CHAPTER 8

COUNTERMEASURES AGAINST RIVER EROSION

8.1 General

Riverbank and/or riverbed erosions and subsequent riverbank collapse due to river flow are referred to as river erosions. This phenomenon initiates scouring of the road foundation and enlarges or widens into a riverbank collapse. Figure 8.1 gives a conceptual diagram of the effect of river erosion on roads.

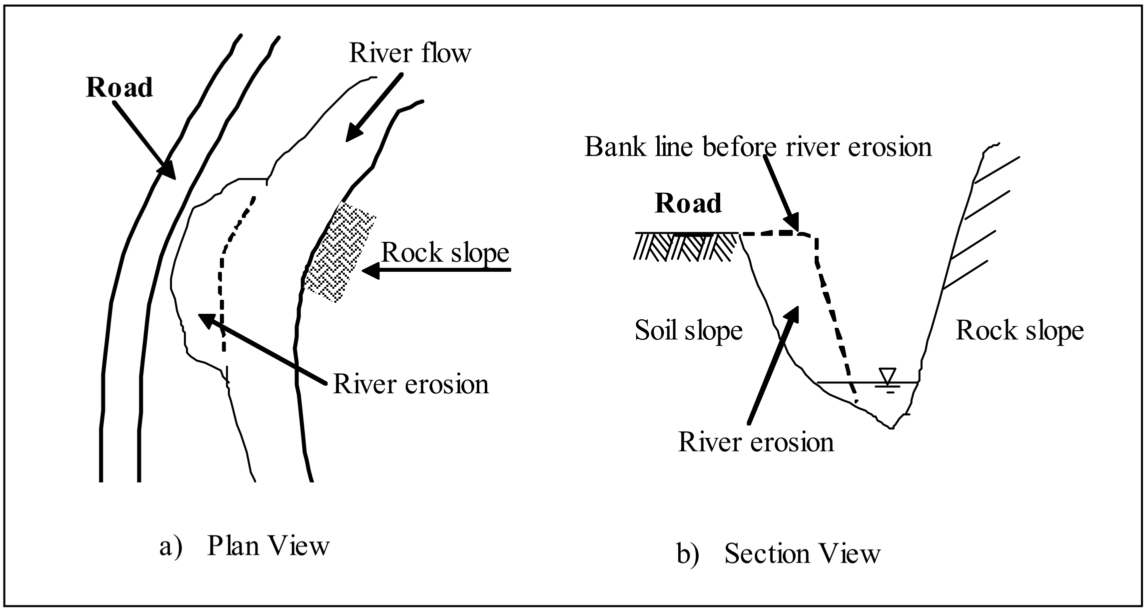


Figure 8.1 Conceptual Diagram of the Effect of River Erosion

River erosions are not common along the national highways; however, their occurrence, particularly those close to the national highways, results in the collapse of the roadway, thereby leading to decreased road capacity and increased potential for road accidents.

Therefore, proper countermeasures for river erosions must be designed by using structures and/or other methods. This chapter gives the basic policy for the selection of countermeasures against river erosions and design considerations for the main countermeasures.

Reference is made to Chapter 3 for design of retaining walls. Chapter 7 - Countermeasures against Debris Flows - contains useful information on planning and design of sabo works. Some countermeasures for road slips, as given in Chapter 6, are also applicable to river erosions.

Moreover, reference is also made to Technical Standards and Guidelines for Planning and Design, Volume III, SABO (EROSION AND SEDIMENT CONTROL) WORKS, and Volume IV, NATURAL SLOPE FAILURE COUNTERMEASURES, *Project for the Enhancement of Capability in Flood Control and Sabo Engineering of DPWH,* March 2002.

8.2 Selection of Countermeasures

8.2.1 Classification of Countermeasures

Table 8.1 shows the general classification of countermeasures against river erosion. The principal countermeasures are classified according to the functions of the structures and the main causes of river erosions.

Table 8.1 Classification of Countermeasures against River Erosion

CLASSIFICATION		TYPE OF WORK
1. BANK EROSION PREVENTION WORK	Revetment Work	Stone pitching
		Block pitching
		Concrete pitching
		Wire cylinders
		Gabion walls
		Stone masonry walls (dry and wet)
		Concrete block walls
		Concrete retaining walls
		Sheet piles (steel and concrete)
2. RIVERBED EROSION PREVENTION WORK	Consolidation Work	Stone consolidation
		Concrete consolidation
		Gabions (or concrete) foot protection
		Stone (or concrete dams)
		Spur dikes
	Check Dam Work	Check dams (concrete, steel pipe, etc.)
		Groundsill (head and non-head types)
3. OTHER WORK	Relocation of Channel	Short cut, Rechannel

(1) Bank erosion prevention works

Revetments represent a typical example of this type of structure, and include concrete and dry masonry revetments. These are applied to protect the related riverbank from erosion and infiltration by river current, preventing the collapse of the riverbank, as shown in Figure 8.2.

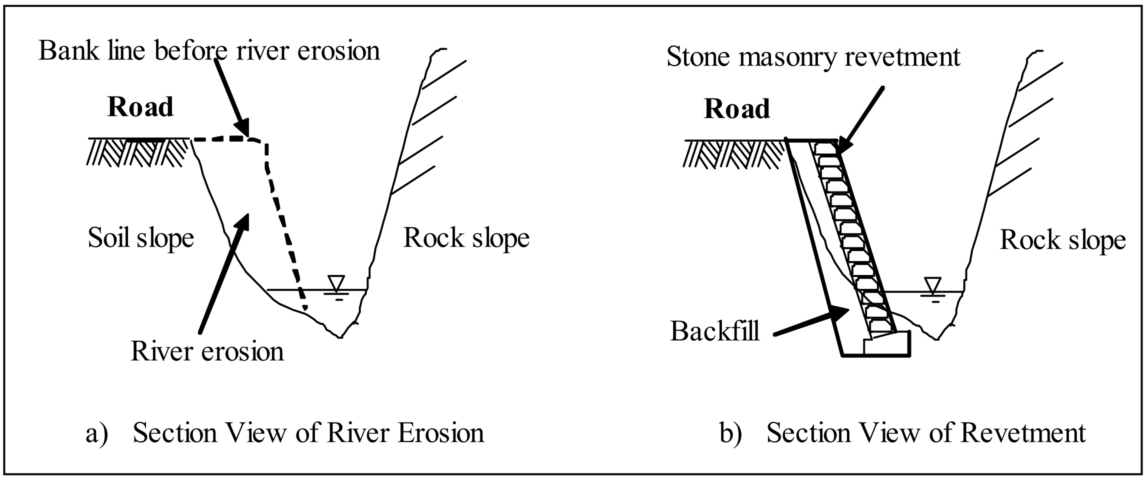


Figure 8.2 Example of Revetments for River Erosion

(2) Riverbed erosion prevention works

Check dams (stepped dams and consolidation dams) and groundsills (head type and non-head type) are typical examples of riverbed erosion prevention works. These structures are used to stabilize the riverbed by preventing riverbed erosion, the movement of riverbed sediments and thus preventing the destruction of riverbanks, as shown in Figure 8.3.

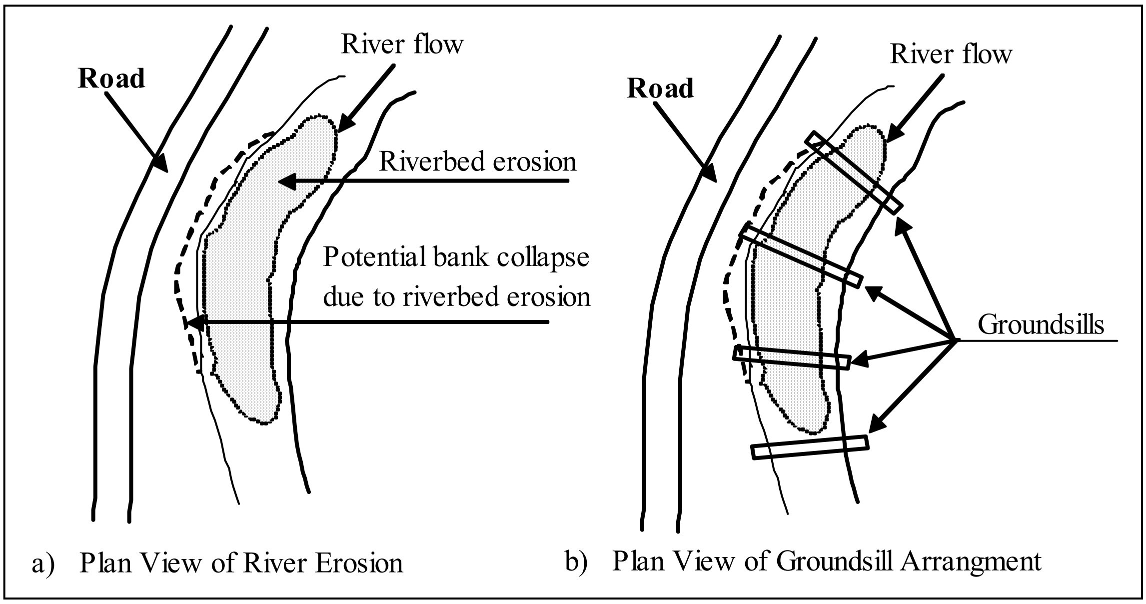


Figure 8.3 Typical Examples of River Erosion Stabilized with Groundsills

(3) Other works

Figure 8.4 gives a conceptual image of re-channelling to mitigate the effect of river erosion on roads.

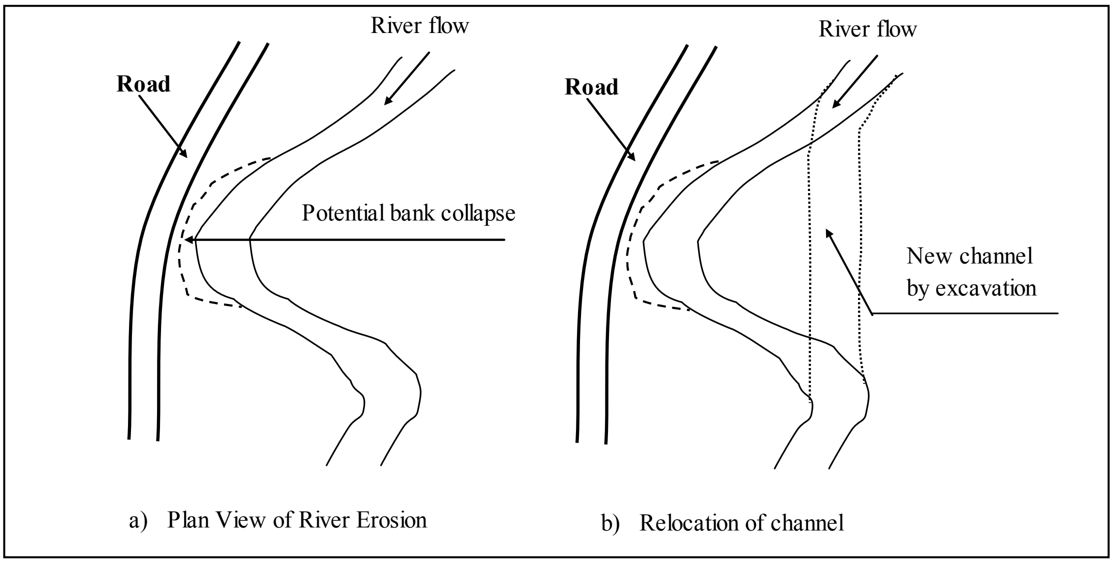


Figure 8.4 Example of Channel Relocation

Relocation of the channel can be the appropriate option, considering river erosion together with channel capacity, records of past disaster, social and economic importance. For the relocation of the channel, excavation is adopted as much as possible and construction of embankments should be avoided since these are prone to breaking or breaching.

8.2.2 Criteria for Selection of Countermeasures

Figure 8.5 gives a flowchart for the selection of countermeasures for river erosion. In planning the countermeasures on a river prone to river erosion, various types of countermeasures should reasonably be combined, considering the causes of river erosion, proximity to the objects (highways, houses, etc.) prone to be damaged due to river erosion, and the river situation (flow velocity, slope of riverbed, river water table, soil properties of the riverbank, movement of sediment on the riverbed, etc.).

For the planning and design of countermeasures against river erosion, the following steps must be taken:

a) Preferably, when river erosions have the potential to cause considerable damage to the road capacity (the distance from the edge of the highway to the river bank is less than 5.0 meters), a comprehensive and permanent combination of countermeasures for river erosion should be provided and designed. On the other hand, when there is an immediate concern that the related section of national highway may not be adequately safe against river erosion, preventive repair works or temporary works should be provided to control further erosion.

