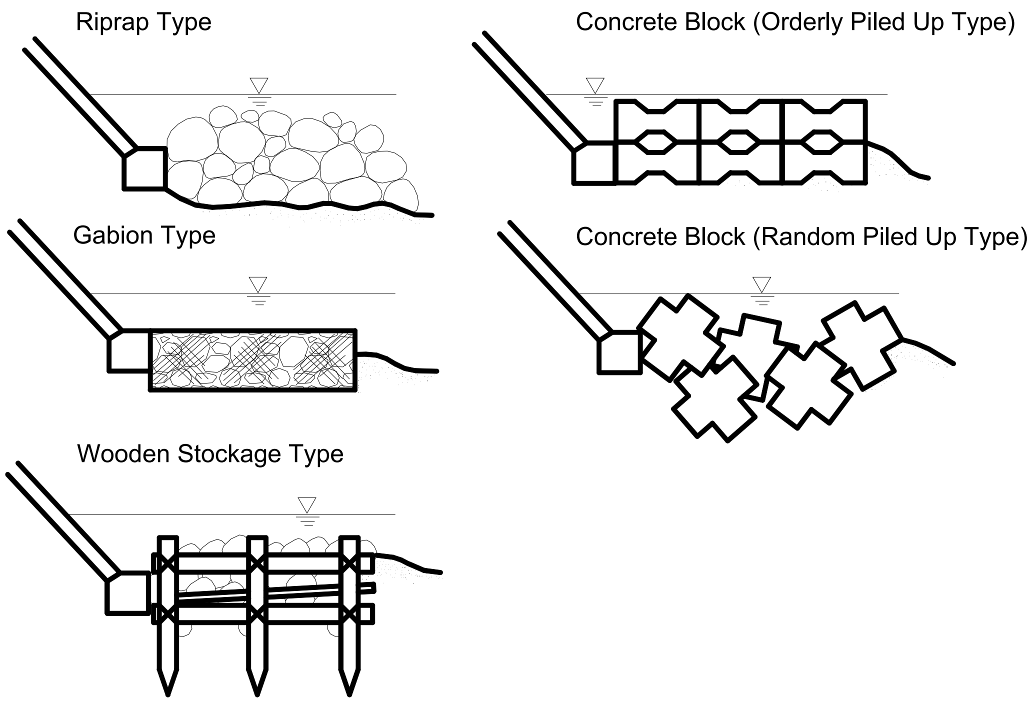


Figure 2.5.2 Types of Foot Protection Work

2.5.3 Design of Foot Protection Works

1) Riprap Type:

The minimum diameter of the boulder shall be determined based on the table below. The larger boulder shall be used at the toe of slope and slope surface. Outflow of materials from the foundation is unavoidable; therefore, proper maintenance shall be carried out.

Table 2.5.3a Minimum Diameter of Boulder (Riprap Type)

Design Velocity (m/s)	Diameter (cm)
2	-
3	30
4	50
5	80
6	120

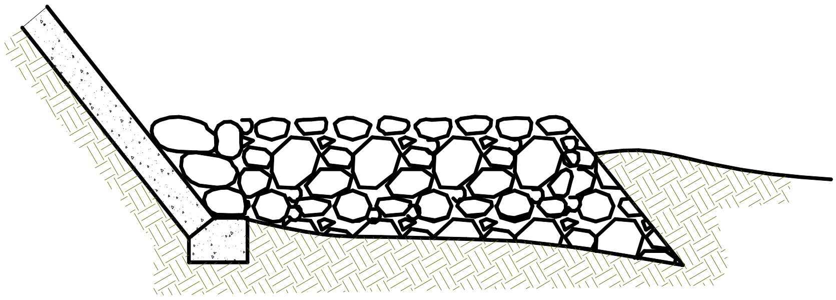


Figure 2.5.3a Riprap Type

2) Gabion Type:

This type shall not be used for rivers with saline water intrusion and for rivers with riverbed and banks consisting of boulders. The gabions shall be connected to each other. The diameter of the filling boulders shall be determined based on the table below.

Table 2.5.3b Diameter of Filling Boulder (Gabion Type) (Unit: cm)

Water Depth (m)	**Design Velocity (m/s)**					
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**
1.0	5-15	5-15	5-15	10-20	-	-
2.0	5-15	5-15	5-15	5-15	15-20	-
3.0	5-15	5-15	5-15	5-15	15-20	15-20
4.0	5-15	5-15	5-15	5-15	5-15	15-20
5.0	5-15	5-15	5-15	5-15	5-15	15-20
6.0	5-15	5-15	5-15	5-15	5-15	15-20

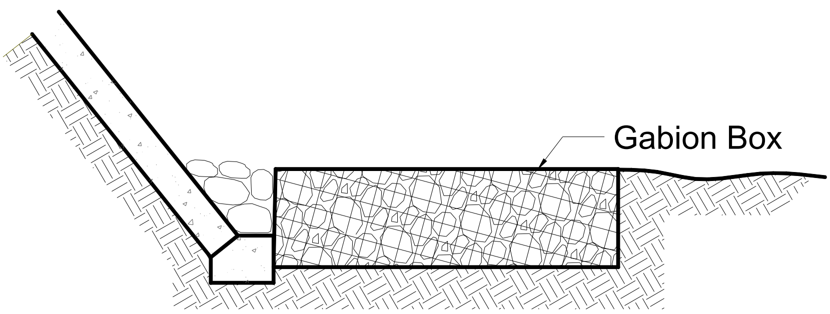


Figure 2.5.3b Gabion Type

3) Wooden Stockade Type:

Wooden Stockade Type is preferable for a river with few boulders. The space between the wooden piles shall be determined based on the diameter of the filling materials. Concrete piles are preferably used instead of wooden piles in some cases.

The minimum diameter of the filling boulders shall be determined based on the table below.

Table 2.5.3c Minimum Diameter of Filling Boulder
(Wooden Stockade Type) (Unit: cm)

Water Depth (m)	Design Velocity (m/s)					
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**
1.0	5	5	10	30	-	-
2.0	5	5	10	15	35	65
3.0	5	5	10	15	25	45
4.0	5	5	5	15	25	40
5.0	5	5	5	10	20	35
6.0	5	5	5	10	20	30

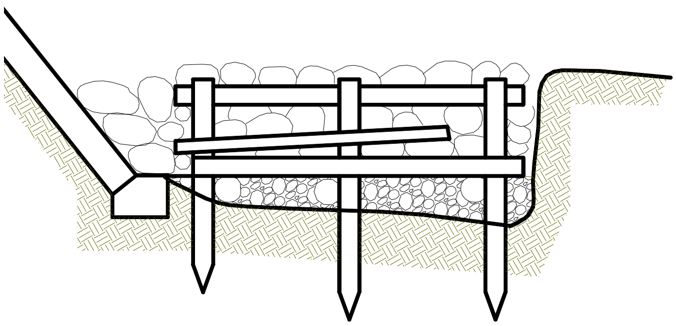


Figure 2.5.3c Wooden Stockade Type

4) Concrete Block Type:

Concrete Block Type consists of two (2) types: the order pile up type and the disorder pile up type. The weight of the block shall be determined based on the figure below:

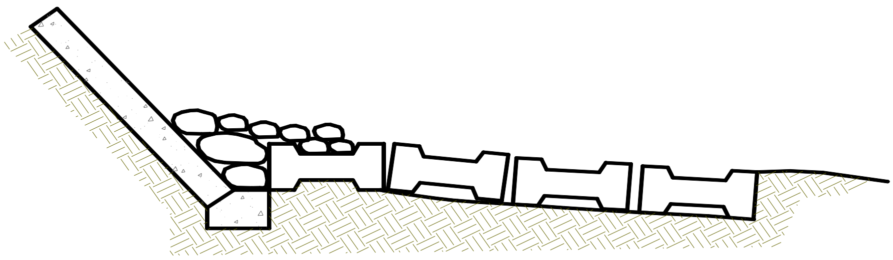
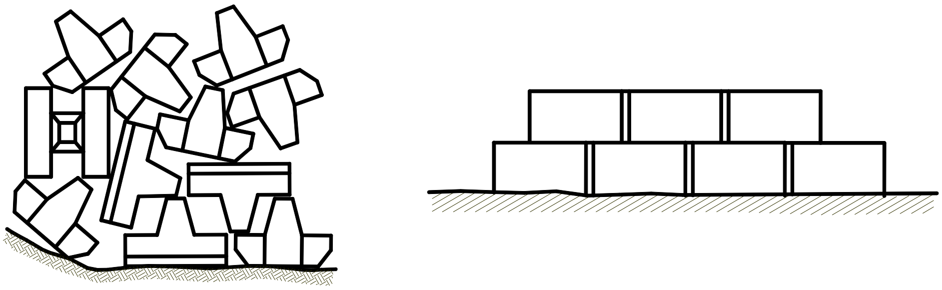


Figure 2.5.3d Concrete Block Type (Single Unit)



Random piled up **Orderly piled up**

Figure2.5. 3e Weight of Concrete Block

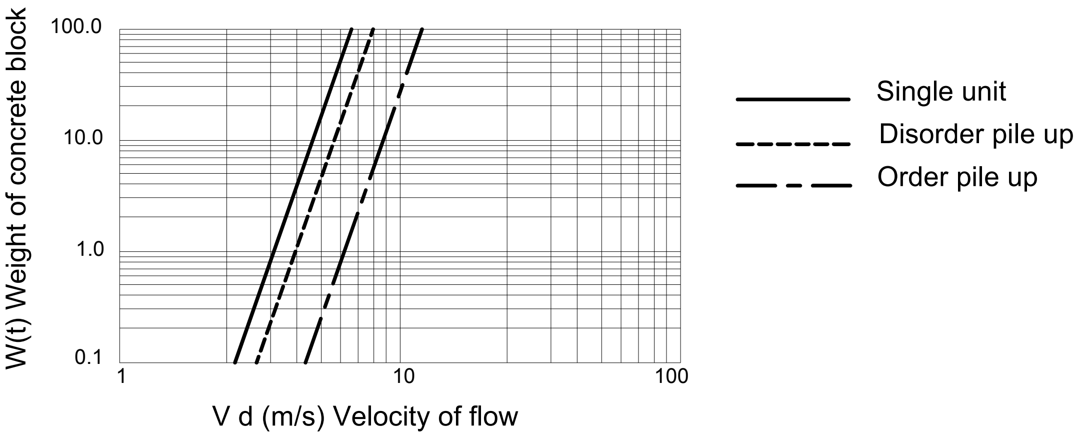


Figure2.5.3f Comparison of Type of Concrete Blocks

2.5.4 Top Elevation of Foot Protection Work

The top elevation of foot protection work shall be at the same elevation as the top of the foundation work of the revetment.