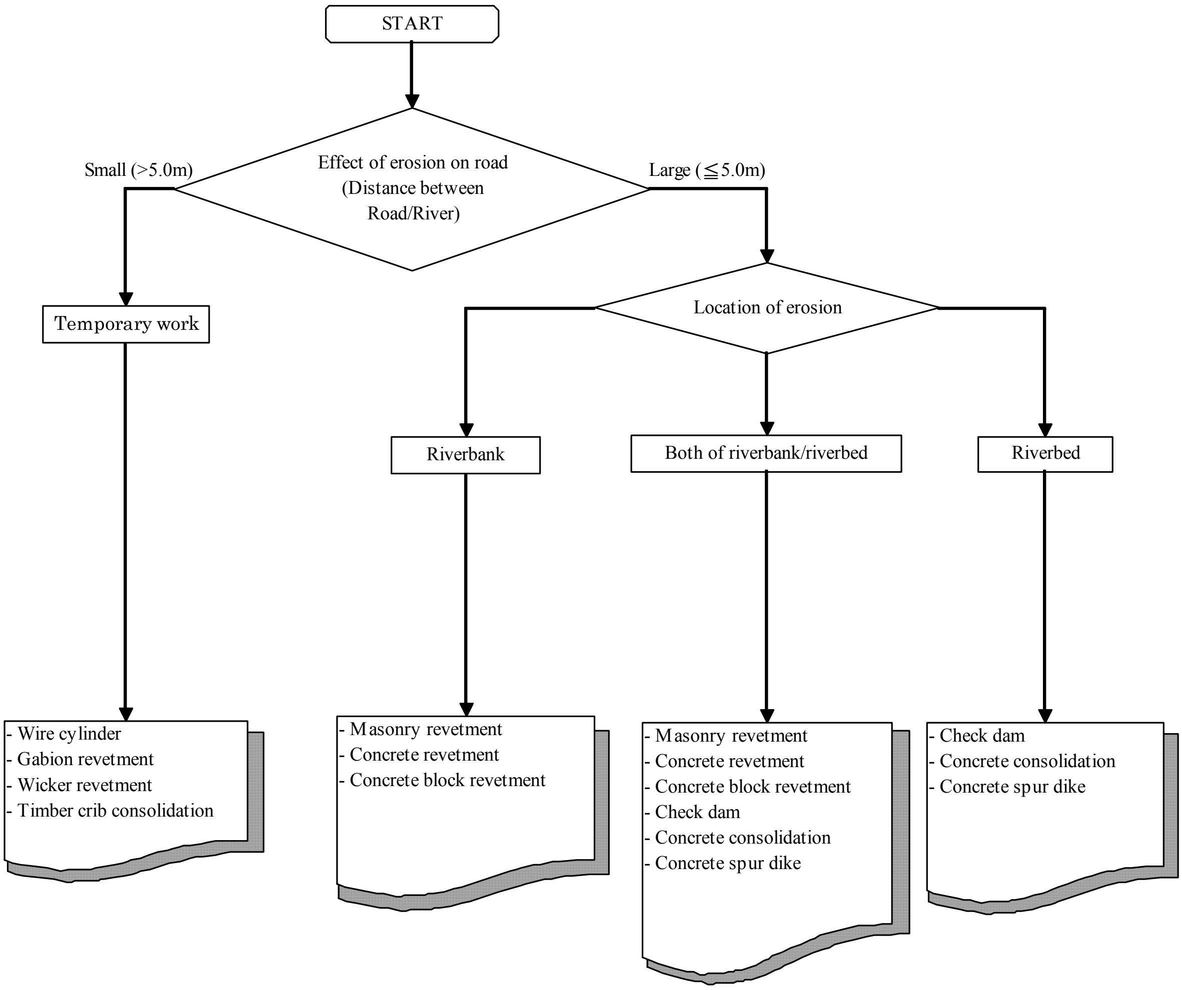


Figure 8.5 Selection Flowchart for Countermeasures against River Erosion

b) In planning the countermeasures, the place of construction, and the extension (length) and type of works, etc. are considered together with the characteristics of river erosion (causes, size, frequency, etc.) as well as the situation of the related river course (river water table, flow velocity, flood discharge, bank geology, etc.).

c) When the countermeasures against river erosion have been determined, consideration is given to the provision of protection for the foot of the countermeasure structure where necessary. Most structures relevant to river erosion, especially revetments, are highly susceptible to foot scouring. Foot scouring will affect the stability of the structures, and consequently, the stability of the riverbank. Therefore, foot protection or treatment for these structures should be designed in a conservative manner.

8.3 Design of Main Countermeasures

8.3.1 Revetments

The most common method relevant to river erosion is revetments. Revetments can be divided into several types in terms of construction method, as shown earlier in Table 8.1. These can be constructed of masonry, concrete, reinforced concrete, steel piles, and others, as shown in Figure 8.6.

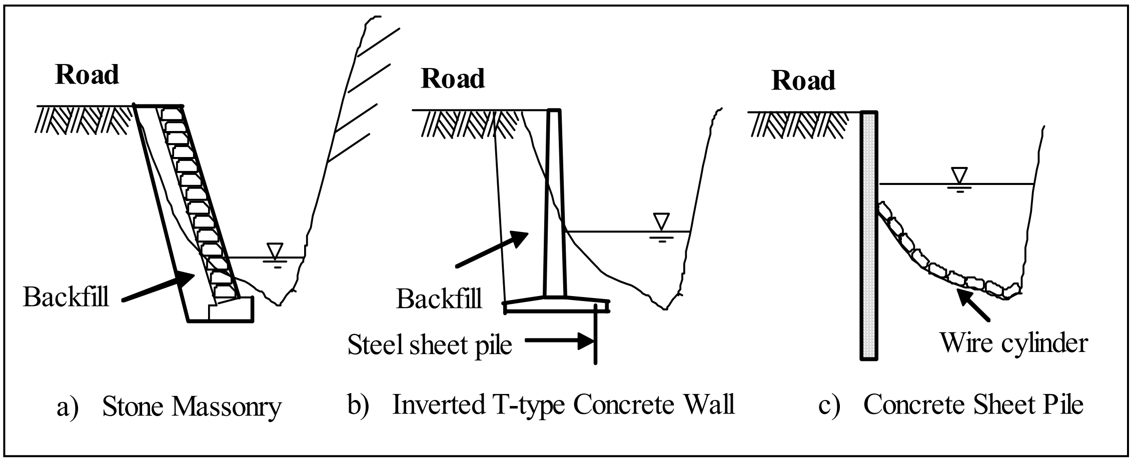


Figure 8.6 Diagrammatical Arrangements of Revetments

(1) Purpose

A revetment is constructed to protect levees and natural banks against erosion and infiltration by water or scouring by the current to ensure and improve the stability of a levee or natural bank slope.

(2) Design considerations

In planning and designing a revetment, the location, length and type of revetment is determined considering the causes of river erosion, river regime, longitudinal and cross-sectional shapes, gradients of riverbed and bank, bank geology, and its proximity to other structures. Figure 8.7 gives the general design procedures for revetments.

The length or extension of a revetment shall be determined in consideration of the change of hydraulic phenomena in the river channel. Revetment alignment should be as smooth as possible with the flow direction of the water course.

The crest height of the revetment should be equal to the sum of the design high water level and freeboard. The revetment on the outer bank of a stream outside bend should be stronger and its crest higher. The top of a revetment upstream of a Sabo dam and groundsill should be equal to or above the wing crest height of the Sabo dam and the groundsill. When flow velocity increases at the outer bend of a stream, the difference in the water level between both banks rises. The water level at the outer bank rises higher than that of the inner bank. For this reason, the revetment on the outer bank should be stronger and its crest higher.

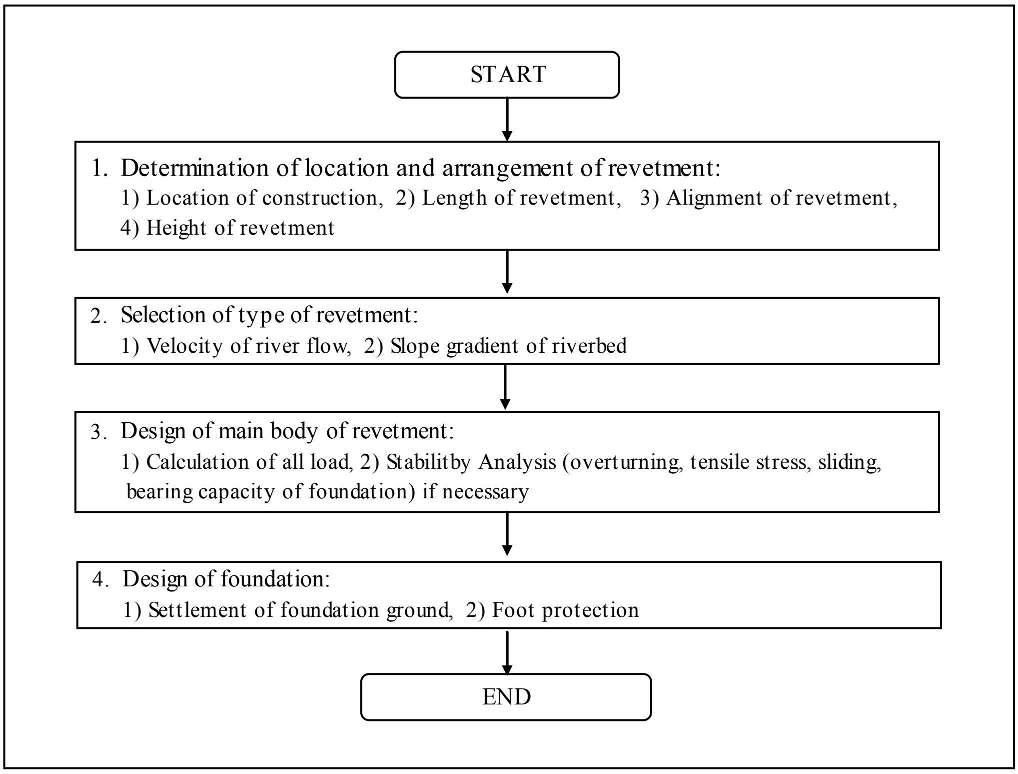


Figure 8.7 Design Procedure for Revetments

The freeboard of a revetment is based on the maximum experienced water level due to swells and afflux during floods. A revetment on a steep stream (slope of riverbed exceeding 1/100) with a bedload of boulders and driftwood should have sufficient crest height (design flood level plus freeboard).

In selecting the type of revetments, consideration should be given to the roughness of the riverbed sediment, the velocity of river flow and the slope gradient of the riverbed at the planned reach of the river. Table 8.2 gives the relationship between height, gradient and type of revetment.

Concrete revetments such as stone masonry, concrete block masonry, concrete retaining, and reinforced concrete revetments are planned for protection against river erosion. Dry masonry revetments are usually inadequate for such a function.

Table 8.2 Relationships between Height, Gradient and Type of Revetments

<table>
<tr><th>Type</th><th>Pavement</th><th>Height (m)</th><th>Gradient (V:H)</th></tr>
<tr><td rowspan="4">Stone masonry, Concrete block masonry</td><td rowspan="3">Wet</td><td>Less than 5</td><td>1:0.5 (standard)</td></tr>
<tr><td>3 or more</td><td>1:0.4 to 1:0.6</td></tr>
<tr><td>Less than 3</td><td>1:0.3 or more</td></tr>
<tr><td>Dry</td><td>Less than 3</td><td>1:0.5 to 1:1.5</td></tr>
<tr><td rowspan="2">Stone pitching, Concrete block pitching</td><td>Wet</td><td>Same as the height of the bank</td><td>1:1.0 to 1:2.0</td></tr>
<tr><td>Dry</td><td>Less than 3</td><td>1:1.0 to 1:3.0</td></tr>
<tr><td>Wire cylinder</td><td rowspan="2"></td><td rowspan="2"></td><td>1:1.5 to 1:2.0</td></tr>
<tr><td>Gabion wall</td><td>1:0.5 to 1:1.0</td></tr>
</table>

The foundation of a structure in a river is easily scoured by high velocity flow. Because river flow often contains sediment and boulders, revetment works are subject to strong impact forces. Simple structures, like dry stone masonry, are easily destroyed. To prevent such damage, concrete rubble, grouted riprap, concrete stone masonry, reinforced concrete or concrete block revetments should be used. Concrete supports should be used for concrete revetments. These structures should be provided with concrete backfill for greater stability. Dry masonry is not to be used unless there is no danger of destruction.

The main body of a revetment is designed in the same manner as that for retaining walls. The slope of the revetment is designed based on the original slope of the riverbank, and in principle is 1:0.5.

The embedment of the revetment should be deep enough to be safe against the scouring of the riverbed during high water levels. The depth of the embedment should be 0.5 to 1.0 m in medium to small rivers and more than 1.0 m in large rivers.

Because the lower part of a revetment is easily damaged, foot protection for revetments is generally provided to ensure its stability by decreasing the flow force at that point and preventing scouring of the revetment foundation. The top of the foot protection should be higher than the design riverbed height or equal to the existing riverbed height when the existing riverbed height is lower than the design riverbed height.

Further, 50 mm diameter weep holes for the drainage of the revetment should be provided at intervals of 2~3 m and staggered.

Figure 8.8 gives the standard sections of stone masonry and concrete block masonry revetments.

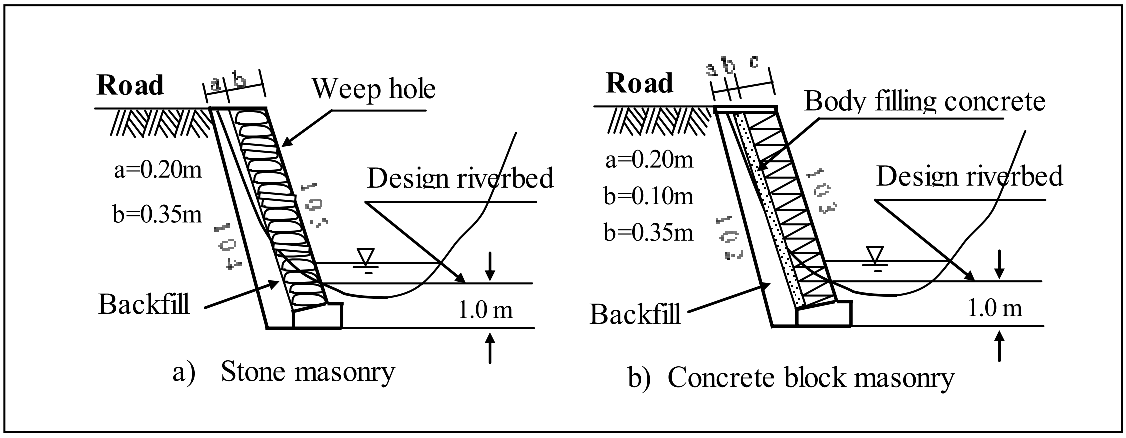


Figure 8.8 Standard Sections of Stone and Concrete Block Masonry Revetments

8.3.2 Groundsills

Groundsills are one of the main countermeasures used as protection against riverbed erosion. They are installed to protect against riverbed erosion due to water flow, thus preventing the collapse of the riverbank. There are two types of groundsills; the head type and the non-head type. A groundsill of the head type stabilizes the riverbed by preventing the riverbed erosion, the movement of riverbed sediments and thus prevents the destruction or collapse of the riverbank. It also protects the foundation of structures like revetments. A groundsill of the non-head type is a supplementary structure to protect the existing head type groundsill if scouring and erosion still progress and riverbed degradation occurs at the transition point of the slopes. Figure 8.9 gives diagrams of groundsills.