In order to prevent scouring, the top elevation of foot protection work may sometimes be set above the top of foundation work of the revetment. When the thickness of the foot protection work is more than 1 m, the bottom elevation of the foot protection work shall be set at the same elevation with the bottom of the foundation work.

2.5.5 Width of Foot Protection Work

The foot protection work requires sufficient width that will prevent scouring of riverbed in front of the foundation work of the revetment.

The foot protection work shall consider flat width of at least 2m in front of the revetment after the scouring. The required width of the foot protection work (B) is as follows:

$$B = Ln + \Delta Z / \sin\theta$$

where,

Ln: Flat width in front of revetment (at least 2 m)

θ: Slope at the scouring (Generally, 30 degrees can be assumed)

ΔZ: height between the foot protection work and the scoured bed.

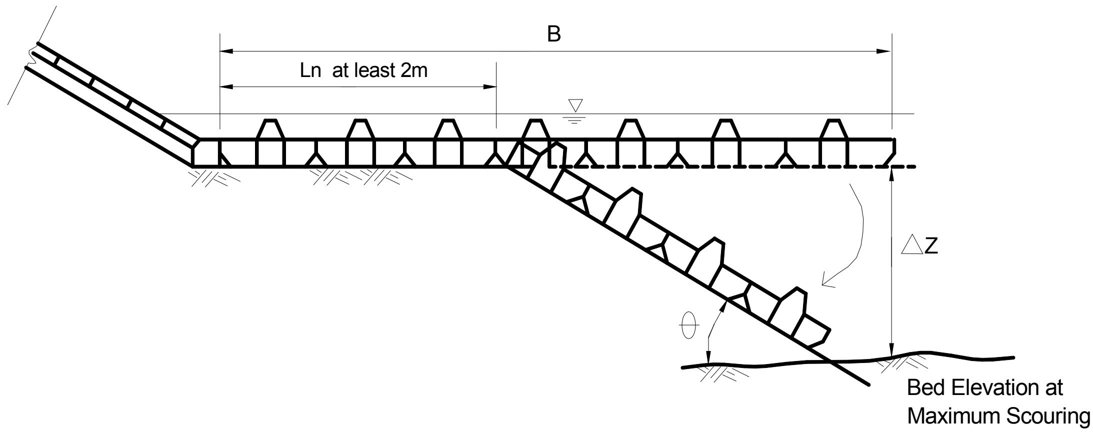


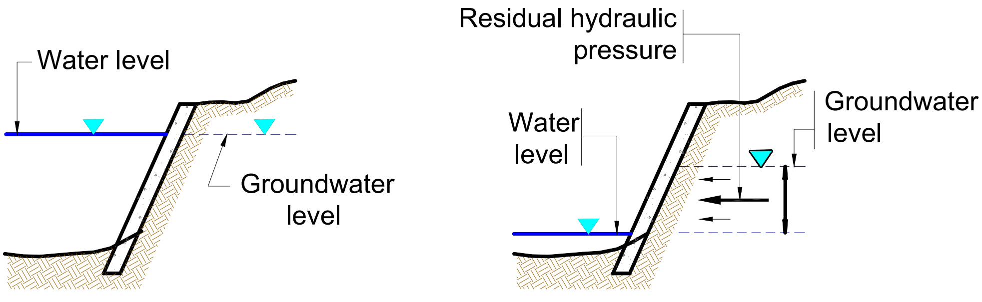
Figure 2.5.5 Width of Foot Protection Work

Foot protection work is planned in order to protect the revetment foundation from local riverbed scouring and/or degradation of riverbed. Foot protection reduces the force of flow at the foundation, thus reduces abrupt scouring of riverbed. Basically, upper surface level of foot protection is set below the original riverbed or designed riverbed. However, if foot protection functions as spur dike, or if it is difficult to provide foot protection below the riverbed due to high water level, then the foot protection can be placed on the original riverbed or designed riverbed with due consideration to the regimen of the stream, river cross-sectional area, river flow direction and type of revetment.

2.6 Other Design Consideration

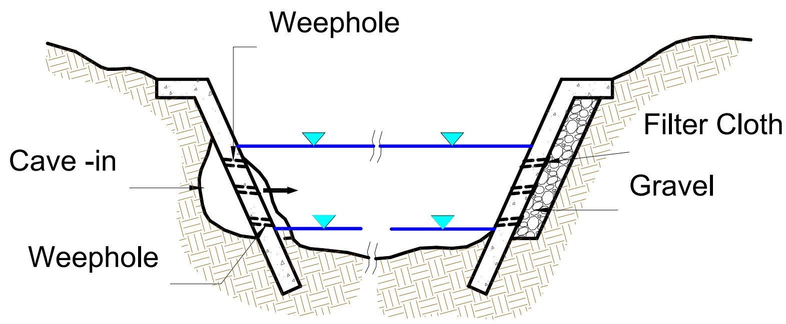
2.6.1 Drainage Pipe / Weep Hole

Drainage pipes/weep holes shall be designed and provided for both types of revetment for diked and non-diked rivers. During flood times, the rise of flood water level in the river almost coincides with the rise of groundwater behind the revetment especially when the ground is already saturated. After the floods, the rate of subsidence of floodwater in the river is usually greater than the recession of groundwater level behind the revetment without drainage pipes/weep holes. If the disparity between the subsiding floodwater and groundwater stages is significantly high, residual hydraulic pressure exists at the back of the revetment which might become higher (Figure 2.6.1a). Weep holes shall be provided in the revetment using 50~75 mm diameter PVC drainpipes, placed in stagger horizontal direction and spaced 2 meters center to center. One of the main causes of caving in of soil particles behind the revetment is the outflow of backfill fine materials through the joints of revetment and weep holes, which eventually leads to the collapse of the revetment. Moreover, pervious materials consisting of crushed gravel or geo-textile is placed between the revetment and original ground to prevent the outflow of the bank materials through the weep holes. The lowest weep holes shall be installed just above the ordinary water level. (Figure 2.6.1 b)



(a) During flood (b) After flood

Fig. 2.6.1a Development of Residual Hydraulic Pressure without Drainage Pipes/Weep Holes



a) without filter cloth and gravel b) with filter cloth and gravel

Fig. 2.6.1b Weep Hole

2.6.2 Backfilling Materials

- For rigid type revetment, backfilling materials shall be installed in order to reduce the residual water pressure to the covering work and to fix the covering work to the original bank slope.
- For permeable type revetment such as wooden fence type and gabion mattress type, the backfilling materials shall not be installed.
- The backfilling materials shall be with high permeability, such as crushed gravel, etc.
- Thickness of the backfilling materials shall be 30-40 cm for wall type and 15-20 cm for pitching or lining type.
- In case of the site with high residual water pressure, such as revetment of the excavated river, weep hole shall be installed.

2.6.3 Outflow Prevention Materials

- Basically, the outflow prevention materials (e.g. filter cloth) shall be installed behind the permeable type revetment.
- For the impermeable type revetment, the outflow prevention materials shall not be used.

2.6.4 Strengthening Upper and Lower Ends

Generally, the end points of revetment are always subjected to external forces which make these portions of the structure become weak and prone to damage or possible collapse. In constructing a piece-meal project, temporary protection works (e.g., boulder and gabion) shall be provided.

The end protection work is indispensable to the rigid structure type revetments.

The end protection shall cover the extent of the covering work and crest work.

The thickness of the end protection work shall be from the surface of revetment up to the backfill material. The thickness of the end protection shall be more than 50 cm.

Most scouring occurs at the upstream and downstream ends of the revetment. The scouring develops sucking out of backfill materials resulting to the gradual destruction of the revetment. Therefore, the revetment ends shall be strengthened by making it massive/thick and by providing structures like gabion/boulder which are called “transition” works of the revetment.

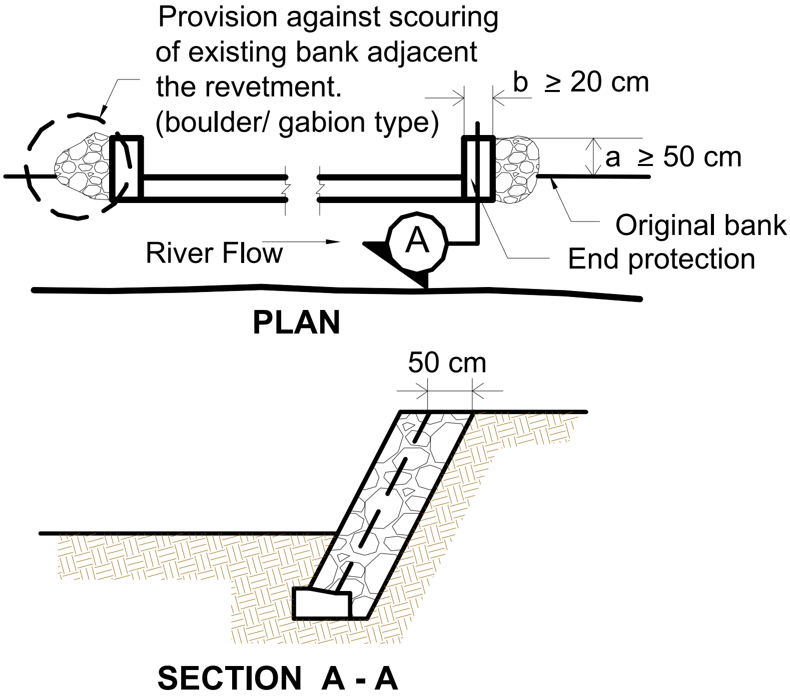


Figure 2.6.4.1 End Protection Works

Angle of Transition Work

- A transition work to the natural bank is installed in order to connect the revetment and the natural bank smoothly and to prevent erosion at the upstream and downstream sides of the revetment from spreading behind the revetment.