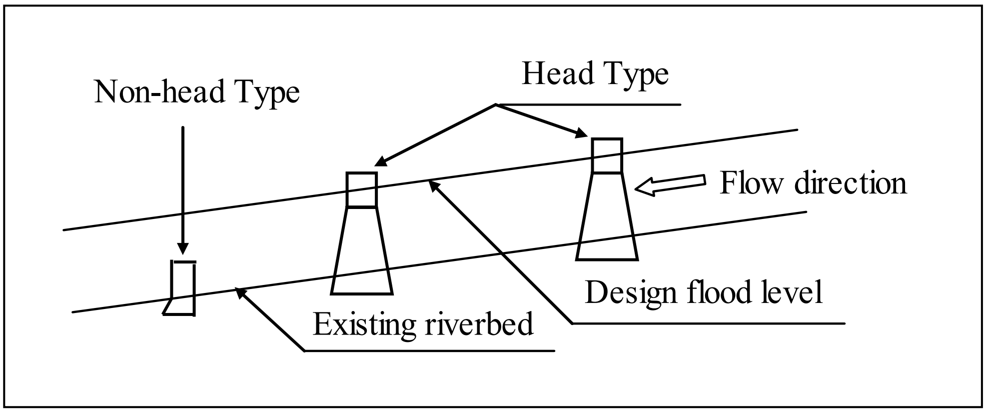


Figure 8.9 Diagrams of Groundsills

(1) Purpose

A groundsill is applied to stabilize the riverbed for the following purposes: 1) to decrease the scouring force of water flow for the stabilization of the riverbed in the upper reach (with head type), 2) To prevent the scouring and lowering of the riverbed (with non-head type), and 3) To ensure the stability of revetment foundations (for both).

(2) Design considerations

Figure 8.10 gives the general design procedure for head type groundsills. According to the above-mentioned purposes, groundsills shall be planned for the following locations: 1) Sections with riverbed erosion or degradation, 2) Sections immediately downstream and on the same side as a riverbank collapse, or 3) The area downstream of the structures to be protected. In general, where the collapsed structures and the scoured areas are large, several groundsills are constructed in a stepped form.

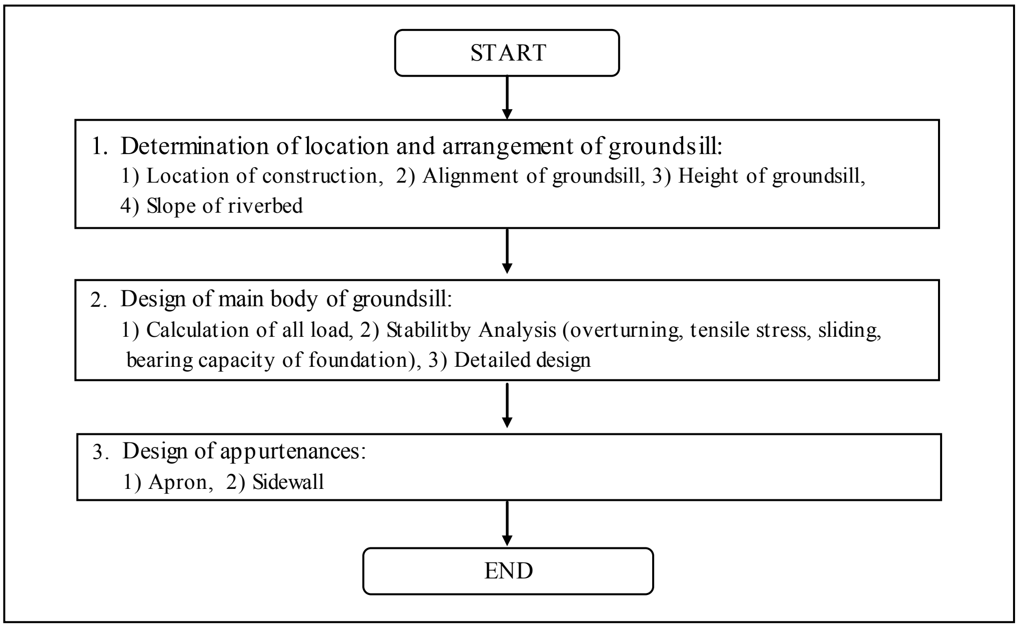


Figure 8.10 Design Procedures for Groundsills (Head Type)

The plane form of a groundsill should be linear and its direction should be at a right angle to the direction of the water flow, as shown in Figure 8.11.

The standard height of a groundsill should be less than or equal to 5 m. When an apron and vertical wall are constructed, the crown height should be 3.5 to 4.5 m from the design riverbed. Further, when the height of the groundsill (groundsill with apron and vertical wall included) exceeds 5 m or when groundsills are required due to the large area needing protection, stepped groundsills are recommended. On the other hand, the crest of non-head type groundsills should be as high as the design riverbed.

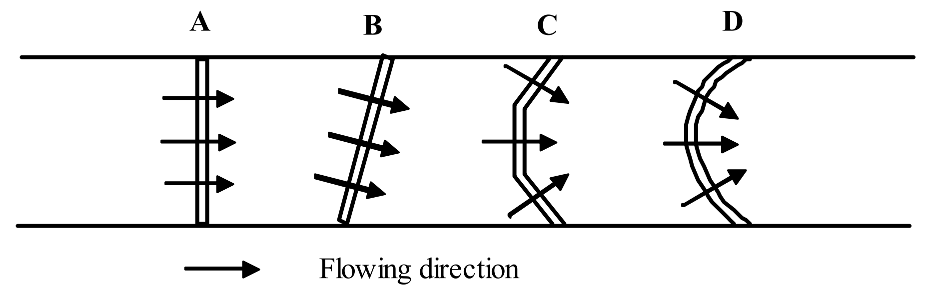
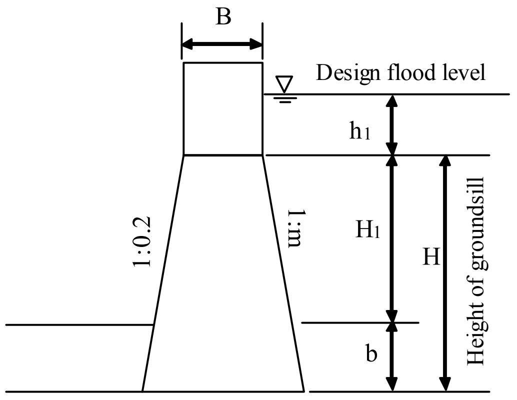


Figure 8.11 Forms of Groundsills and Flow Direction

The design slope of the riverbed is determined by considering flow velocity, water depth and resisting force of the riverbed. The slope is designed to balance erosion and accumulation of sediments. The riverbed, at the toe of the downstream slope of the groundsill, is deepened by the scouring of overflowing water. This must be considered in determining the design slope of the stepped groundsills. In the case of stepped groundsills, the foundation of the upstream groundsill should be embedded below the design riverbed slope of the downstream groundsills.

The stability analysis of a groundsill body is basically the same as that of a sabo dam as described in Chapter 7 in this Manual. The external loads include dead weight, hydrostatic pressure at design flood level and earth pressure produced from sediments behind the groundsill.

The details of groundsill bodies are illustrated in Figures 8.12 and 8.13 and summarized as follows:



H=height of groundsill body (m)
B=Thickness of crest opening (m)
H_1=Height of fall (m)
h_1=Overflow depth (m)
b=Embedment depth of main body (m)
m=Upstream slope of groundsill

Figure 8.12 Detail of Groundsill Body

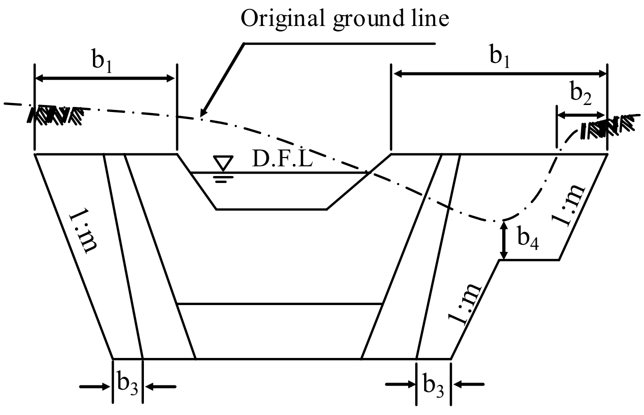


Figure 8.13 Embedment of Groundsill Wings

- The design riverbed slope should be 1/2 of the existing riverbed slope. If the riverbed is unstable, a non-head type of groundsill is provided between existing groundsills.
- The side slope of the downstream groundsill is 1:0.2, and 1:0.2 or less for the upstream groundsill.
- The thickness of the crest opening B is 1.0~1.5m.
- The embedment of the wings varies from 0.5 ~2 m (refer to Table 8.3).
- Both sides of the groundsill wing should extend sufficiently into the bank slope, as given in Table 8.3.

Table 8.3 Embedment of Groundsill Wings

Item	Sand and gravel	Soft rock	Hard rock
b	1.5 m ~ 2.0 m	0.5 m ~ 1.5 m	0.5 m
b_1	0.5 m	0.5 m	At least 1.0 m
b_2	At least 0.5 m	At least 0.5 m	0.5 m
b_3	1.0 m	At least 0.5 m	At least 0.5 m
b_4	At least 1.0 m		
m	According to the material	1:0.5	1:0.3

Furthermore, some protection works are required to protect the groundsill bodies from breaking and collapse due to scouring by flowing water in the lower reach. Such protection works include aprons, sidewalls and vertical walls.

An apron should be installed flat with a maximum slope of 1/10. The thickness of the apron shall be 0.3 m ~1.0m depending on the height of fall and geology of the river bank.

Sidewalls are best made of concrete. In some instances, concrete rubble or stone masonry may be used. The thickness of the crest shall be 0.3 m, with a front slope of 1:0.5 and back slope of 1:0.3.

CHAPTER 9 COUNTERMEASURES AGAINST COASTAL EROSION

9.1 General

Coastal slope revetments are provided to prevent road bank erosion and to preserve the lives or assets behind the seashore from the impacts caused by waves, flood tides and tsunami. Figure 9.1 is a conceptual illustration of coastal slope revetment design. The slope revetment shall be designed to perform the following functions:

- To minimize impact by seawater due to flood tide or tsunami,
- To reduce overtopping by waves, and
- To prevent erosion due to seawater reaction.

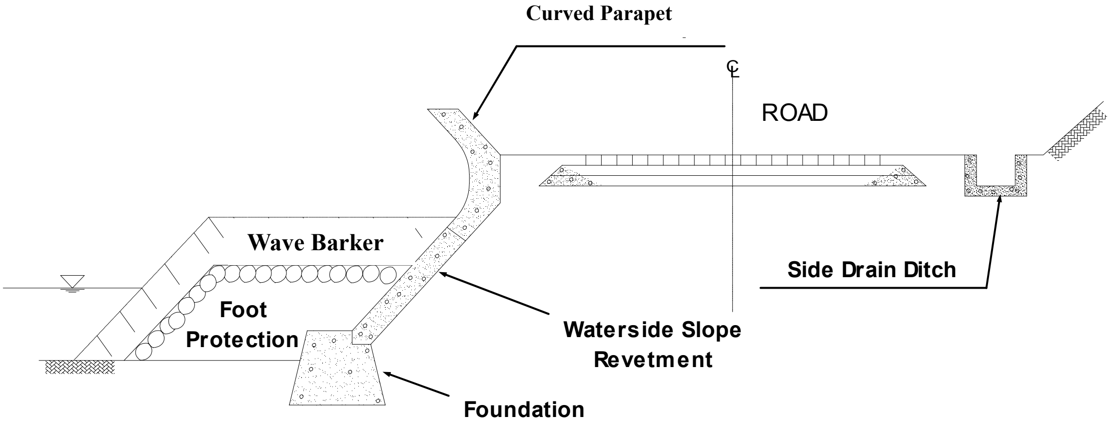


Figure 9.1 Illustration of Conceptual Design for Coastal Slope Revetment

In general, coastal revetments are composed of a slope revetment, foundation, foot protection, wave breaker and curved parapet wall, which are installed as required in consideration of the tidal level, wave force and sub-soil ground condition, which are to be accounted for in the design,. Each structural element shall be designed as demonstrated hereafter:

The Slope Revetment, which is covered with stone or concrete material, directly protects the bank slope from erosion. The structure of the revetment including backfill material, should be examined as measures against scouring and washing-out of the backfill materials, which can be caused by seepage water from inside of the bank, not only by coastal waves.

The Foundation is required to prevent settlement and sliding in order to ensure the stability of the upper structure, and is opposes scouring caused by waves. In addition, in case the revetment foundation is to stand on permeable sub-soil, a water cut-off is taken into consideration in the design of the foundation. Figure 9.2 shows a typical countermeasure for leakage from sub-soil under the foundation or from the void between the foundation and the sub-soil under the foundation using a concrete wall and sheet pile. The short infiltration route length in comparison with the water head difference between the inside and outside of the bank poses the risk of ground failure at the foundation caused by piping phenomenon or heaving phenomenon. Providing a cut-off wall as shown in Figure 9.2 makes the infiltration route longer, thus reducing the risk of piping and heaving.

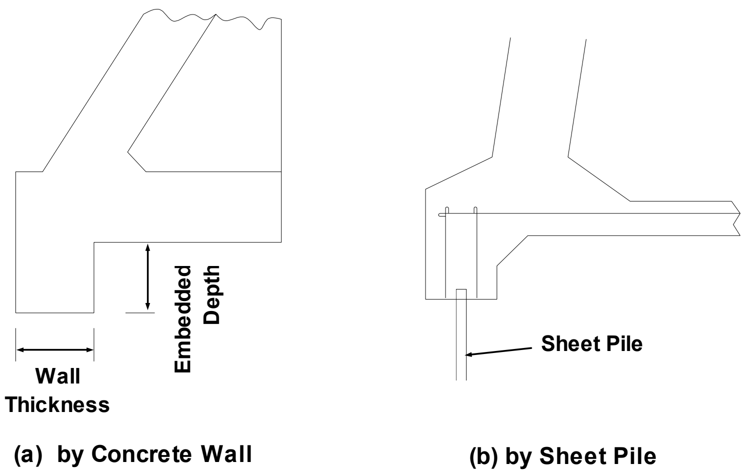


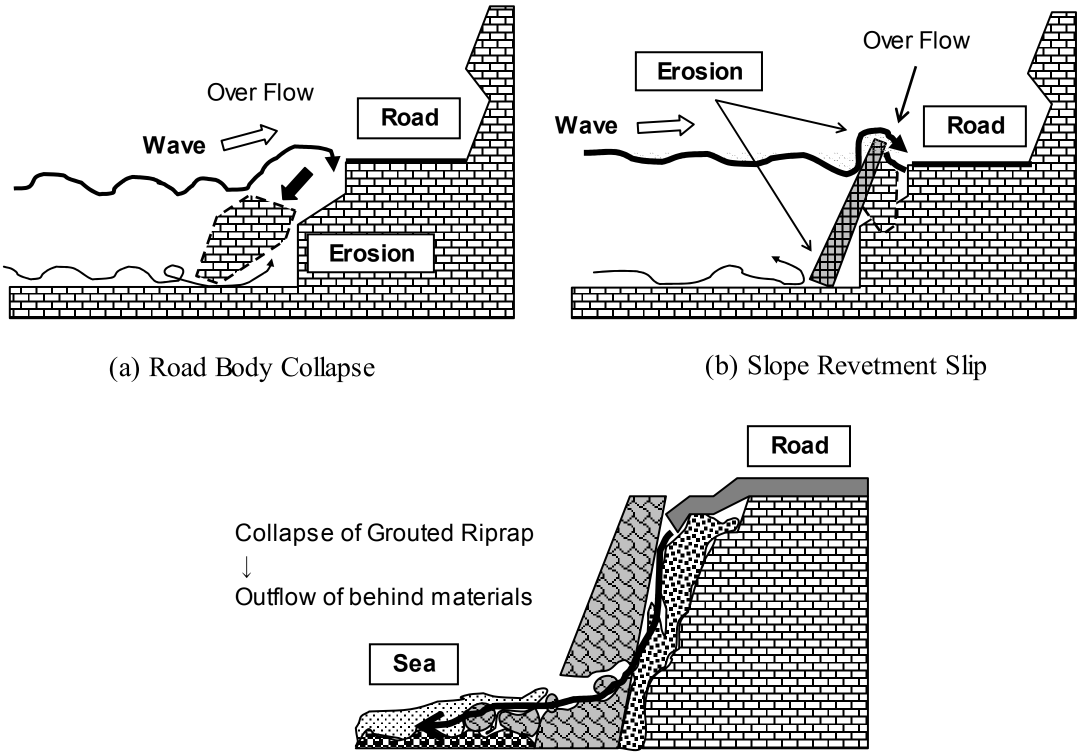
Figure 9.2 Typical Foundation Water Cut-off Wall
Source: Modified from Reference No. 12, Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

Foot Protection to be provided on the waterside slope of the revetment or in front of the foundation, aimed to prevent scouring at the toe of the revetment due to wave action, and to protect the structure of the foundation and the slope revetment. Rubble stone and concrete block, usually utilized as foot protection, shall have sufficient weight to resist the wave force, and the movement of which, such as settlement or bending, shall be separate from and will not affect the revetment body.

The Break Water Structure to be provided in front of the revetment wall is aimed to reduce the splash height of the waves, the number of overtopping waves and the power for the wave force that will act on the revetment wall. The wave breaker is made by piling cast concrete-blocks on the mound of rubble stone or by piling cast blocks on all sections.

The Curved Parapet is installed to extend the top of the slope revetment in order to decrease overtopping of the waves or spray. Since the wave force acts on the parapet wall, the parapet wall is required to be strongly connected to the revetment body. For this reason, the joint section shall be cast with a smooth curved surface without discontinuity so that the impacting wave will flow along a smooth surface.

The damage and collapse of road slopes commonly observed along the national highways running along the coastline are as shown in Figure 9.3. This figure indicates the slip part of the road on the sea side slope such as (a), breakage of a grouted riprap due to erosion of the foundation and (b) settlement of the road pavement or shoulder on the sea side slopes due to out flowing of fill material caused by the collapse of the grouted riprap (c).



(a) Road Body Collapse

(b) Slope Revetment Slip

(c) Wash out of Backfilling Material

Figure 9.3 Typical Slope Damage Caused by Coastal Erosion

9.2 Selection of Countermeasures

In principle, a countermeasure shall be chosen that is appropriate for the damage level, the importance of the road, the social condition along the damaged section, and the cause of damage. The major causes of damage are enumerated as follows;

- The strength of the revetment structure including backfill materials is not adequate to withstand the continuous wave force, or does not meet the technical requirements.
- Scouring at the slope toe by wave force or seepage water from the backside of the slope revetment.
- Washing out of the backfill material due to seepage water.

- Damage due to overflow water.

Figure 9.4 shows the general classification of countermeasures against coastal erosion. The permanent countermeasure shall be selected to examine the impact on road function and impact on economic activity, population, property and so on.

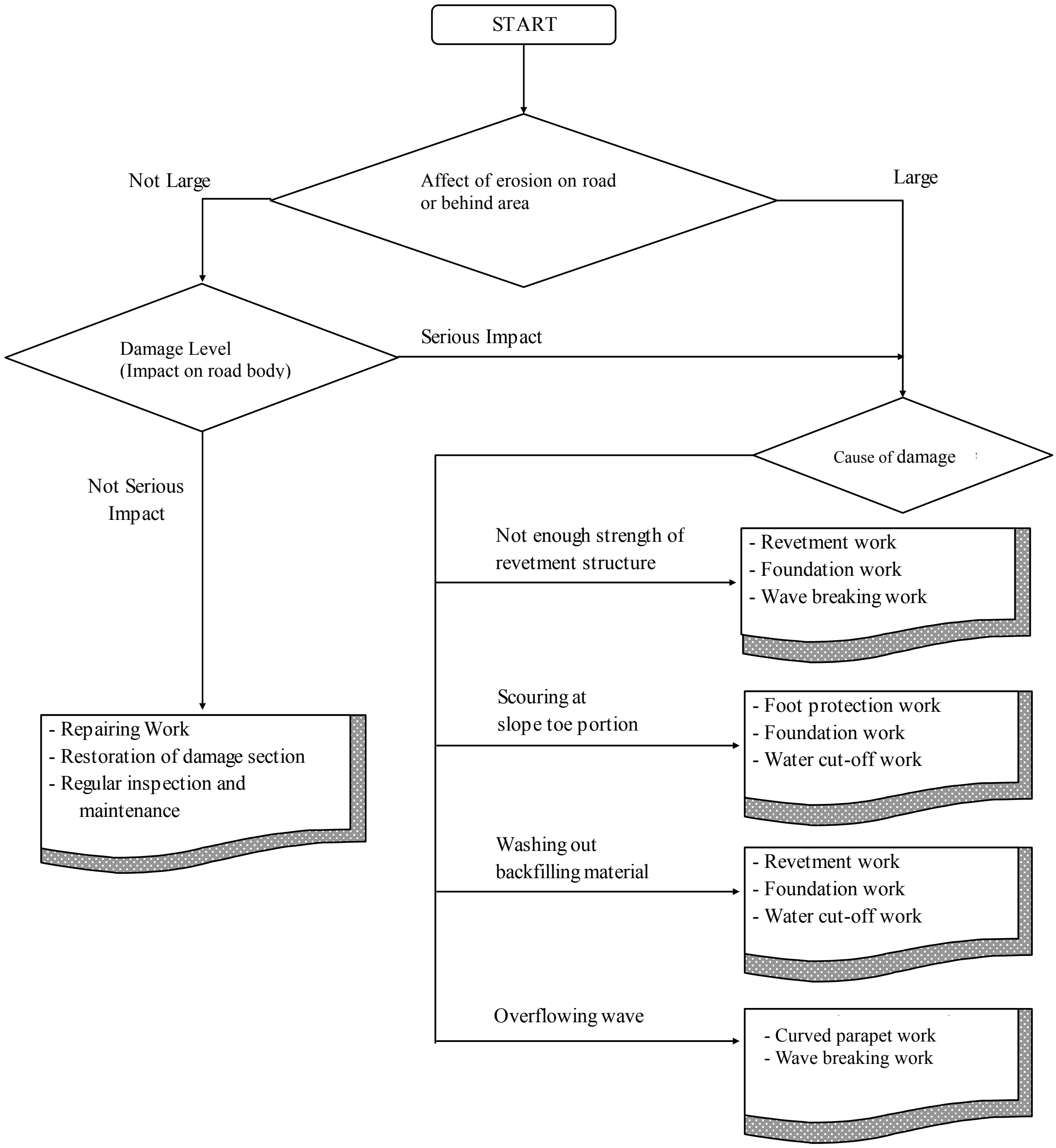


Figure 9.4 Countermeasures for Coastal Erosion as per Damage Cause

The principal countermeasures may be selected as per the following types of damage cause:

a) Not enough strength of slope revetment to withstand wave force

- Re-construct slope revetment and foundation to meet the technical requirements.
- Install wave breakers to reduce wave force that will act on the surface of the slope revetment.

b) Scouring at toe by wave force

- Ensure that the required embedment depth of the foundation is provided.
- Provide foot protection on the toe or in front of the foundation.

c) Scouring at the toe by seepage water

- Replace the backfill materials of the revetment and/or embankment material with non-permeable materials.
- Ensure the required embedment depth of the foundation is provided and/or provide water cut-off wall.

d) Washing out of backfill material

- Replace revetment with water-proof structure.
- Replace backfill material of the revetment with non-permeable materials.
- Ensure the required embedment depth of the foundation is provided and/or provide water cut-off wall

e) Overflowing waves

- Install wave breakers in front of the revetment slope.
- Provide a curved parapet wall connected to the existing revetment body.

9.3 Design of Coastal Slope Revetments

9.3.1 General

The revetment type, alignment, slope gradient, and crest elevation shall be determined taking into consideration the location, existing conditions, such as natural conditions, importance of the road and the social environment along the road, adjoining coastal preservation facilities and usage of land and seacoast. The design procedure is given in Figure 9.5, considering the following design conditions:

a) Tide level and wave conditions

Information regarding the tide level and wave properties are indispensable in slope revetment design, since the crest elevation and structure detail is determined from overtopping waves and splash height, and the stability of the bank body is analyzed considering the wave force.

b) Seashore topography

START

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1. Setting General Design Conditions:
(1) Tide level and wave height condition, (2) Seashore topography condition,
(3) Ground and borrow material condition, (4) Seismic condition,
(5) Constriction condition, (6) Social and environmental condition

↓

2. Determination of Basic Design Structural Parameter:
(1) Structure type of revetment, (2) Design crest elevation of revetment,
(3) Slope gradient of revetment

↓

3. Stability Analysis of Bank Body:
(1) Examination of settlement, (2) Shearing failure on sub-soil,
(3) Permeability of embankment body

↓

4. Detailed Structural Examination:
(1) Slope revetment works, (2) Foundation works, (3) Foot protection works,
(4) Wave breaking works

↓

END

Figure 9.5 Design Procedure for Coastal Slope Revetments

The topography of the seaside and seabed sensitivity affects the peculiarity of the waves. On a coast with a steep sea-bottom slope, the waves are broken and the height of the breakers tend to be higher at the shore than where the seabed has a shallow slope, consequently, overflow and stronger wave force are apt to occur. If the revetment is located on the coast beach, scouring damage to the dike foot is apt to occur at flood tide due to current turbulence.

c) Ground conditions

In the case of slope revetments built on weak ground, sub-soil replacement, piling work or soil improvement to increase the bearing capacity is considered in the foundation design. In addition, the borrow material for the embankment is selected to be sufficient for the degree of compaction requirement and to have low permeability.

d) Seismic conditions

The authority should establish standard specifications to regulate what seismic loads should be used as input into the design of structures taking into consideration the stability of the bank, the importance of the road, probability of earthquake, social conditions in the surrounding area and so on.

e) Construction conditions

Offshore work is restricted by the daily variation of tides, waves and tidal current which affect work time. Preventive measures to minimize water pollution caused by construction should also be considered in construction planning.

f) Social and environmental conditions

Specified conditions relative to the prevention of degradation or loss of cultural heritage assets, national parks, use of the seacoast for things such as fishery, sightseeing, marine leisure and recreation, future development planning, are all to be considered in the design.

9.3.2 Type of Revetment Structure

The type of revetment structure is determined from comprehensive examinations and study of hydraulic conditions, sub-soil conditions, materials available for the embankment, land acquisition requirements, seaside use, construction conditions and so on.

Table 9.1 Slope Revetment Structural Types

Sloping Type and Gently Sloping Type	Stone pitching type, Concrete-block pitching type, Blanket concrete type, Rubble stone type, Mass concrete block type
Standing Type	Wet masonry type, Gravity type, Concrete buttress wall type, Reverse T-type or L-type concrete wall type, Caisson type, Cellular type, Sheet pile type, Concrete-block piling type , Rock-fill type
Combination Type	――――

Source: Modified from Reference No. 13, National Association of Sea Coast, Japan 1987, Revisional Technical Criteria and Commentaries on Construction of Costal Protection Facilities

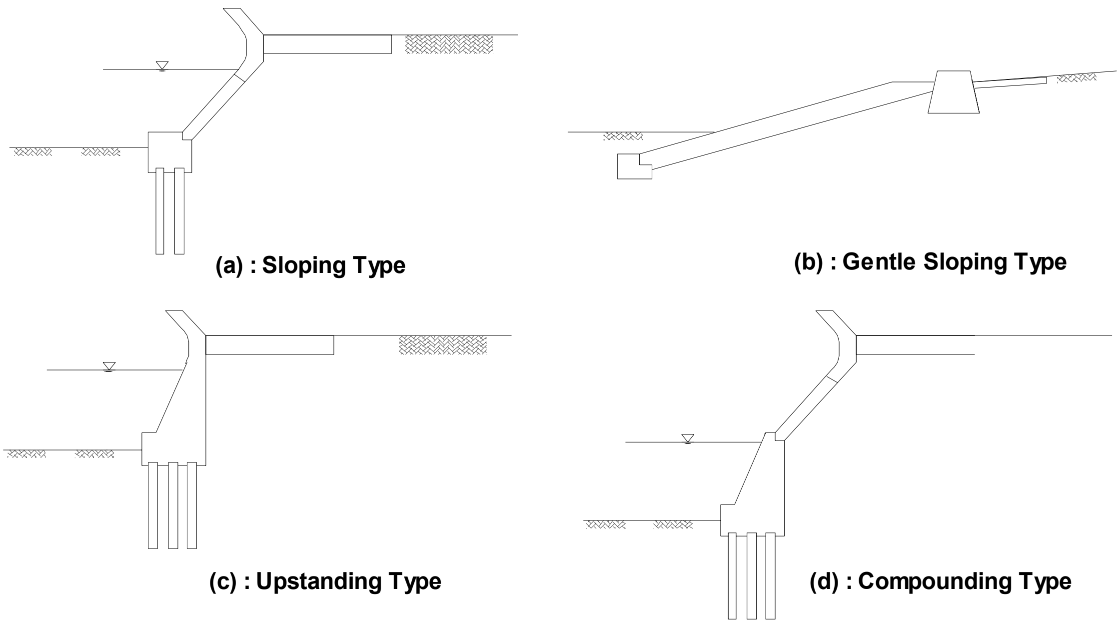


Figure 9.6 Classification of Slope Revetment by Slope Gradient

Source: Modified from Reference No. 13, National Association of Sea Coast, Japan 1987, Revisional Technical Criteria and Commentaries on Construction of Costal Protection Facilities

Table 9.1 shows the structure type to be adapted for various slope revetments and Figure 9.6 illustrates the classifications of revetment type. The revetment types are classified into four types pursuant to the gradient of the revetment slope, which are the sloping type, gentle sloping type, standing type and combined type;

Sloping Type Revetment is for slope gradients more than 1:1.0,

Standing Type Revetment is for slope gradients less than 1:1.0,

Gentle Sloping Type Revetment is for slope gradients of more than 1:3.0 in the sloping type revetment category,

Combined Type Revetments are usually applied in locations where there is deep standing water. These are a combination of the standing type in the lower part and the sloping type in the upper part as installed in Figure 9.6 (d), or to install caissons or to pile concrete blocks on the inclined structure like a rubble stone mount.