- A transition work shall be of flexible type like gabion mattress.

- The fitting angle to the natural bank shall be less than 30° at the upstream side and less than 45° at the downstream side. However the fitting angle shall be determined based on the present condition of bank.

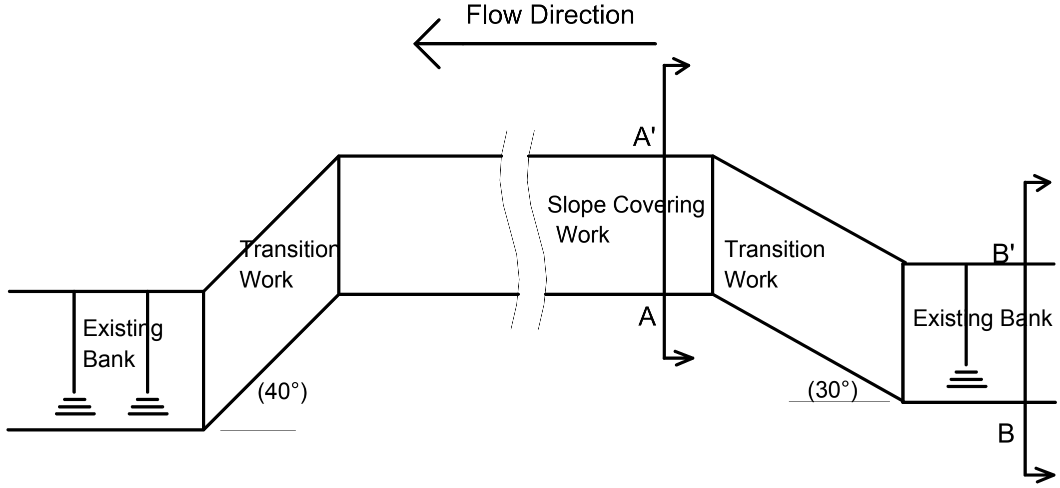


Figure 2.6.4.2. Fitting Angle of Transition Work

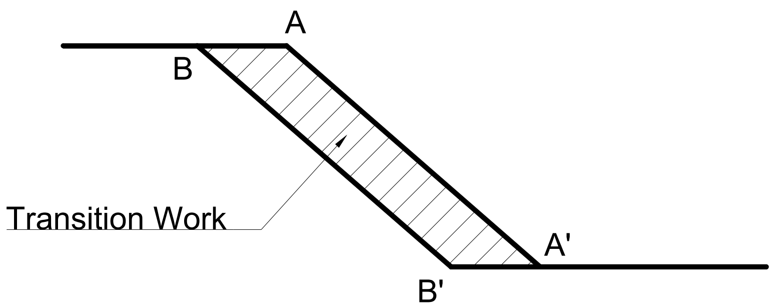


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Figure 2.6.4.3 Cross-Sectional View

2.6.5 Protection of Revetment Crest

For non-diked rivers, if the overflow frequency is very high due to inadequate flow capacity, it is necessary to plan the protection of the crest. Basically crest protection is planned for the low water channel revetment in a compound cross section waterway if the frequency of flow on the high water channel is high. Once overflow exists or reaches the high water channel, damage possibility is very high at the shoulder of the revetment.

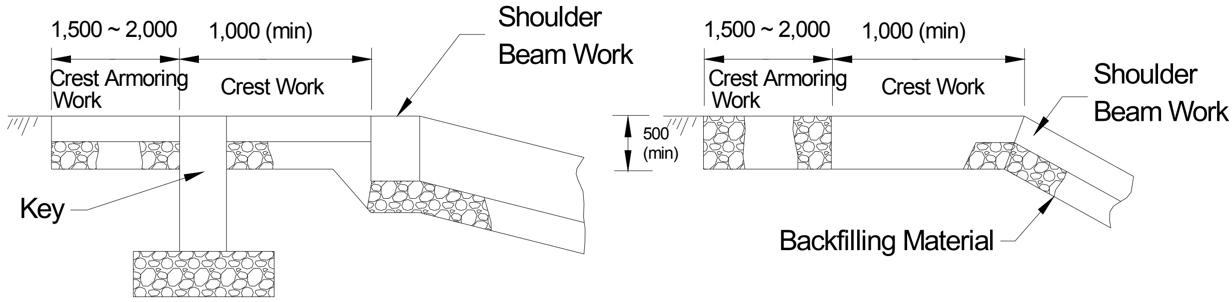


Figure 2.6.5 Crest Protection

- The width of crest work shall be more than 1m.
- The height of crest end work shall be more than 0.5 m.

2.6.6 Bridge Site and Tributary Confluence

At the upstream and downstream portions of the bridge, sluice gate and culvert, weir, groundsill and confluence of rivers, the river flow is constricted by the presence of these structures, which change the river conditions. It is, therefore, necessary to provide adequate length of revetment in these areas to prevent bank erosion due to the adverse effects of constricted river flow.

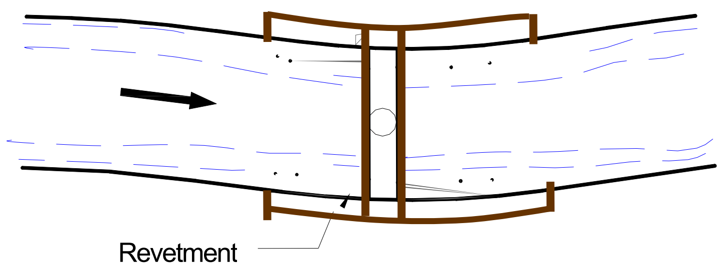


Figure 2.6.6 Revetment at bridge abutment

2.6.7 Structural Change Point (Construction Joint)

Revetment should be constructed continuously with no structural change point. Destruction might happen where the revetment slope has suddenly changed, such as at construction joints. In such case, the joints should be adequately strengthened by providing reinforcing bar with mortar.

2.7 Main Causes of Revetment Damages

In order to design a stable revetment, the main causes of damages must be known and understood.

1) Local scouring and riverbed degradation

   The scouring at riverbed along foundation of revetment is one of the main causes of revetment damages.

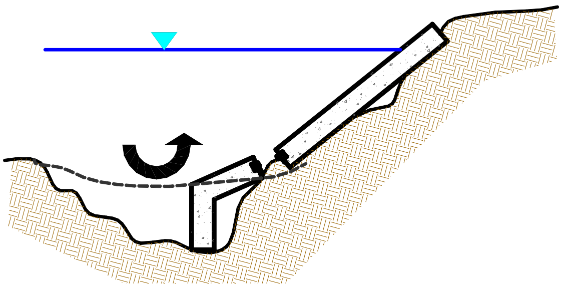


Figure 2.7a Local scouring and riverbed degradation

2) Movement/extraction of particle/block caused by high velocity flow.

   Particle(s)/block(s) (e.g., dry boulder riprap) of revetment are detached by strong velocity flow.

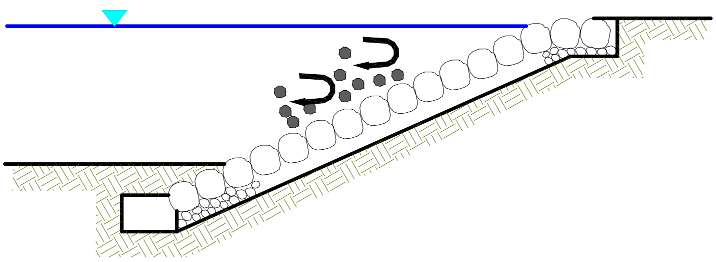


Figure 2.7b Movement/extraction of particle/block caused by high velocity flow

3) Damage at the end section due to direct water attack and scouring

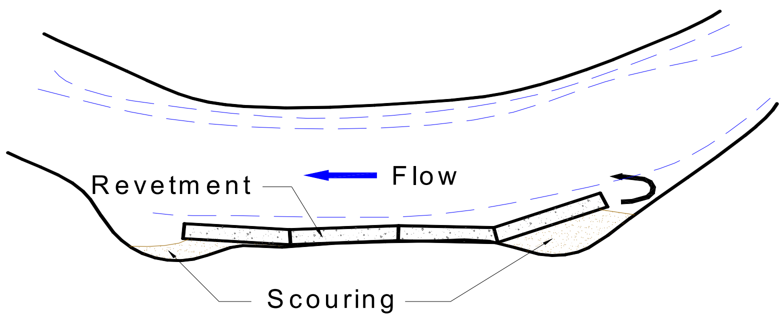


Figure 2.7c Damage at the end section due to direct water attack and scouring

4) Outflow of fine materials behind the revetment

The fine materials behind the revetment are sucked out from the crevice/weep hole of revetment.