The favourable conditions for adopting each type revetment are summarized as follows:

a) Standing type revetment

- Sub-soil is comparatively hard ground.
- When land acquisition for construction is relatively difficult, the standing type has the advantage of requiring a relatively small amount of land/working area.
- The standing type is deemed to be suitable under favourable hydraulic conditions and compatibility with the existing revetment.

b) Sloping type revetment

- Sub-soil condition is comparatively soft ground.
- Where hydrophilic soil and/or seaside installation are required.
- Embankment material is available in adequate quantities near the construction site
- Land acquisition for construction is relatively easy, since the construction area required is wider than the other types.
- The sloping type is deemed to be suitable under favourable hydraulic conditions and compatibility with the existing revetment.

c) Gentle sloping type revetment

- The new revetment is to be constructed on a wide seacoast of which the seabed inclination is gentle.
- On the above seacoast, the gentle sloping type revetment is provided at the front side of the existing revetment to strengthen, reinforce and improve the existing.
- The additional revetment can be provided as an alternative measure against overtopping and splashing due to waves where the existing standing type revetment was located on a seabed that is gently inclined, even if the front has been scoured.
- The gently sloping revetment can be constructed by piling blocks with enough weight in front of the existing revetment to improve or reinforce it if the existing revetment is located on a gently inclination seabed, even though the water depth at the slope foot is deep.

9.3.3 Design Crest Elevation of the Revetment

The design crest elevation of the revetment or top elevation of the recurved parapet, if provided, is determined from the following three elements:

Design Crest Elevation = (Design Tide Level) + (Design Wave Height) + (Free board)

1) Design tide level

The design tide level used is higher high water, which the most critical elevation as regards overtopping and splashing waves. The design water elevation is generally set up based on the following flood tide level;

- Highest high water level in past records, or
- Mean high water level plus maximum deviation of past tide level, or
- Mean high water level plus predicted maximum deviation of tide level.

2) Design wave height

a) Design wave to be considered

Probable wave of 30 to 50 years return is generally adopted for the design wave, of which the period corresponds to the projected operational period of the infrastructure.

b) Design height of design wave

The required height of the design wave is determined with due consideration of the effect of splash. However, this height prevents only overflowing of the substance of the wave; therefore, overtopping waves may occur in actual wave conditions. From this viewpoint, the design height of the wave is determined considering the following;

- To control the amount of overflow water within an allowable scale based on the importance and traffic volume of the road.
- To set up the required height to prevent wave overtopping by calculating the height of the wave spray.

Calculation of wave height to determine the design crest elevation is based on the following local conditions relative to revetment alignment;

- In case the revetment alignment is located offshore of the shoreline, the elevation is determined from the discharge volume of wave overtopping.

c) Other considerations

- In case of forecasting consolidation settlement due to soft ground or settlement of bank body due to earthquake reaction, the design crest elevation as determined from the hydraulic aspect is to be increased to meet forecast settlement.
- The tide height and overflowing volume on a gentle sloping revetment of which the slope gradient is approximately 1:3.0 are not reduced as much compared with the standing type revetment. Adopting a more gentle slope gradient, increasing the roughness of the revetment surface, or selection of a stair type revetment contribute to the reduction of overflowing and splashing.

3) Free board

There is no certain factor being adopted in the design tide level and design wave height for setting up the design crest elevation. The idea of introducing free board is aimed to counterbalance the said uncertain factor. The height of 1.0 m is given as maximum free board considering road importance, adjacent urbanization conditions, public facility locations and so on.

9.3.4 Gradient of Slope Revetment

Standard gradients for each revetment type are given in Table 9.2. The gradient of a revetment slope is determined from the stability of the bank, hydraulic condition, use of seashore, sub-soil and topographic conditions. The scouring at the toe and stability of the bank body must be considered, when the revetment is being constructed in deep water and steep sea bottom slope.

A gentle gradient revetment slope is recommended as a measure in case of powerful wave pressure acting on the revetment or where the seashore is suitable for recreational act utilized for sea bathing and sightseeing. However, the possibility of overflowing should be noted. In order to avoid this problem, increasing surface roughness or adopting a concrete block type or stair type revetment is effective for the reduction of overtopping waves.

For the gentle sloping type revetment, the swash height as well as reflectivity of the waves may decrease in proportion to the inclination of the slope. Being gentler, it may be expected to mitigate the scouring on the other side. However, increased covering of the beach front due to the slope being gentler leads to a decrease in the wave breaking action of the natural breach, and reduces the available beach space.

Table 9.2 Standard Gradient of Revetment Slopes (Modified from reference No. 12)

Revetment Type	Slope Gradient	Revetment Type	Slope Gradient
Stone pitching type	More than 1 : 1.0	Gravity type	Vertical ~ 1 : 0.5
Concrete-block pitching type		Concrete buttress wall type	
Blanket concrete type		Reverse T-type (or L-type) concrete wall type	
Rubble stone type	1 : 1.0 ~ 1 : 3.0	Cellular type	Vertical ~ 1 : 0.4
Mass concrete block type		Sheet pile type	
		Concrete-block piling type	Vertical ~ 1 : 1.0
Wet masonry type	1 : 0.3 ~ 1 : 1.0	Rock-fill type	

9.3.5 Stability of Bank Body

1) Embankment material

Sandy soil with moderate clay codification or gravel sandy soil is recommended to be used as embankment material, since the material requirement is trafficability(construction car's run-ability) and sufficient compaction. The following standard quality values are given;

- Grain size : within a grading curve range as given in Figure 9.7.

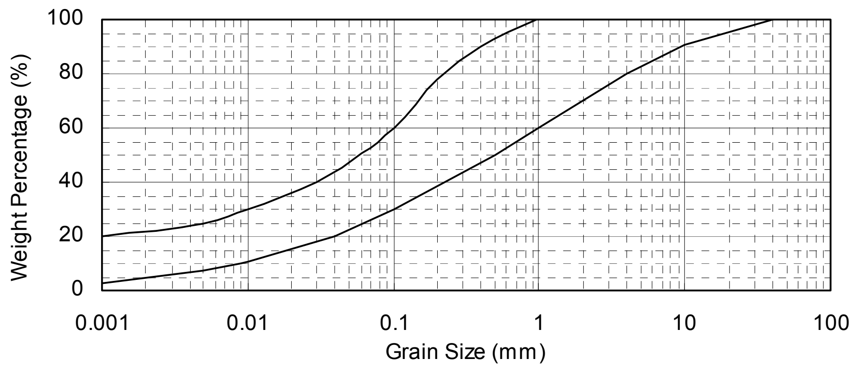


Figure 9.7 Grading Curve of Range of Embankment Material (Modified from Reference No. 14)

- Maximum dry density of soil = more than 1.0 t /m^3
- Internal friction angle = more than 30^o
- Coefficient of permeability = less than 2 x 10^{-3} m/sec

2) Shearing failure of subsoil ground

In case the bank stands on weak ground, there is a possibility that a shearing failure may occur in the ground and the entire bank body break to slip. Major factors that cause this type of slip failure are the deadweight of the bank and overhead loading water pressure. Additionally, seismic force and wave force act as sliding forces. Foot protection is expected to act as a counterweight against sliding. Stability analysis methods are given as follows;

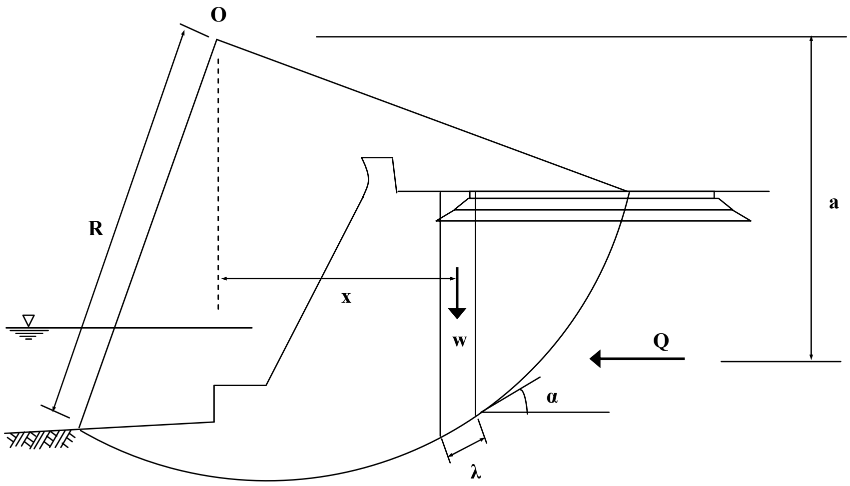


Figure 9.8 Illustration Explaining Circular Failure Analysis

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

$$F = \frac{R\sum(c\lambda + w\cos\alpha\tan\phi)}{\sum wx + \sum Qa} \qquad \text{(Modified Fellenius Method)}$$

where,

F = Safety factor against circular failure

R = Radius of circular sliding (m)

c = Cohesion of soil (kN/m^2)

λ = Length of base of divided soil piece (m)

w = Effective weight of divided soil piece (use submerged unit weigh in case of under water) (kN/m)

α = Angle of divided soil piece (deg)

x = Distance between centre of gravity of divided soil piece and centre of circular sliding (m)

Q = Horizontal forces to act on soil mass (wave force, seismic force, water pressure and so on.) (kN/m)

a = Distance between horizontal force and centre of circular sliding (m)

(Modified from Reference No. 12)

3) Permeability of bank

a) Washing-out of embankment material

Continuous wave action causes washing-out such that that embankment materials flow out through the gaps at the surface coating of the revetment, foundation and permeable rubble stone, which advances collapse. From this technical point of view, joints in the slope surface coating and connections between the surface revetment and the foundation should be taken care of to avoid any gaps or cracks, and to maintain continuality.

c) Piping

When the creep ratio, which is a function of the ratio of the difference between the inside and outside water head and the infiltration route length through the bank, is large, the piping phenomena can occur at the toe of the revetment. To ensure safety against piping, it is required that the creep ratio parameter, C_e, should be below the value given in Table 9.3.

$$C_e = \frac{B/3 + l}{h_{er}}$$

where,

C_e = Creep ratio with weighting factor
l = Water head difference between inside and outside of bank (m)
B = Length of horizontal infiltration path (m)
h_{er} = Length of vertical infiltration path (m)

Table 9.3 Creep Ratio with Weighting Factor by Soil Classification

Soil Material of bank or sub-soil	Creep Ratio	Note
Silt	8.5	Use low permeable material between bank material and sub-soil material.
Fine Sand	7.0	
Coarse Sand	6.0	
Medium Sand	5.0	
Gravel	3.5	
Gravel mixed Boulders	3.0	

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

9.3.6 Structural Details

1) Slope revetment work

Since the slope revetment should protect the bank body and prevent incursion of flood tide and waves, the following performance is required;

- Resist external forces such as earth pressure, wave force and seismic force,
- Withstand the erosion by wave action,
- Prevent outflow and slip of the bank material.

Under powerful wave force or uneven settlement, reinforcing bars should be installed in the covering slab. The gradient transition of the revetment slope should vary smoothly and gradually since the wave force concentrates on the said section and this becomes its structural weak point.

Table 9.4 is given the standard structural requirements for typical slope revetment types.

Table 9.4 Standard Requirements for Typical Slope Revetments

Revetment Type			Standard Requirements
Sloping Type	Stone pitching type		- Brace length of stone: more than 35 cm - Thickness of backfill material : more than 30 cm
	Concrete-block pitching type		- Thickness of block: more than 50 cm - Total weight of blocks : more than 2 ton - Thickness of backfill material : more than 50 cm
	Blanket concrete type	Flat casting type	- Thickness of concrete: more than 50 cm - To provide concrete backfill.
		Stair type	- Thickness of concrete: more than 50 cm - Stair Height : 20 ~ 30 cm
		Frame type	- Width of crib: 20 ~ 30 cm, frame height: 30 ~ 50cm . - Crib structure: reinforced concrete. - Crib interval: 1 ~ 3 m (to provide haunch) - Filling inside of crib: with stone or concrete blocks
Standing Type	Wet masonry type		- Use wet masonry in principle. - Brace length of stone: more than 35 cm - Thickness of concrete backfill: more than 10 cm - Thickness of backfill material: more than 50 cm
	Concrete-block piling type		Same as Wet masonry type
	Gravity type		As per structure of bank
	Concrete buttress wall type		Thickness of wall: more than 50 cm (plain concrete), more than 30 cm (reinforced concrete) Interval of buttresses: 3 m as standard

Source: Modified from Reference No. 14 Japan Port & Harbour Association April 1994, Technical Standards and Commentaries for Port and Harbour Facilities

a) Expansion joints and construction joints

For the site-cast concrete revetment type, expansion joints should be provided at every 6 m to 10 m interval. To prevent outflow of bank material, water stops and slip bars as illustrated in Figure 9.9 should be installed at the expansion joints. Bituminous material or synthetic resin material is commonly used as joint filler material.