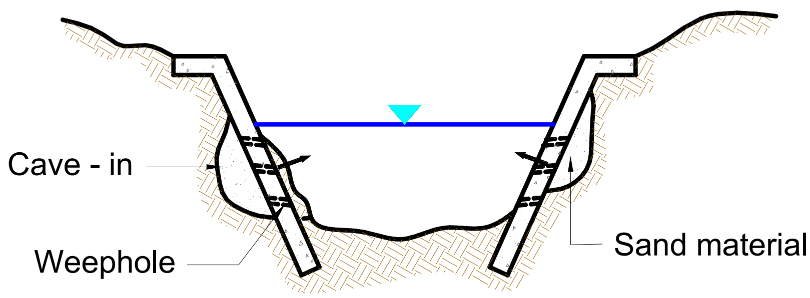


Figure 2.7d Outflow of fine materials behind the revetment

5) Residual water pressure

When the floodwater level is receding, residual water pressure of the remaining groundwater at the back of the revetment may create piping. In case of steep slope revetment, the residual water pressure and earth pressure causes the revetment to collapse.

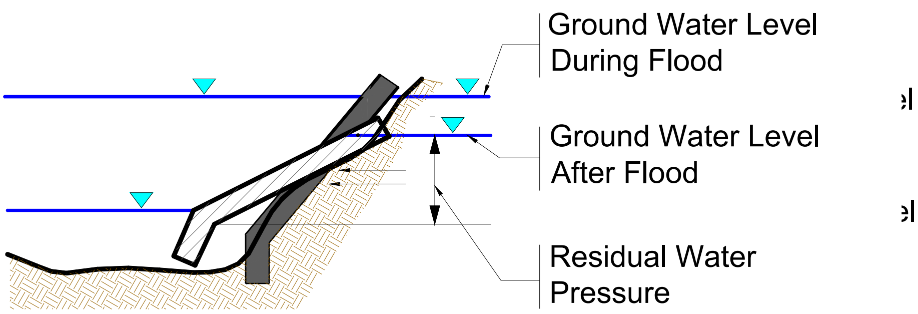


Figure 2.7e Residual water pressure

6) Erosion on the top of the revetment

When the floods overtops the revetment and flows back to the river, the back portion of the top of revetment might be damaged.

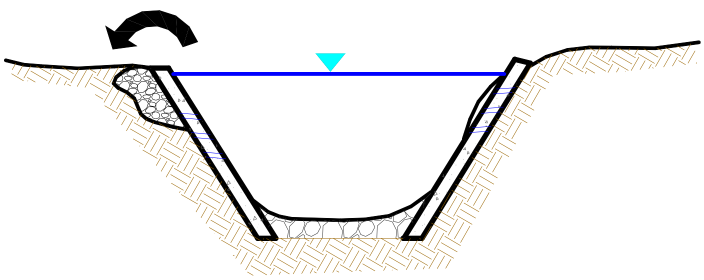


Figure 2.7f Erosion on the top of the revetment

7) Direct hit by big boulder and/or logs

Logs and rocks carried by strong river flow directly hit the revetment resulting to damages.

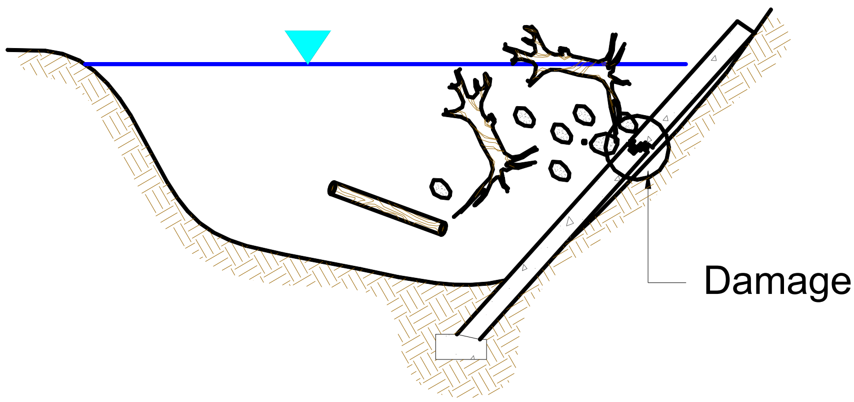


Figure 2.7g Direct hit by big boulder and/or logs

Chapter 3 SPUR DIKE

3.1 Basic Concept

A spur dike is a river structure with the following functions:

1) Increases the flow roughness and reduces the flow velocity around the riverbank.
2) Redirects river flow away from the riverbank.

Corollary to the above functions, installation of spur dikes has the following purposes:

1) Prevents bank erosion and damage to revetment.
2) Deepens water depth for navigation.

This chapter deals mainly for planning and design of spur dike as an urgent measure in preventing bank erosion and damage to revetment.

The causes of bank erosion, scouring or riverbed degradation and/or damages of banks must be analyzed thoroughly to establish an appropriate river improvement plan. At present, planning and design of spur dikes and its effect are not quantitatively verified but analyzed from the past records.

The following shall be considered during planning and design of spur dike:

1) Type of the spur dike shall be determined based on the ranking of bank (see Section 3.2.1 "Ranking of Bank") and the river segment of the site.
2) Materials of the spur dike shall be determined based on the estimated external forces (velocity of flood flow) at the design flood.
3) Spur dike shall be installed step by step through confirmation of the effectiveness of existing spur dikes.

3.2 Selection of Types

Basically, spur dikes are classified into permeable and impermeable/semi-permeable types, as described below:

1) Permeable Type

   Spur dike of this type is made of piles and frames, preferably in series. Its purpose is to reduce the river flow velocity at the immediate downstream of the spur dike and induce sedimentation. In cases where piles cannot be driven due to the presence of boulders on the riverbed, crib frame, skeleton works or concrete block type shall be used.

2) Impermeable/semi-permeable type

   This type of spur dike is made of wet masonry (impermeable) or concrete blocks and loose boulder (semi-permeable), preferably in series. Its purpose is to divert the river flow direction away from the riverbank.

Impermeable/semi-permeable type is classified as:

a) Overflow Type

Its main purpose is to reduce the river flow velocity. This type of spur dike shall be considered as a series of spur dike. At least three (3) spur dikes shall be planned.

b) Non Overflow Type

Its main purpose is to change the river flow direction away from the riverbank. This type of spur dike shall be considered as strong structure.

Any damage on the spur dike, especially at the tip, as a result of strong velocity during floods or sediment runoff, etc., may not be considered as a major problem provided that the structure functions are achieved in relation to its intended purpose.

3.2.1 Ranking of Bank

The evaluation of bank vulnerability is the basic consideration for the application of the spur dike. The bank shall be ranked according to the following procedure:

1) Calculate the bank heights of the respective cross-sections.

   Bank height = Elevation of bank shoulder – Average riverbed elevation.

2) Calculate the average bank height (H_b) from the above values

3) Estimate the possible bank erosion width during a flood (b_e) from H_b; as follows:

   $b_e = 5 H_b$

4) Find the minimum distance (D) from the bank to the protected area with the cross sectional profiles.

5) Evaluate bank vulnerability as follows:

   Rank A: D < be
   Rank B: D $\geq$ be
   Rank C: Bank consists of rock or consolidated sediment. No property around bank.

3.2.2 Determination of Type of Spur Dike

Structural type of the spur dike shall be determined from the available materials, economy, convenience of construction works, etc. Basing on the river segment, the following are considered:

1) River Segment and Countermeasure for Bank Erosion

For the determination of the countermeasure for bank erosion, the river segment and the ranking of bank based on the vulnerability of the bank against a flood are important factors.

a) Ranking of Bank

Banks will be ranked as follows in accordance with the risk of bank against a flood:

Rank A: Banks under condition that the damage of bank erosion caused by a flood influences the protected area/embankments during the same flood, and is likely to reduce the safety of the protected area/embankments.

Rank B: Banks vulnerable to erosion without any protection works, with the protection area/embankments remaining safe against any bank erosion caused by a single flood; and banks provided with erosion protection, not directly influenced by the damage to the protection works.

Rank C: Banks unlikely to be eroded or even if eroded, not expected to suffer progressive erosion. Rank C banks include banks inside the bend, and those consisting of rocks or consolidated sediment.

b) Countermeasure for Bank Erosion

Riverbanks with Rank C do not require any countermeasures for bank erosion, generally. For riverbanks with Ranks A and B, the countermeasures for bank erosion need to be compatible with the channel characteristics and hydraulic conditions where the countermeasure is installed.

2) River Segment and Type of Spur Dike

a) Segment 1 (alluvial fan):

The bank shall be protected by revetment and foot protection work. Spur dike is installed usually in addition to foot protection work.

Installing spur dike with 1.5 to 2.0 meters height could induce deposition in front of the bank, which establishes protection line (connecting spur dike tips) against scouring.

In case the bank is ranked A and the scouring portion is fixed, it is difficult to protect the bank by revetment and foot protection alone; hence, it is proposed in addition to install a group of spur dikes to redirect the flow direction. Spur dikes shall be impermeable type with high crest, to avoid damage of revetment by the overflow from the spur dikes.

b) Segment 2:

The bank with rank A shall be protected by revetment and foot protection work. In addition to improve the safety of the bank, when it is adjacent to the scouring portion, a group of overflow type spur dikes to reduce the river flow velocity is recommended Applicable types of the spur dike are as follows:

i. Stretches with bed materials of gravels: Impermeable or semi-permeable type
ii. Stretches with bed materials of sands: Permeable type

c) Segment 3:

A spur dike is not applied as countermeasure for bank erosion.

3.3 Design Flood

3.3.1. Scale of the Design Flood

Design Flood should be the same as determined in the master plan. But in case the master plan has not been formulated, the design flood of the proposed site shall be determined based on the following recommendations:

1) The flood corresponding to the experienced maximum flood is recommended as the minimum of design flood to avoid similar disaster.

2) In case where the beneficiary area is very important, the design flood higher than the experienced maximum flood shall be considered.

3.3.2 Selection of Representative Cross-sectional Profile

In case a spur dike is applied as a sole measure against bank erosion, the cross-section with minimum discharge capacity of target stretch shall be selected as the representative cross-sectional profile.

3.3.3 Design Water Level

The design flood level and the ordinary water level during rainy season are the basic considerations in the design of spur dike. The water levels at the site shall be as follows:

1) Set the design flood level at the representative cross section by using the master plan or by using the height of Maximum Experienced High Water Level;

2) Draw the average line of the ordinal water levels of respective sections on the longitudinal profile. This line shall be the ordinary water levels during rainy season.

3.3.4 Design Velocity

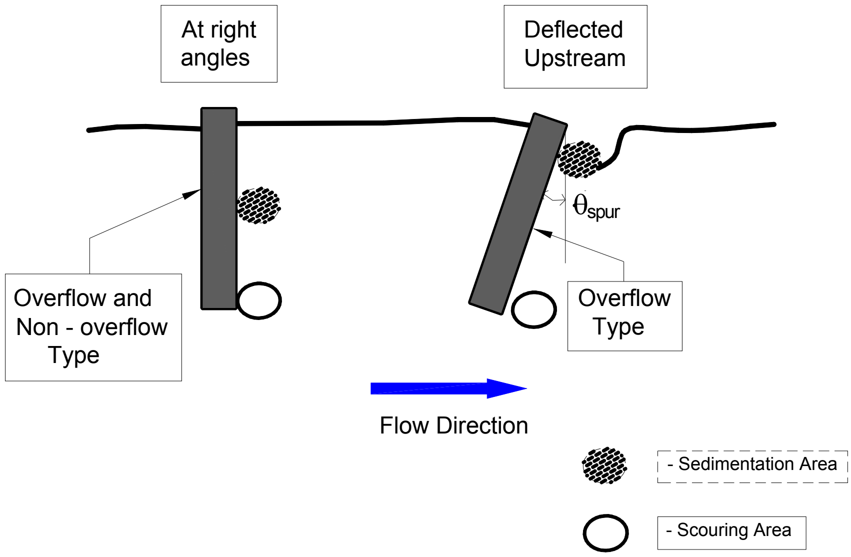
The mean velocity of the representative cross section at the design flood discharge shall be set as the design velocity.

3.4 Design Criteria

3.4.1 Direction of Spur Dike

The alignment of spur dikes deflected towards the upstream, should have an angle (θ_{spur}) between 10° to 15° with the line perpendicular to the riverbank at straight sections and 0° to 10° at flow attack zones. This type of alignment induces sedimentation at the foot/front of the riverbank immediately downstream that serves as protection for the toe of revetment and/or dike.

The right angle spur dike is usually adopted because of its average effects. The relationship between the alignment of spur dike and scouring/sedimentation is shown in Figure 3.4.1.



PLAN

Figure 3.4.1 Relationship between spur dike alignment and resulting sedimentation scouring

3.4.2 Dimensions

The dimensions and section of a spur dike shall be as follows:

The width of the crest of a spur dike shall range from 1 to 3 meters.

The height of low crest type spur dike shall be fixed within 10% to 40% of the distance reckoned from the average riverbed to the design flood level. Otherwise, the height shall be 0.5 to 1.0 meter above the ordinary water level during rainy season.

The height of high crest type spur dike shall be the same as the design flood level at the base of spur dike.

The crest slope of spur dike shall be 20:1 to 100:1 (Horizontal to Vertical).

The side slope of a non permeable spur dike in the upstream side ranges from 1:1 to 2:1, whereas in the downstream side ranges from 1:1 to 3:1, (Horizontal to Vertical).

For the concrete block or gabion type spur dike, the weight of a block or the grain size of filling boulders shall be determined in accordance with the design velocity, as shown in the foot protection of the revetment.

For the riprap (boulder) type, the grain size of the filling boulders shall be determined by adjusting the size mentioned in the foot protection work by the the following formula:

$$D = k\, D_m$$

$$k = \frac{1}{\cos\theta\sqrt{1 - \dfrac{\tan^2\theta}{\tan^2\phi}}}$$

where:

D : Minimum diameter of boulder

K : Adjustment factor

Dm: Diameter of boulder derived from table in the foot protection work.

θ : Slope of riprap

ϕ : Angle of repose of riprap in water
(Boulder: 38°, crushed stone: 41°)

W: Width of River

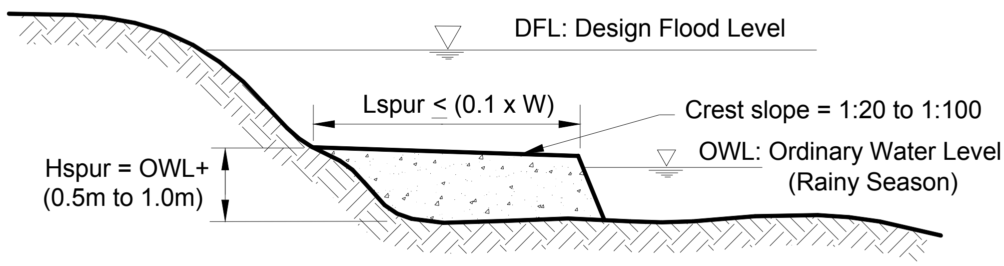


Figure 3.4.2a Dimensions of Spur Dike (Segment 2: Low Crest Type)

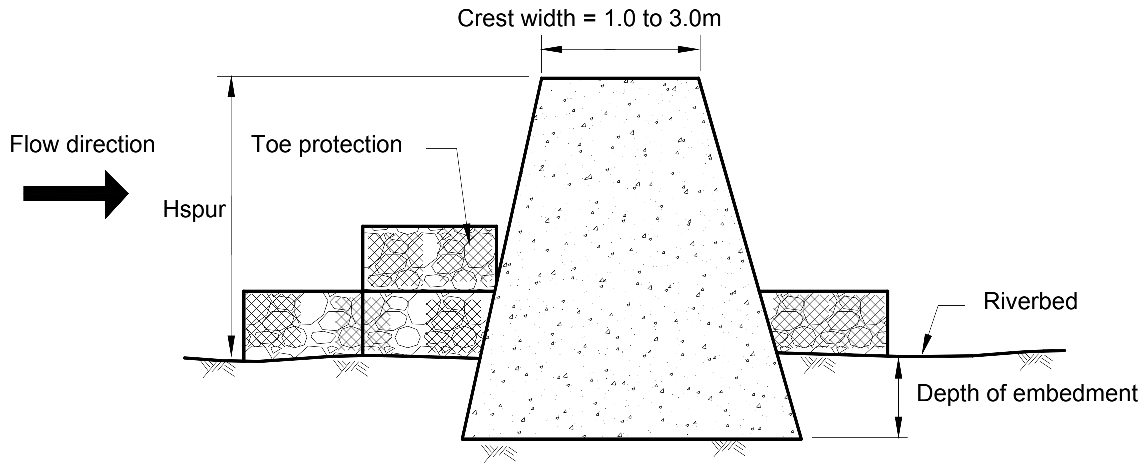


Figure3.4.2b Section of Spur Dike

3.4.3 Length

The length of a spur dike is generally 10% of the river width or less, but not to exceed 100m. The river flow capacity should be examined when the length of the spur dike is more than 10% of the river width (distance of left to right bank); or when the spur dike is to be constructed in a narrow river, since this could affect the opposite bank and considerably reduce the river flow capacity.

3.4.4 Spacing

The spacing of spur dikes shall be less than 2 times its length at flow attack zones and 2 to 4 times at straight sections.

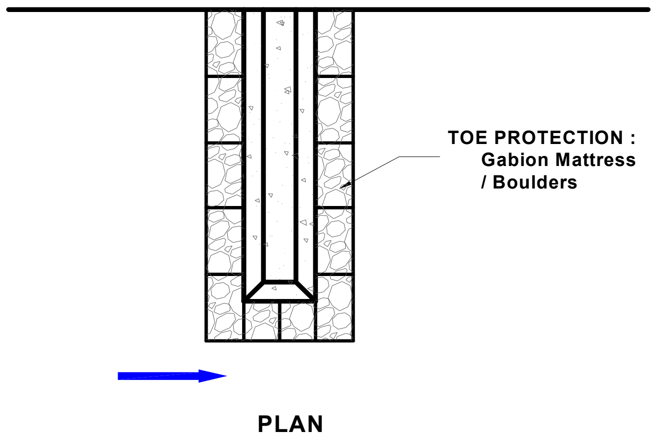
3.4.5 Embedment Depth

For concrete and stone masonry type spur dike, a minimum embedment depth of 0.5 m is recommended. For the permeable type (i.e., pile-type, crib-type, etc.), an embedment depth of 2/3 the pile length is recommended.

For gabion-type, boulder type and concrete block type spur dikes, only a provision of about 0.2 m layer of gravel before placement of the main body is sufficient.

3.4.6 Toe Protection Works

In case there is severe riverbed degradation or scouring, toe protection should be provided to prevent collapse of the spur dike. Riprap or gabion is used for toe protection work. The grain size of boulders for the riprap and gabion shall be determined in accordance with the design velocity, as shown in the foot protection of the revetment.