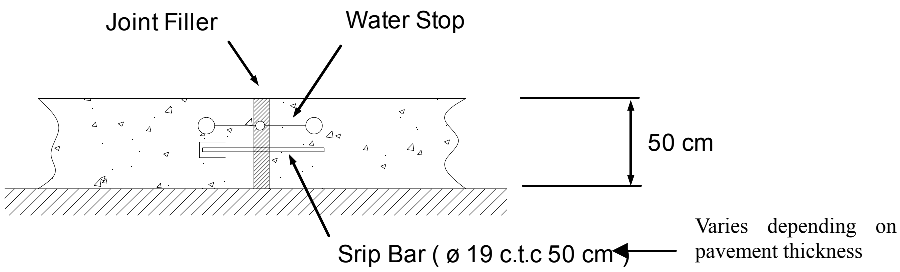


Figure 9.9 Example of Expansion Joint

Source: Modified from Reference No. 13 National Association of Sea Coast, Japan 1987, Revisional Technical Criteria and Commentaries on Construction of Costal Protection Facilities

Regarding the construction joints of site cast concrete, the horizontal jointing causes the development of cracks. In order to avoid such damage, splice reinforcing bars are provided at the joint portion at a right-angle to the joint, especially for the gentle sloping type revetment, as shown in Figure 9.10.

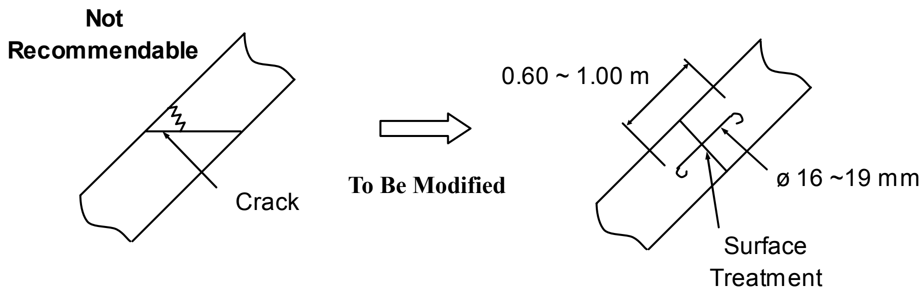


Figure 9.10 Treatment of Construction Joints (Modified from Reference No. 15)

b) Stone Pitching Type Revetment

The advantages of the stone pitching type are higher flexibility and ease of construction, therefore, this type of revetment is suitable for the locations with lower wave height and uneven settlement due to soft ground. On the disadvantage side, the tendency to be scattered by wave force may be pointed out.

Stones of more than 35 cm of brace length are used as surface covering material and crusher run of more than 30cm of thickness, which consists of cobblestone of 5 to 15 cm diameter and gravel is used for fill.

c) Concrete-block Pitching Type Revetment

An example of a concrete-block type revetment is shown in Figure 9.11. This type of revetment is also applied to ensure flexibility where there is the potential for uneven settlement on soft ground. In addition, it is applied to prevent continuous erosion caused by reflected waves in front of the revetment slope on an erosive seacoast. This revetment type has the following features;

- The extremely rough revetment surface may contribute to break the waves as they run-up along the revetment and to reduce water velocity and the amount of returning flow.
- The revetment body has flexibility to follow deformations due to front scouring on some level.
- If the bank soil is washing out, the concrete blocks may cover the surface to prevent complete collapse or levee crevasses.
- If the revetment is ever destroyed, it is relatively easy to rebuild it by collecting the scattered or submerged blocks.

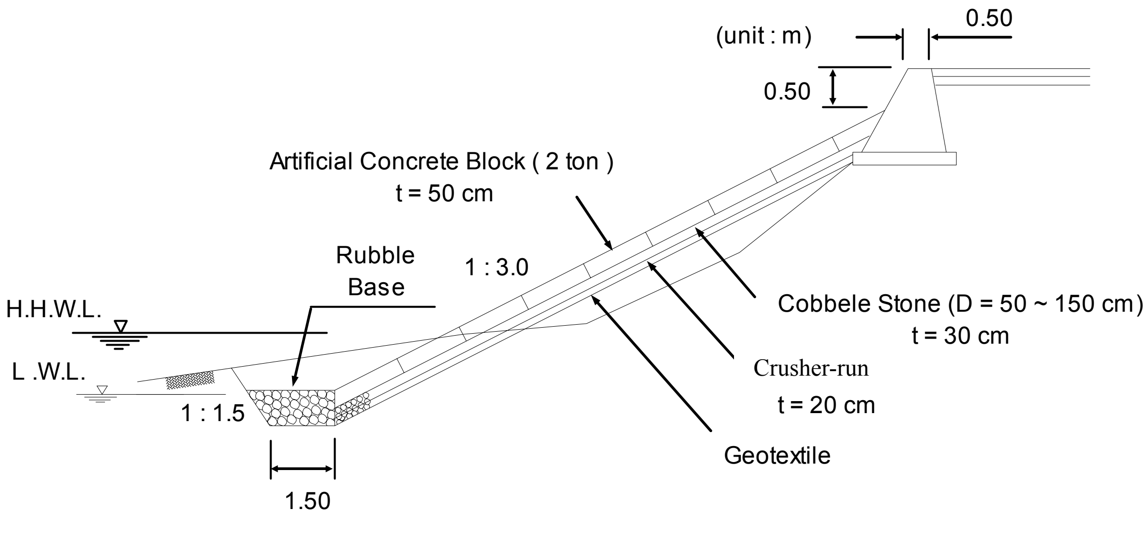


Figure 9.11 Example of Concrete-block Pitching Type Revetment

[Embedment of Gentle Sloping Type Revetment]

The gradient of the slope should be gentler than 1:3.0 and the dike foot should be embedded into the ground at the same gradient in order to avoid continuous scouring caused by reflected waves on the block surface, as shown in Figure 9.12.

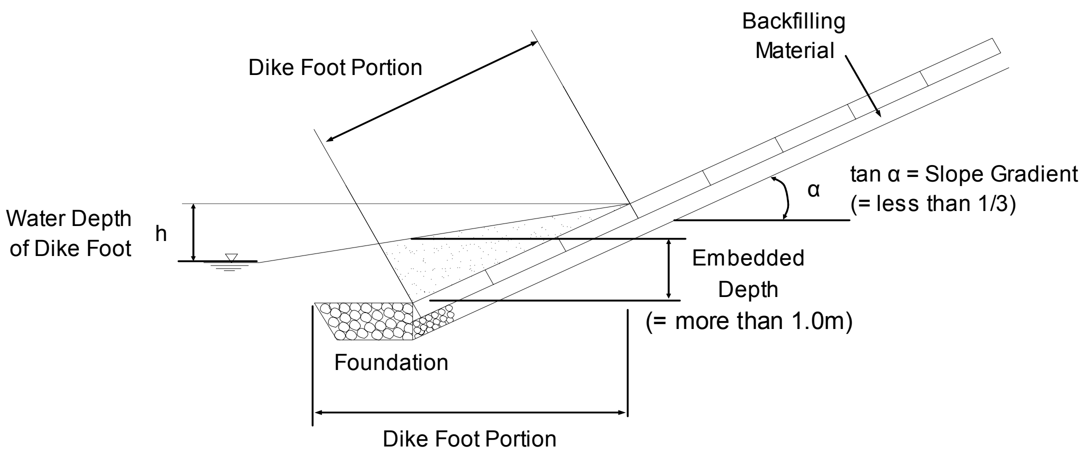


Figure 9.12 Embedment of Gentle Sloping Type Revetment

Source: Modified from Reference No. 17 Japan Port & Harbour Association April 1989, under the editorship of Ministry of Construction, Japan, Guide to Design of Gentle Slope Dike

The embedded depth is required to ensure safety against the following topography variations:

Topography Variation (i) : seacoast deformation caused by coastal erosion due to disproportion of coastal littoral drift.

Topography Variation (ii) : seacoast variation caused by littoral drift due to high waves in an offshore direction, even where a slope revetment is not installed.

Topography Variation (iii): local scouring at dike foot due to high waves.

In case of dry work construction, the embedded depth should satisfy the following condition;

Embedded Depth > {Long Term Deformation by (i)} + {Maximum Scouring Depth (ii) added to (iii) }

In normal case of gentle sloping type revetment, an embedment depth of more than 1.0 m is required and foot protection should be provided for scouring prevention. The concrete foundation is installed under water.

[Backfilling of Revetment]

Thickness of backfill material must be more than 50 cm. The following two functions are required for backfill;

- To strengthen the bearing capacity of the slope soil surface.
- To perform as a filter for seepage water from outside of the revetment and inside of the bank body.

The appropriate backfilling arrangement may be expected to prevent scouring, since return flow due to the infiltration effect is reduced. Types of backfill material are classified as follows;

- Cobble stone, gravel, crusher-run
- Wire mats and wire cylinders

Since the length of the slope for the gentle sloping type revetment is longer, the washing-out of backfill material may occur around the shoreline as explained in Figure 9.13, even if the thickness of the backfill meets the requirements. The following treatments (as shown in Figure 9.14) are recommended measures for washing-out damage:

- Divide backfill layers into two layers of which grain size ratio (d /D) is more than 0.15 (Figure 9.14 (a)).
- Lay geotextile material at the bottom of the backfill materials (Figure 9.14 (b)).

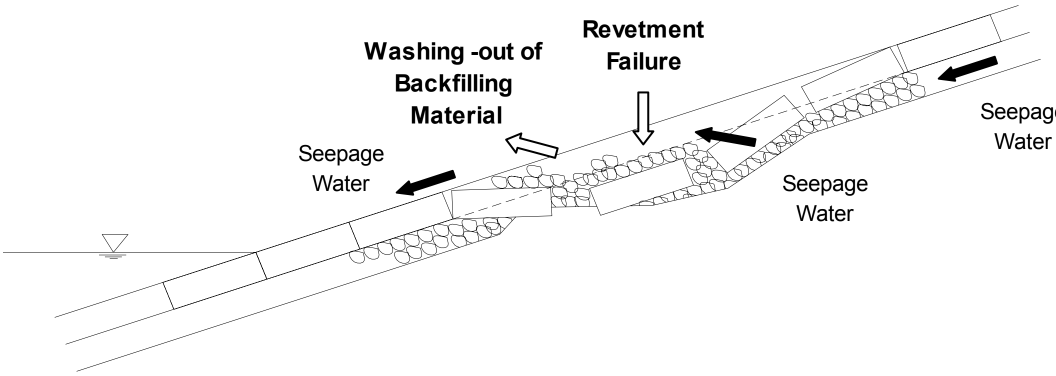
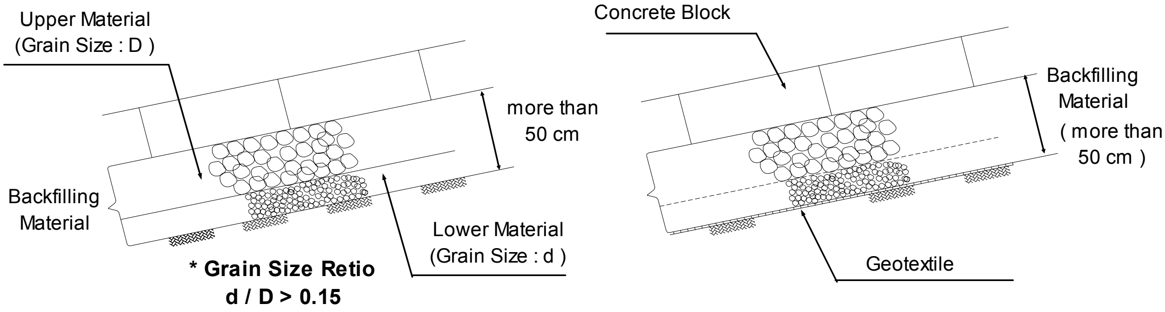


Figure 9.13 Example of Washing-out of Backfill Material on Shoreline

Source: Modified from Reference No. 17 Japan Port & Harbour Association April 1989, under the editorship of Ministry of Construction, Japan, Guide to Design of Gentle Slope Dike



(a) Grain Size Retio of Backfilling Materials **(b) Example of Geotextile Spereding**

Figure 9.14 Measures to Prevent Washing-out of Backfill Material (Modified from Reference No. 17)

c) Blanket Concrete Type Revetment

[Flat Cast Type]

Figure 9.15 shows an example of a flat cast type of revetment. The required thickness of the blanket concrete is 50 cm, in principle, and thicker concrete is required at the locations where wearing out is to be expected, such as gravel seacoasts or boulder/cobblestone seacoasts.

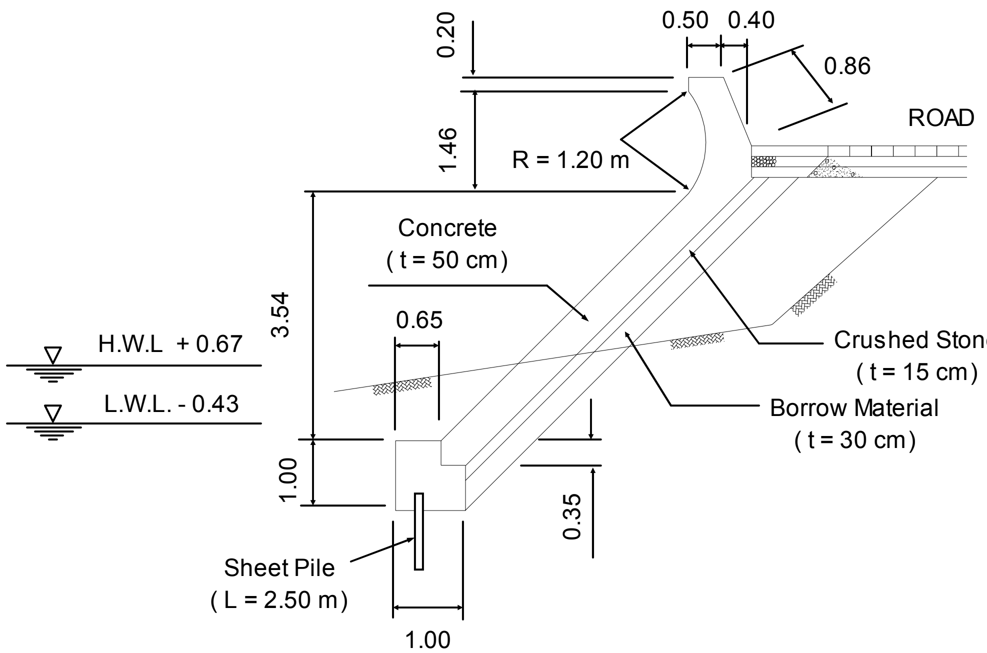


Figure 9.15 Example of Flat Concrete Cast Type Revetment

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

Cobblestone with a thickness of 30 cm as shown in Figure 9.16 (a) are commonly used for backfilling, however, this type of backfilling sometimes causes water leakage and washing-out. The following measures may be recommended as alternatives for cobblestone backfilling;

- To place levelling concrete on a coarse stone (or cobblestone) surface (Figure 9.16 (b)).
- Where bank material is sandy, place 5 to 10 cm thick concrete after surface trimming (Figure 9.16 (c)).
- Place 10 to 20 cm thick soil cement material or asphalt after surface trimming (Figure 9.16 (d)).