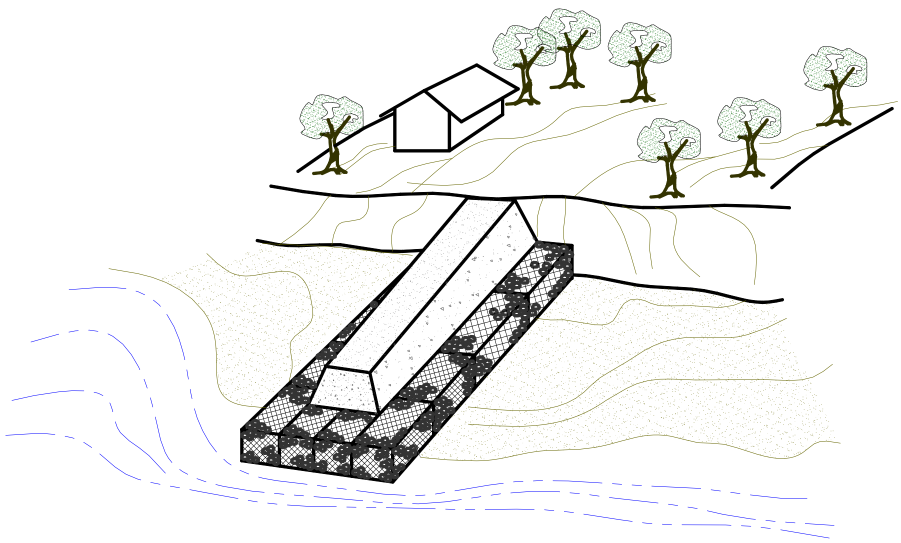


Figure 3.5.1 Toe Protection Works

3.4.7 Base protection

The base of spur dike is the joint to the bank or to the revetment usually prone to damage. Therefore, the gap between the base and bank shall be filled up by adequate materials, such as riprap and gabion.

Chapter 4 GROUNDSILL

4.1 Basic Concept

A groundsill is a river structure that prevents riverbed degradation, stabilizes the riverbed and maintains the longitudinal and cross-sectional profiles. Since groundsill is installed across the river channel, it has drastic influences on the river conditions and the ecosystem of the upstream and downstream stretches; hence, maintenance of groundsill requires great effort.

In general, groundsill is not highly recommended in river works. However, if groundsill is applied, the actual river conditions including the causes of riverbed degradation and the influences of groundsill are investigated thoroughly. To determine the shape and dimensions and to confirm the influences of the groundsill, hydraulic model tests are recommended. The corresponding negative impacts shall be mitigated or controlled.

From the above viewpoint, planning and design of groundsill shall be based on the master plan.

The groundsill consists of the following components:

1) Main Structure

A Groundsill is classified into two types; Drop Structure Type (with head) and Sill Type (without head).

Drop structure type is selected when there is an elevation difference between upstream and downstream. When there is small or no difference, still type is selected.

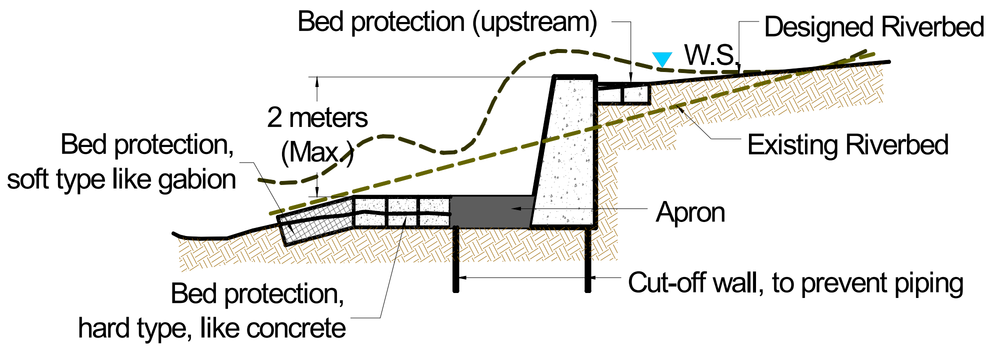


Figure 4.1a Plan, Drop structure type

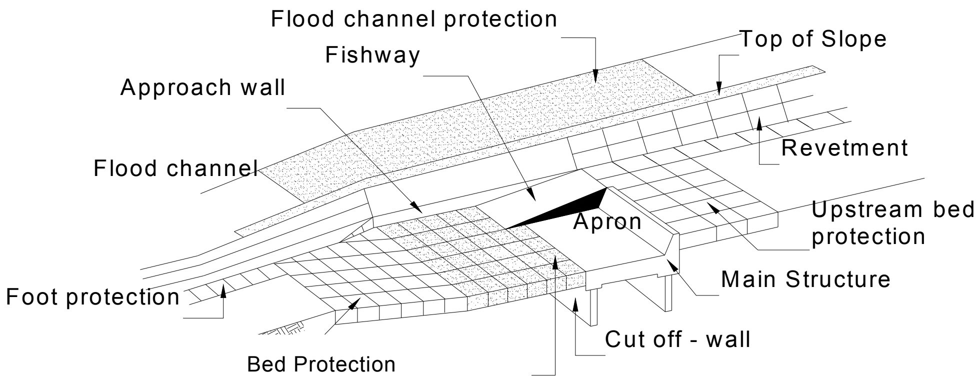


Figure 4.1b Perspective, Drop structure type

2) Apron

- Riverbed scouring protective structure, provided to prevent scouring which is caused by drop water.

1) Bed Protection

a) Upstream Bed Protection

Riverbed scouring protective structure - provided to prevent local scouring caused by increasing tractive force. At this portion, water level goes down, therefore flow velocity and tractive force increase.

b) Downstream Bed Protection

i. Bed protection (Hard type)

Riverbed scouring protective structure - provided from the water drop point after the point of hydraulic jump. In this section, super critical flow occurs. Usually this structure is made of concrete blocks.

ii Bed protection (Soft type)

Riverbed scouring protective structure - provided after the portion of hydraulic jump. Usually this structure is made of gabion mattress.

2) Approach Wall

Slope protective structure - provided to prevent scouring just after the main structure. Usually this structure is from 5 m upstream side of the main structure to the point after the hydraulic jump. This structure shall be designed as retaining wall.

5) Fish Way

Groundsill breaks the continuity of upstream and downstream water flow. It prevents the migration of fishes and other wildlife. When a groundsill is planned fish way shall be considered.

6) High Water Channel Protection

High water channel protective structure - provided to prevent scouring caused by overflow from low water channel. This structure shall be considered in a compound cross-section river.

4.2 Location and Alignment

When a stretch of the river bed is scoured by the river flow, and the foundation of structures, such as revetment, comes out; the risk of flood damage increase and sometimes accompanied by intake difficulties.

Groundsill is constructed where the river bed slope and bed height should be kept stable and according to the plan; respectively.

The portion with straight and parallel bank alignment is recommended.

The plane form of a groundsill shall be linear as a general rule. The direction shall be at right angles to the direction of river flow in the lower reaches in principle, (considering the direction of river flow on the occasion of flood flow).

The relationship between the plane form of groundsill and flow direction is shown in Figure 4.2.

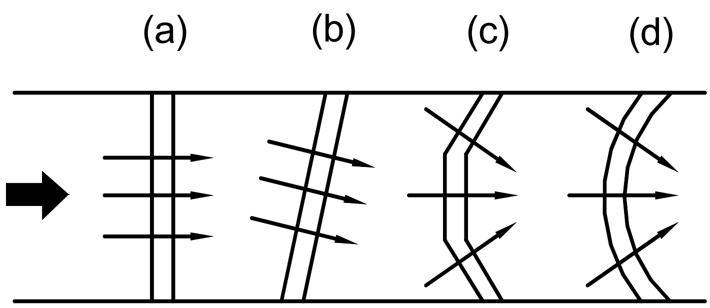


Figure 4.2. Plane Forms of Ground Sills and Flow Direction

1) Linear form at right angles to the flow direction:

   This is most commonly used and is less problematic for flood control and cheaper in work cost than other forms.

2) Linear form at an angle to the flow direction:

   This should not be used in principle, except in the case of meeting the flow direction in the lower reach of the groundsill, in consideration of the dike alignment in the lower reaches, etc. This is often seen in old agricultural intake weirs, etc. but often has bad affects on the river.

3) Polygonal form with a vertex at the center of river:

The midstream in the lower reach of the groundsill can be centrally collected. But it involves high cost, being liable to cause deep scouring in the lower reach, and the maintenance of the groundsill and the riverbed in the lower reaches becomes difficult.

4) Curved form with a vertex at the center of the river:

A circular arc of parabola is used mostly, but it has the same difficulty as the polygonal form.

4.3 Design Criteria

4.3.1 Height

The groundsill is provided to stabilize the riverbed, but it creates instability in the immediate lower riverbed reaches. In a river with considerable riverbed variation, the crest height must be decided in reference to the existing riverbed and future trends.

In general, the crest height of a groundsill shall coincide with the design bed height, and the standard height shall be less than 2 m.

Water level profile drops at the groundsill location accompanied by the sudden increase of flow velocity. This condition tends to lower the riverbed immediately upstream of the groundsill and should be considered in the determination of the crest height.

Both ends of the groundsill body shall be anchored sufficiently in the dike or revetment. In the lower reach of the groundsill, an apron shall be properly provided according to necessity.

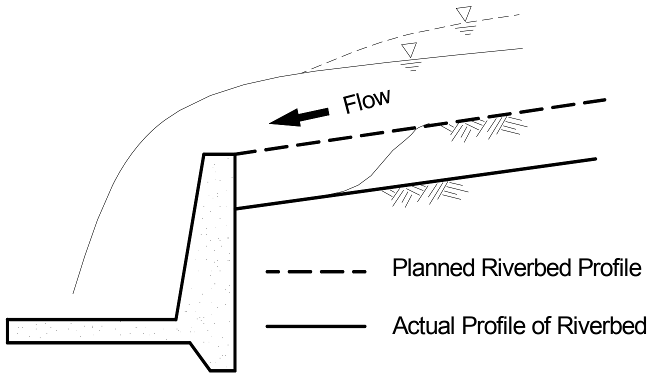


Figure 4.4.1.1 Flow at Upstream side of Drop Structure

Tendency of the riverbed variation at the upstream of the groundsill is summarized as follows:

Segment	Condition of Upstream Riverbed
1	Riverbed degradation is small.
2-1	Riverbed degradation is large.
2-2	Riverbed degradation is always large.

4.3.2 Apron and Mattress

The apron and mattress shall have the necessary lengths and the appropriate structures required for keeping the safety of the groundsill body.