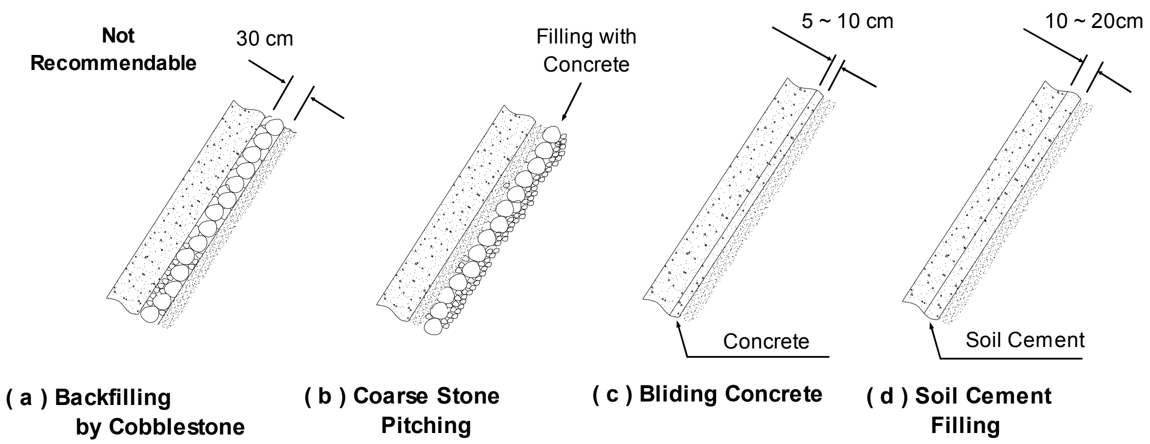


Figure 9.16 Backfilling of Concrete Cast Revetment

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

[Stair Type]

The stair type revetment also requires a concrete blanket thickness of more than 50 cm. A stair height of 20 to 30 cm is generally selected for the following reasons or conditions: i) decease the height of impacting waves along the revetment slope, ii) prevent scouring by reducing the velocity of the backwash, iii) use of seacoast.

It is possible that the desired concrete strength of onsite casting of concrete will not be attained by direct pouring without formwork. It is recommended that reinforcing bars be provided. Figure 9.15 shows an example of a stair type of revetment.

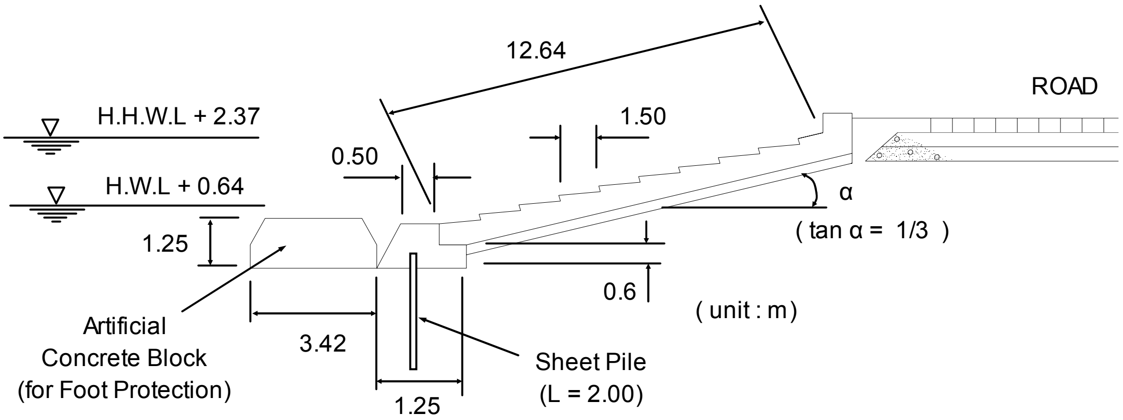


Figure 9.17 Example of Stair Type Revetment

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

[Frame Type]

Frame type revetments are applied in locations where continuous waves do not reach the revetment. The revetment is constructed by covering the existing sand-hill or beach with a concrete frame. Figure 9.18 presents a standard size of frame for a revetment.

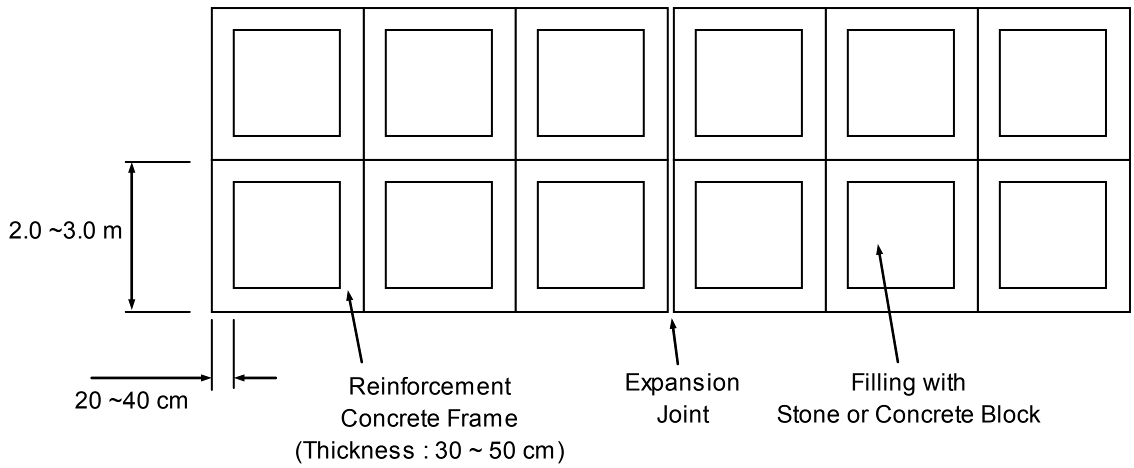


Figure 9.18 Standard Size of Concrete Frame

d) Gravity Type Revetment

Figure 9.19 shows an example of a gravity type of revetment. On the case of a reinforced concrete revetment, the minimum thickness should be 30 cm.

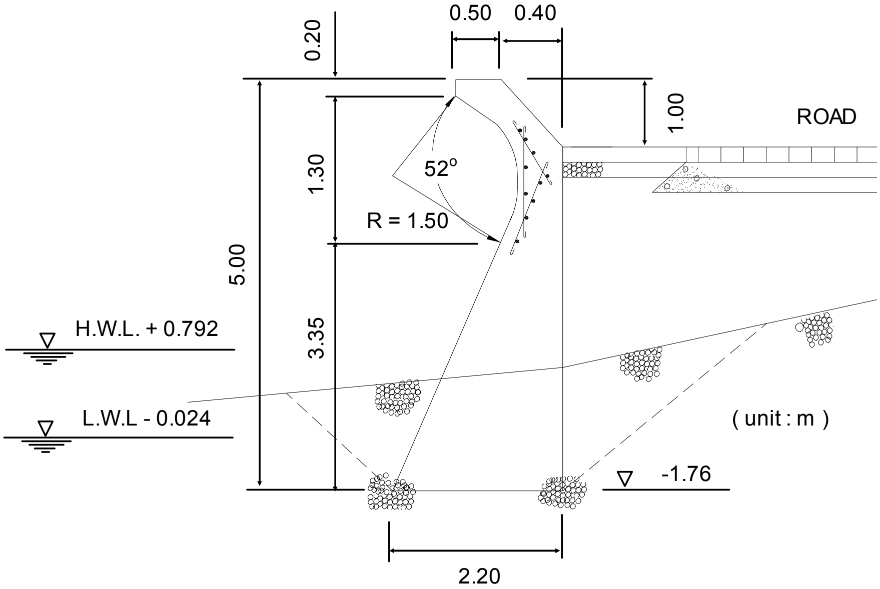


Figure 9.19 Example of Gravity Type Concrete Wall Revetment

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

e) Concrete Buttress Wall Type Revetment

The concrete buttress wall type is generally applied when the weight of the revetment must be reduced. The minimum thickness of reinforced concrete and buttress wall should be 30 cm at the thinnest section, in consideration to the reaction of seawater with steel. Figure 9.20 shows an example of this type of revetment.

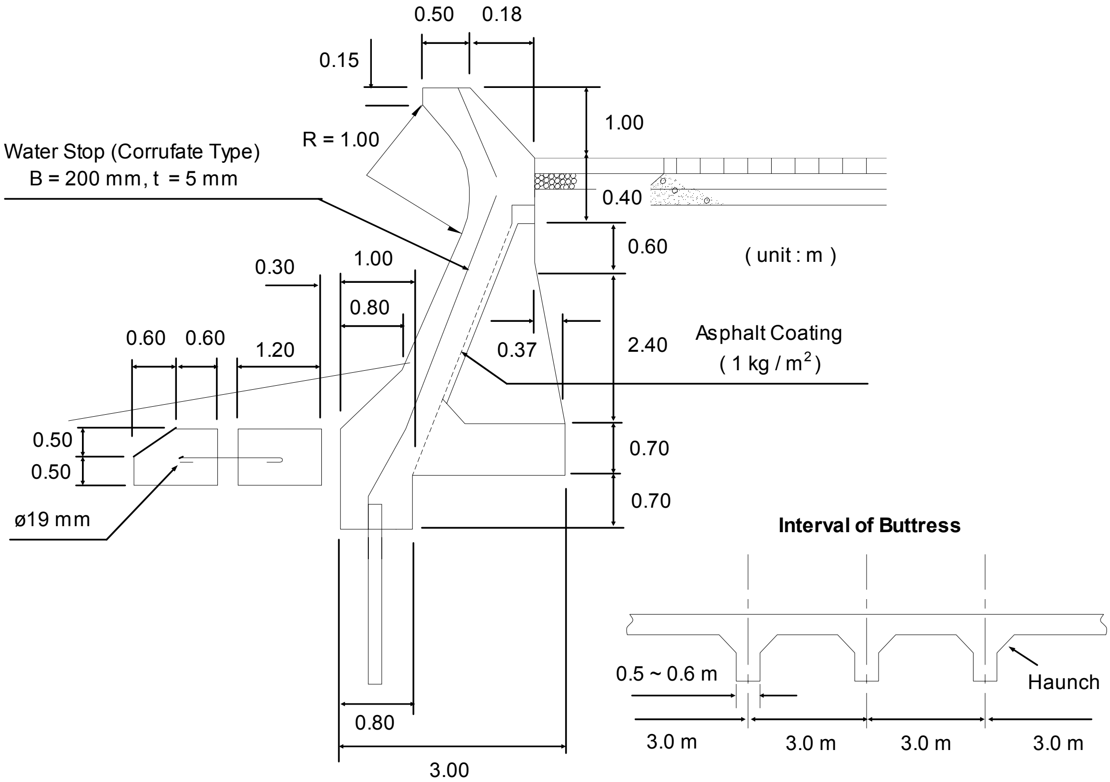


Figure 9.20 Example of Concrete Buttress Wall Type Revetment
Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

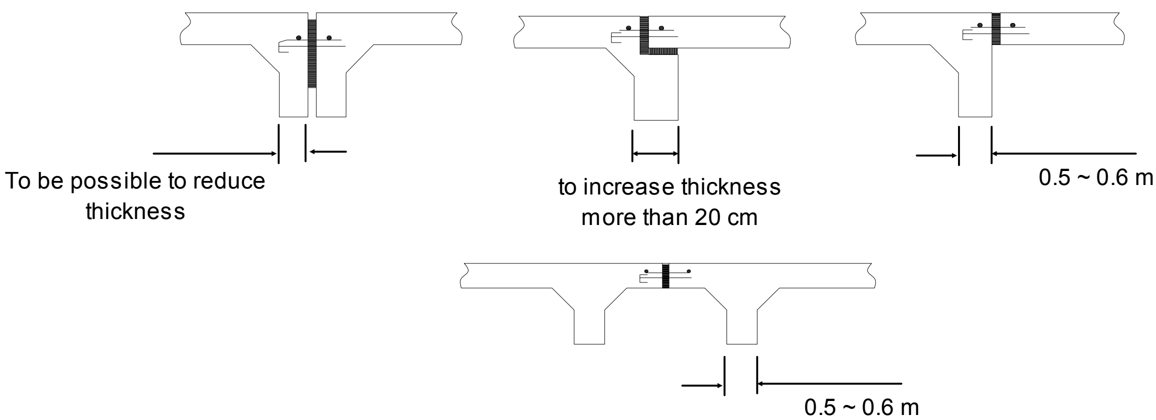
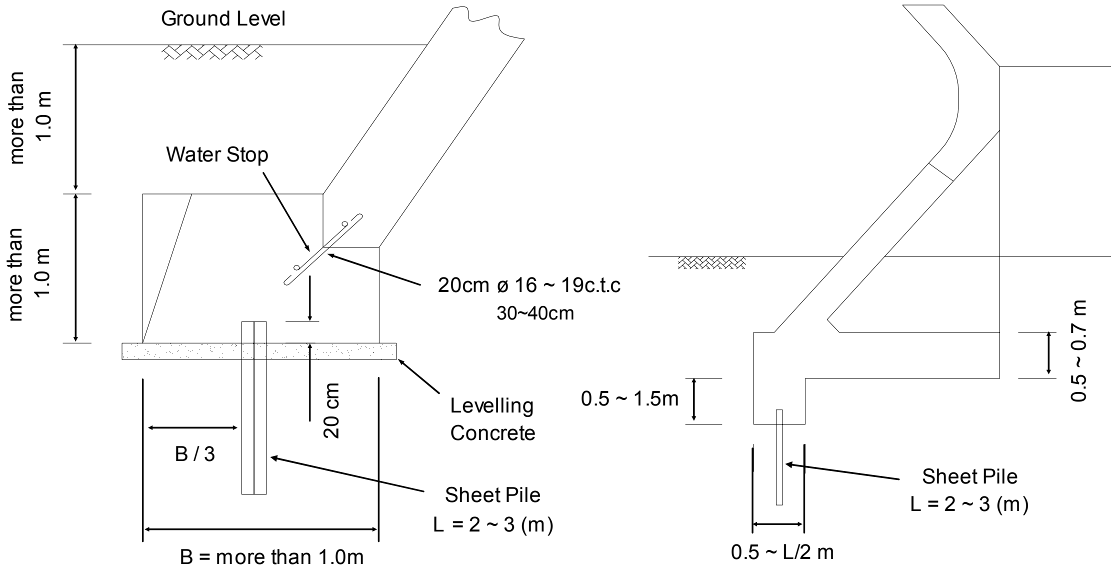


Figure 9.21 Expansion Joints for Concrete Buttress Wall
Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

3) Foundation Work

a) On-Site Casting of Concrete Foundations

The foundation of slope revetments should prevent the slipping and settlement of the upper structure and withstand scouring by wave action.