Main causes of damage on the groundsill include runoff of water/sediment resulting to scouring especially in the downstream reach and water seepage below the body. As a countermeasure against scouring, sufficient length of apron and bed protection, like mattress for the upstream and downstream of the groundsill, must be provided. Against water seepage, cut-off walls like sheet piles must be provided.

There are several types of mattresses, namely; fascine, wooden, gabion, concrete, concrete block, etc. These are intended to reduce the effects of flowing force as practically as possible. Gradual arrangement from hard structure to soft structure should be adopted for making it flexible enough with the riverbed.

4.3.3 Embedment and Foundation

In a wide river, groundsill need not be embedded in the dike/revetment in order not to induce damage to the dike/revetment. In a narrow river especially with high velocity flow, embedment is necessary.

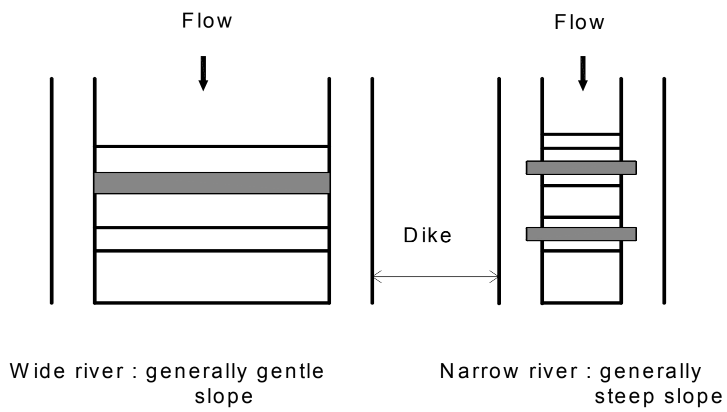


Figure 4.3.3 Embedment in the dike/revetment

The base of groundsill shall be placed on solid foundation. There are two types of foundation: spread type and pile type. When spread type is selected, the thickness of foundation shall be larger than 3 meters.

Chapter 5 DAM (Small Size)

5.1 Basic Concept

Dams are hydraulic structures constructed to control and/or conserve water. For purposes of flood control, dams are used to retard flood runoff and minimize the effect of sudden floods. In the design of these structures, the following are considered:

5.2 Location, Alignment and Selection of Types

1) In streams flowing between high rocky walls, a concrete overflow dam is recommended.

2) Low and rolling plains, an earthfill dam with separate spillway is recommended.

3) Rockfill dams are recommended in remote locations where cement and materials for an earthfill dam are not available.

4) For gravity dams these can be built on earth foundation, and their height in this case is limited to 20 metres.

5) Dams must be evaluated for earthquake loads and stresses.

6) Solid rock foundation has a high bearing capacity and resistance to erosion and seepage.

7) Gravel foundation, if well compacted is suitable for earthfill, rockfill and low gravity dams. This kind of foundation is often subject to water seepage at high rates and therefore needs special precautions to provide effective water cutoffs or seal.

5.3 Design Criteria

5.3.1 Freeboard

The freeboard allowance for dams is based on a water surface level determined by assuming an arbitrary discharge that might result from a possible emergency. Usually, an encroachment on the freeboard provided for the designed maximum water surface level is allowed considering the design of an emergency spillway.

5.3.2 Slope

1) The upstream slope for earthfill dams may vary from 2:1 to as flat as 4:1 for stability; usually it is 2.5:1 or 3:1.

2) Flat upstream slope for earthfill dams are sometimes used in order to eliminate expensive slope protection.

3) The usual downstream slopes for small earthfill dams are 2:1 where a downstream pervious zone is provided in the embankment and 2.5:1 where the embankment is impervious.

4) The upstream of an earthfill dam should be protected against wave action by a cover of riprap or concrete. When available, a 1 meter layer of dumped rocks is usually most economical.

5) Low rock-fill dams may have upstream face slopes of 1 vertical on ½ horizontal; usually have face slopes of 1 on 1.3, the natural angle of repose of rockfill. Downstream slopes of all rockfill dams should be about 1 on 1.3.

6) For concrete dams the customary method is to assume a section with the upstream slope vertical and the downstream face approximately 0.70 horizontal to 1.0 vertical.

5.3.3 Height

1) Normally, the height of a dam is determined by the depth of water in the reservoir based on the design flood level plus an allowance for freeboard.

2) The required height of an earthfill dam is the distance from the foundation to the water surface in the reservoir when the spillway is discharging at design capacity, plus a freeboard allowance for wind tide and wave action.

3) In fine-grained soils consolidation is less rapid, and it may be necessary to provide additional height of fill so that, after settlement, the earth embankment will be at the desired height. The usual consolidation allowance is between 2 and 5 per cent of the total height of the dam.

5.3.4 Top width

1) The top width of an earthfill dam should be sufficient to keep the phreatic line, or upper surface of seepage, within the dam when the reservoir is full.

2) Top width should also be sufficient to withstand earthquake shock and wave action.

3) Top widths of low dams may be governed by secondary requirements such as minimum roadway widths.

4) A minimum width of 3 meters is usually required for maintenance.

5.4 Other Design Consideration

5.4.1 Easement Requirement

Easement requirement along the shores of a man-made lake or reservoir shall conform to Articles 50 and 51 of the Water Code of the Philippines.

5.4.2 Design Procedure

Initially, in the design of dams, the type of dam shall be determined based on the topography of the area, the kind of foundation and the available materials at or in the vicinity of the project site.

A trial section of the dam is then established considering the design criteria discussed in 5.3 as regards the height of the dam, the crest width, the slopes of the upstream and downstream faces, etc.

Determine the location of the resultant force, i.e., the point where it acts at the base, to check whether it lies within the middle third of the base of the dam for stability and for sound structural design, in the case of concrete dams.

Considering a unit length of the dam, compute for all forces acting on the dam, such as the weight of the dam, hydrostatic pressure, uplift pressure, earth pressure, including forces due to earthquakes, wind pressure and wave actions, as may be required.

Check the stability of the trial section against overturning by computing the corresponding factor of safety and find out if they are within acceptable limit.

For concrete gravity dams, compute the maximum compressive and shearing stresses and check if they are within the maximum allowable values for concrete.

For earth dams, draw the line of seepage or phreatic line on the dam body to check against piping of the embankment material as well as seepage thru the same and underneath the embankment. Slope stability analysis for the embankment slopes should also be undertaken using the Swedish slip circle method or any other acceptable method. Check also if berms will be needed for erosion and greater stability of the embankment slopes.

Check for possible settlement of the dam by analyzing the bearing capacity of the foundation and comparing it with the bearing stress on the same as caused by the dam. In no case should the bearing capacity be less than the bearing stress in order to avoid settlement of the dam.

If the design requirements are not completely satisfied, the trial section will have to be modified and the computations repeated until all design requirements are fully complied with.

Chapter 6 SLUICEWAY AND CONDUIT

6.1 Basic Concept

Sluiceway is a structure that connects the culvert passing through the dikes and its gate. Sluiceway is categorized into two (2) types according to its purpose: one is to drain the inland water into river, and the other is to draw the water (as an intake structure) from the river for irrigation use or some other purposes.

Sluiceways shall be carefully planned and so designed to conform to the master plan and other relevant plans to meet with the functional and safety requirements for the dikes/levees.

1) Sluiceway for drainage**:**

When the drainage area is too big, the drainage way might be considered as a tributary. Generally in this case, the profile of the confluence should be an open-type river channel. When the drainage area is small and the height of dike is high, sluiceway (culvert) is planned. Sluiceway is not planned in non-diked rivers.

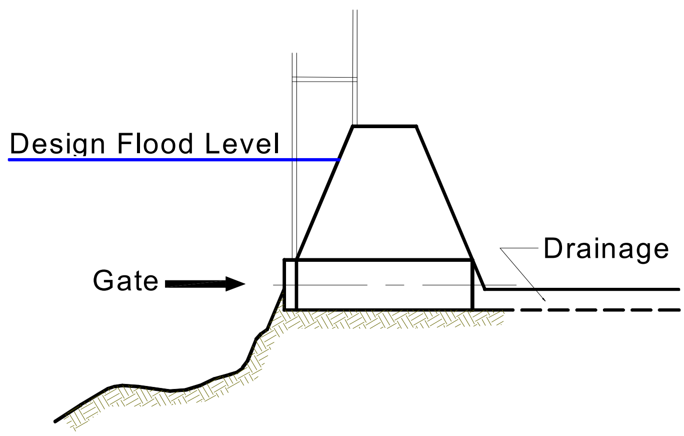


Figure 6.1 Sluiceway for Drainage

The gate of sluiceway is usually opened even during rainy days to drain the inland water. When the water level of river rises and is about to flow out through the sluiceway, the gate should be closed. Thus, this facility always requires a person to operate the gate.

2) Sluiceway for water intake**:**

During water intake, the gate is opened. On the other hand, the gate should be closed when it is not necessary to take water. However, when the water level of the river rises due to flood, then the gate should be closed. Moreover, this facility also requires a person to operate the gate always.

6.2 Location

The location of a sluiceway shall be selected according to its intended purpose. However, the sluiceway is not recommended in cases where sections of the river in which the dike is constructed has unstable river regime. Furthermore, the number of construction points shall be limited as much as possible so as to promote integration with the dike structure.

A sluiceway structure tends to make the dike weak. Considering the operation and maintenance cost, the number of sluiceway should be limited as much as possible for its full integration.

6.3 Design Criteria

6.3.1 Direction

The direction of a sluiceway shall be at right angles to the dike alignment in principle.

Since the construction of a sluice gate poses a weak point in the dike, its direction is specific to avoid complication of the structure and to ensure the intended function. However, if an oblique arrangement is inevitable due to confluence; sufficient measures should be taken for the safety of the structure and of execution of work.

6.3.2 Opening Level

The opening level of a sluiceway for irrigation shall be according to its particular intake taking into account the bed variations in the future. For drainage, the opening level shall consider the height of the riverbed or the foundation height of the channel to be connected.

There are cases where water intake for irrigation becomes difficult due to bed drop. For the construction of a sluiceway, it is necessary to examine and analyze the trend of bed variation in the past and the possibility of bed drops in the future. However, if the opening level is too low, the volume of intake might be more than the water demand; and therefore, the volume of intake must be adjusted.

As for the drainage sluiceway, if the opening height is too low, sedimentation is induced; thus decreasing the effective sectional area. On the other hand, if the foundation height is too high, the drainage capacity decreases, requiring high cost for the maintenance of the outfall. The relationship with the river bed height or opening height (level) of the channel to be connected with a conduit must be analyzed and evaluated to decide the opening level of the sluiceway.

6.3.3 Decision of Sectional Profile

For irrigation sluiceway, the sectional profile shall be large enough to secure the design intake volume even during dry season.

For drainage sluiceway, the sectional profile shall be determined by analyzing inner water based on the rainfall depth within the drainage basin covered by the sluiceway and the sudden rise or the overflow of water of the main river and the inner water. It shall be carefully arranged so that the velocity in the drainage sluiceway does not considerably change in comparison with the velocity of a tributary to be connected.

The minimum diameter of the section of a sluiceway shall be 60 cm. However, if the possible intake volume becomes excessive due to too low opening level or employment of a minimum section of 60 cm, measures shall be taken into account in the channel to be connected to the sluiceway.

Chapter 7 WEIR

7.1 Basic Concept

1) Principle of Structure

   Weir is a structure which safeguards the action of water flow at a level *equal* to or lower than the design high-water level (or the design high-tide level in case of the high-tide section). The flow of flood at a water level equal to or lower than the design high-water level and the adjacent river bank and the structure of river facilities should not be hindered by the weir. It shall be designed in consideration of prevention of scour in river bed which connects to the weir.

2) Relation with Flow Section

   The portion (excluding gate piers) except for movable section of a movable weir and the fixed weir must not be built within the flow section. However, this shall not apply when there is no hindrance against flood control (by reason that the weir is to built in a ravine, due to conditions of river and topography, etc. or provided that a measure deemed appropriate for flood control is in a river bed within the flow section).

7.2 Location

The weir is classified into intake weir, diversion weir, tide weir, etc., and is further classified into fixed weir, and movable weir according to the intended purpose.

The location of the weir is selected to efficiently achieve their specific purposes. However, since the construction of the weirs affects the river regime, and blocks the passage of water especially during floods, the location is selected at the point where the axis of channel is straight with little velocity variation. Moreover, the midstream should be stable enough with little riverbed variation.

It is advantageous to select location with narrow river width due to construction cost; however, it must be avoided as much as possible to assure safe flow during floods and considering that the weir will constrict the waterway in the future.

7.3 Design Criteria

7.3.1 Form and Direction

The plane form of a weir shall be linear in principle. The direction shall be at right angles to the direction of the river flow in the lower reach of the weir in consideration of the direction of river flow at the time of high water.

7.3.2 Ponding Level

The design ponding level of a weir shall be at least 50 cm lower than the height of the high water channel. Also, it shall be lower than the inland ground water level but not applicable when proper measures, such as embankment, are taken.

River dikes are generally not designed as structures to support normal ponding resulting to inefficient drainage in the inland or rise of ground water level. This should be considered in site selection of weir.

7.3.3 Span Length

The span length of a movable weir shall be long enough not to hinder the flow of water at the time of flood, but shall be more than the length given in Table 7.3.3.

Table 7.3.3 Design Flood Discharge and Span Length

Design flood discharge (m^3/sec)	Span length (m)
Less than 500	15
500 and up to 2,000	20
2,000 and up to 4,000	30
4,000 and over	40

If the span length of a movable weir is not within the given parameters shown in Table 7.3.3, it can be specified as follows:

a. When the overall length of the movable section of a weir is less than 30 m, and the design flood discharge is less than 500 m^3/s/, the span length of the movable section could be 12.5 m or more.

b. Based on Table 7.3.3, when the span length of a movable weir becomes 50 m or more due to span allocation, the span length could be more than the value gained from the calculation of total weir length divided by the span number plus 1. In this case, the span lengths of the movable sections must be equal. However, if the average value of span lengths of movable sections is 30 m or more, the span length of the movable section relating to the portion other than the midstream shall be more than 30 m.

c. In spans with the function of sediment discharge, if the design flood discharge is 2,000 m^3/s or more, the span length could be more than a half (that is 15 m if the span is less than 15 m) of the value specified in Table 7.3.3. And if the design flood discharge is 2,000 m^3/s or less, the value shall be reduced to 12.5 m; provided that the average of span lengths shall not be less than the value specified in the said table.

d. If the design flood discharge is 4000 m^3/s or more, the span length in the other portion except the midstream portion shall be 30 m or more; provided that the average of span lengths of the total weir length shall not be less than 40 m.

Since the columns of a movable weir may hinder the safe passage of a flood, it is desirable to adopt long span length as much as possible.

7.3.4 Height of Gate in Movable Section of Movable Weir

The height of the lower fringe of a lift gate in the movable weir at the maximum lift height (based on the design high-water discharge) must be larger than the obtained by adding value in Table 1.3.3 as shown in Section 1.3.3, to the design high-water level. It must not be lower than the design high-tide level in a high-tide section. In other sections, it must not be lower than the height of a line connecting the tops of the riverside slopes of the levees on both sides of the river at the target spot (in case the design cross-section features are fixed, in case the heights of the design levees are lower than those of the existing levees, and it is deemed that there is no hindrance against flood control or in case the heights of the design levees are higher than those of the existing levees).

7.3.5 Administration Facilities

A movable weir shall be equipped at need with inspection bridge and other facilities suitable for administration.

Chapter 8 BRIDGE

8.1 Basic Concept

Abutment and pier to be built within the river area shall be a structure which will be safe against the action of river flow at a water level equal to or lower than the design water level (or the design high tide level in a high tide section).

Abutment and pier shall not disturb the flood flow at a water level equal to or lower than the design high water level. They shall not severely hinder the structure of adjacent river banks and facilities. And they shall be designed in consideration of prevention of scour in river bed and high water channel adjoining the abutment or the pier.

8.2 Design Criteria

8.2.1 Abutment

Abutment to be built in river bank or on a river with width larger than 50 m or in a levee under back-water section or high-tide section (or the design levee in case the design cross-section is fixed) must not be built within the flow section. However, this shall not apply if the abutment is in a ravine or due to river conditions, the topography, etc. and there is no hindrance against flood control.

Abutment planned to be built in a levee (except those provided in the preceding paragraph) shall not be built on the river side front slope of the levee. The river-side face of an abutment to be built in a levee must be parallel to the slope alignment of the levee. However, this shall not apply in case the necessary measure is taken that will not hinder severely the structure of the levee.

The bottom of an abutment to be built in a levee shall be fixed on the foundation of the levee.

8.2.2 Bridge Pier

The horizontal cross section of a pier to be built in a river channel shall be as thin as possible in elliptical or other similar shape. The direction of the major diameter must be the same as the direction of flood flow. However, the horizontal cross section of pier may be a circular or other similar shape. The latter applies when the horizontal cross section of pier is very small, in case the component of load acting on the pier perpendicular to the direction of the flood flow is very large and it is deemed to be unavoidable from the structural standpoint, or in case the pier is built at a site where the direction of flood flow is unstable.

The foundation part of the bridge pier to be built in a river channel shall be deeper than 2m below the surface of the low water bed in the low-water channel (including the low-water channel of the design cross-section when it has been fixed) and the high-water channel within 20m from the top of slope of river bank of the low-water channel. The foundation shall be deeper than 1m below the surface of the high water bed (including the high water bed related to the design cross-section when it has been fixed) in other part of the high water channel. However, in case change in river bed is very small or in case it is unavoidable due to river condition or other special circumstances, it is permissible to settle the foundation part of the pier at a depth below the surface of the low water bed or the surface of the high water bed.

8.2.3 Span Length

In case bridge piers are built in a river channel, the standard span length between the centerlines of two adjoining piers shall have larger value than the value obtained by the formula shown below. All the distances between piers should be the same. In case span length exceeds 50m, the distance shall be 50 m, when there is no hindrance against flood control such as the site is in ravine or there is constraint in river conditions and topography. However, if the span length is larger than the value obtained by the formula, the average value will exceed the obtained value by adding 5 m to the standard span length. The span length may be larger than the obtained value by deducting 5 m from the standard span length (or span length shall be 30 m in case this will become less than 30m).

$L=20+0.005Q$

Where :

L: span length (m),
Q: design-high-water discharge (m^3/s)

Example

Q=3000m3/s, B=100m, L=20+0.005*3000=35m
2 span: L=50 m > 35m +5m,
3 span: L=33m >= 35m - 5m

8.3 Other Design Consideration

8.3.1 Overhead Clearance

Overhead clearance of the bridge, per design high-water discharge, must be a value by adding freeboard to the design high-water level.

The height of the bridge surface, in a back-water section or a high-tide section, shall be higher than the height of the levee to be crossed by the bridge

8.3.2 Revetment

In building a bridge, revetment for preventing scour at river bank or levee which may occur due to change of the flow shall be built. An appropriate bed-protection work shall be executed.

In addition, if it is necessary to protect the river banks or the levees under the bridge, they shall be protected with concrete or other material similar thereto.

8.3.3 Inspection passage

A bridge (including connection part) shall be a structure which will not hinder the structure of inspection passage.

8.3.4 Exception of Application

The provisions above shall not apply to bridge proposed in a lake or swamp, a detention reservoir or other similar areas. Ideally, a bridge is to be planned in an environment which can minimally influence flood control.