(a) Foundation of Sloping Type Revetment **(b) Foundation of Concrete Buttress Type Revetment**

Figure 9.22 Site Cast Concrete Foundations

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

Figure 9.22 shows examples of on-site cast concrete foundations. The foundation is generally embedded into the ground at more than 1.0 m. On the other side, the embedded depth may be reduced to approximately 0.5 m where the influence by waves is low.

In the case of sloping type revetments as shown in Figure 9.22 (a), the foundation concrete is more than 1.0 m in height and more than 1.0 m in width. Leveling concrete with a thickness of 10 cm to 20 cm is placed under the concrete foundation (do not use cobblestone). Expansion joints, which include water stops and splice reinforcing bars, are provided at similar locations as for the upper revetment structure.

In the case of buttress wall type revetments as shown in Figure 9.22 (b), the thickness of the bottom slab is should be 0.5 m to 0.7 m, and water cut-off sheet piles are installed with the bottom slab, in principle. When building on a rock base, the cut-off wall is not required.

b) Rubble Stone Foundation

The foundation of concrete-block type revetments of the gentle sloping type is commonly of rubble stone. If the seabed is hard enough such as rock, no foundation is required. When sufficient embedment depth to protect against scouring is ensured, a large size foundation is not required. In such case, in order to prevent settlement due to the action of the waves, a rubble stone or coarse stone foundation may be adopted as shown in Figure 9.23 (a).

Where the installation is being done under water and there is no embedment, the foundation is installed to reduce negative influence from geographic variation on high waves. Figure 9.23 (b) shows standard requirements for installations without embedment, in which case the gradient of the rubble stone mount foundation varies from 1:4.0 to more than 1:2.0 when the foundation deforms due to subsidence of the ground caused by high waves.

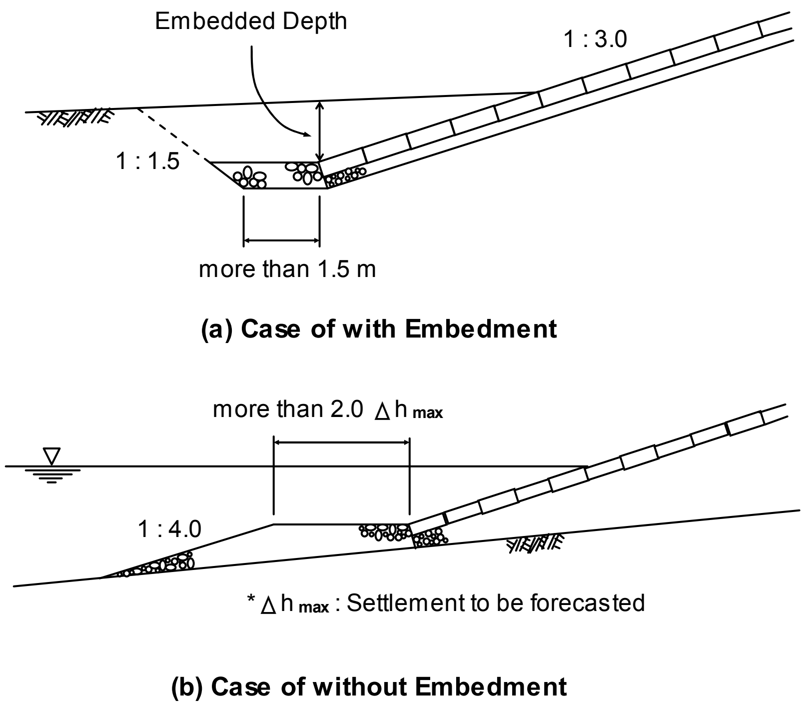


Figure 9.23 Rubble Stone Foundations

4) Foot Protection Work

Figure 9.24 shows a typical method of foot protection. The types of foot protection commonly used are the rubble stone or concrete-block types because the material is easily obtained, construction procedure is easy and the finished structure is flexible to move by itself. The foot protection should be placed on the end of the revetment slope or the front of the foundation to insulate it from the revetment body so that it will subside and/or deform separately.

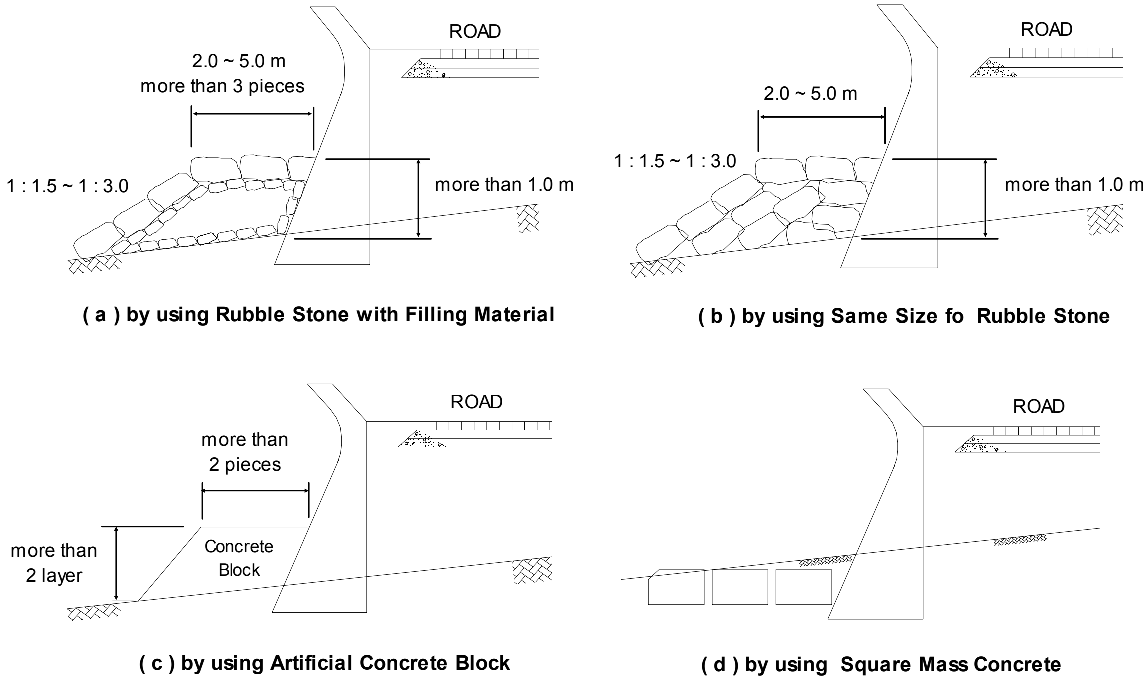


Figure 9.24 Typical Foot Protection for Slope Revetments
Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

When using rubble stone, it is required that the height be more than 1.0 m, the width of the crest be 2.0 m to 5.0 m and the front slope gradient be between 1:1.5 and 1.3. The required weight of each stone (or concrete-block) may be determined from the Hudson Formula as follows;

$$M = \frac{\rho_r H^3}{N_S{}^3 (S_r - 1)^3}$$

where,

M = Required weight of each rubble stone or concrete-block (ton)
ρ_r = Density of rubble stone or concrete-block (ton/m^3)
S_r = Ratio of the specific gravity of the stone or concrete-block to that of sea water (ρ_r/ρ_o)
ρ_o = Density of sea water (1.03 ton/m^3)
H = Design wave height (m)
N_S = Parameter based on shape, incline and damage ratio

$$N_S^{\,3} = K_D \cot\alpha$$

where,

α = Angle between slope and horizontal plane (deg.)

K_D = Parameter based on shape, incline and damage ratio

(Table 9.5 shows K_D-values of rubble stone by U.S. Army Coast Engineering Research Canter)

Table 9.5 KD-values of Rubble Stone by U.S. Army (C.E.R.C)

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

Rubble Stone Material	Number of Layers	Piling layout	K_D breaking waves	K_D non-breaking waves	$\cot\alpha$
Round Rubble	2	random	(1.2)	2.4	1.5 ~ 5.0
	more than 3	random	(1.6)	(3.2)	1.5 ~ 5.0
Squarish Rubble	2	random	2.0	4.0	1.5 ~ 5.0
	more than 3	random	(2.2)	(4.5)	1.5 ~ 5.0

* ( ) : supposed value

The rubble stone foot protection must key into the ground at the foot to absorb the energy of the waves acting on ground at the toe and to prevent washing out. From this viewpoint, stones with weights of 1/10 to 1/20 of the weight of the rubble stone to be placed on the top layer should be used as fill materials, and rubble stone with the required weight is arranged more than 3 deep on the top layer. If the revetment alignment runs along a shoreline, the foot protection is installed at approximately 1.0 m of the foundation depth or the crest width is increased to cover seabed steadfastly and to prevent scouring.

For concrete-block foot protection, cast blocks are commonly used. In this case, blocks more than 2 deep should be placed on the top layer and more than 2 layers should be created in order to ensure an engaging effect. However, the said foot protection type may suffer scouring at the toe of the foot protection, so toe protection of the lower bank and so on should be provided. And it is necessary that cobblestones or rubble stones are installed for the foundation of the concrete-block foot protection when the toe portion is located on sandy ground that would be apt to scour or wash out.

5) Wave Breaking Structures

The wave breaking structures are provided to reduce the affect of wave force and overtopping waves, and to reduce wash height. Several cast blocks may be built on the stone mount fill.

a) Mechanism of Breaking Waves

The wave striking the revetment from the sea deforms as follows:

i) A part of the wave reflects on the slope of the armour unit.

ii) A part of the reflected wave deforms into spray or large masses of water and flies up.

iii) The remaining water flows more slowly up along the slope of the armour unit and partially enters into the voids in the armour unit.

iv) After the voids are filled with water, the remaining water proceeds to the crest of the armour unit.

v) The rising water collides with the revetment slope or armour unit and partially jumps up as spray or large masses of water.

vi) A part of spray or large masses of water reflects, and the remaining water overtops the crest of the revetment and flows onto the road.

Considering the above phenomena, breaking up the wave force effectively requires the following conditions:

- Block or stone to be used for the wave breaking structure should have sufficient surface roughness.
- The armour unit must have moderate size, shape and distribution of voids according to the design wave.
- The armour unit must have moderate void volume so that some of the water accumulates inside of the unit. This corresponds with the portion of "ii" as shown in Figure 9.25 that is to be located above the sea surface, where greater designed volume is preferable.
- The crest of the recurved parapet wall portion of the slope revetment is designed to be moderately higher than the crest of the armour unit.

b) Standard Wave Breaking Structures

No determinate theory for the design of wave braking structures has been formulated up to now. Most studies and research concerning to effectiveness of wave braking structures are based on experimental model tests and no data to be adopted for general design methods has been accumulated. The following design principles, based on the past model tests and case studies of constructions are given:

- The crest of the amour unit is commonly 2 to 3 blocks wide. The minimum block arrangement on the crest should be a width of 2 lines. When the revetment is located where there are long period waves or the water is deep, the crest should be wider, sometimes 3 to 5 lines or more.

- The height of the crest from the water level is determined from the standard parameters given in Table 9.6. The height between the crest of armour unit and revetment ("a" in Figure 9.25) should be at least 1.0 m.

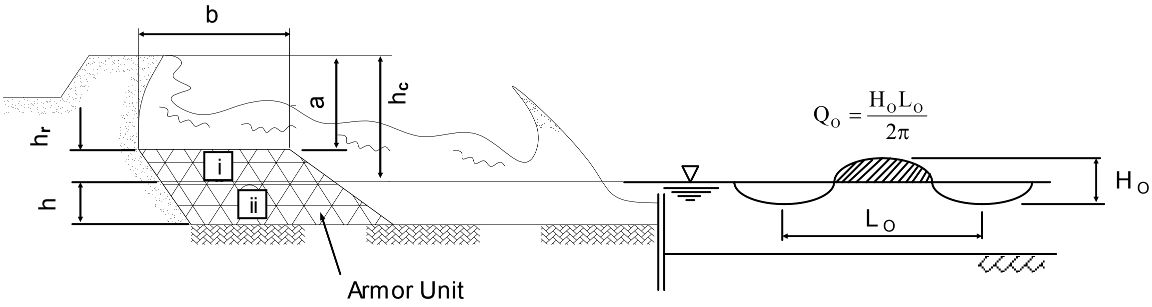


Figure 9.25 Illustration of Wave Breaking Structure

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

Table 9.6 Standard Parameters for Wave Breaking Structures

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

Water Depth \ Height of Armor Unit	(1) $\frac{H_r}{H_o}$	(2) $\frac{H_e + H_r}{H_o}$	(3) $\frac{H_e + H_r}{h}$	(4) $\frac{2\pi S}{H_o L_o}$
$h / H_o = 0$	0.5	1.0	-	0.1
$h / H_o = 0.5$	0.6	1.5	3.5	0.2
$h / H_o = 1.0$	0.8	2.0	2.5	0.4
h / H_o = more than 2.0	0.7	1.7	1.9	0.2

where,

h = Water Depth of Dike Form (m)

h c = Height of Dike (Parapet) Crest from Water Level (m)

h r = Height of Armor Unit Crest from Water Level (m)

a = Height between Crest of Armor Unit and Dike Crest (a = hc – hr) (m)

S = Area of Armour Unit under Water Level (m^2)

H o = Height of deepwater wave (m)

L o = Length of deepwater wave (m)

- Slope gradient of armour unit is 1:1.3 to 1:1.5.
- The required weight of the cast concrete blocks is based on the Hudson Formula mentioned in Clause 9.3.4 4).

6) Recurved Parapet Wall

The recurved parapet aims to eliminate overtopping waves or swash. Figure 9.26 shows an example of a recurved parapet wall.

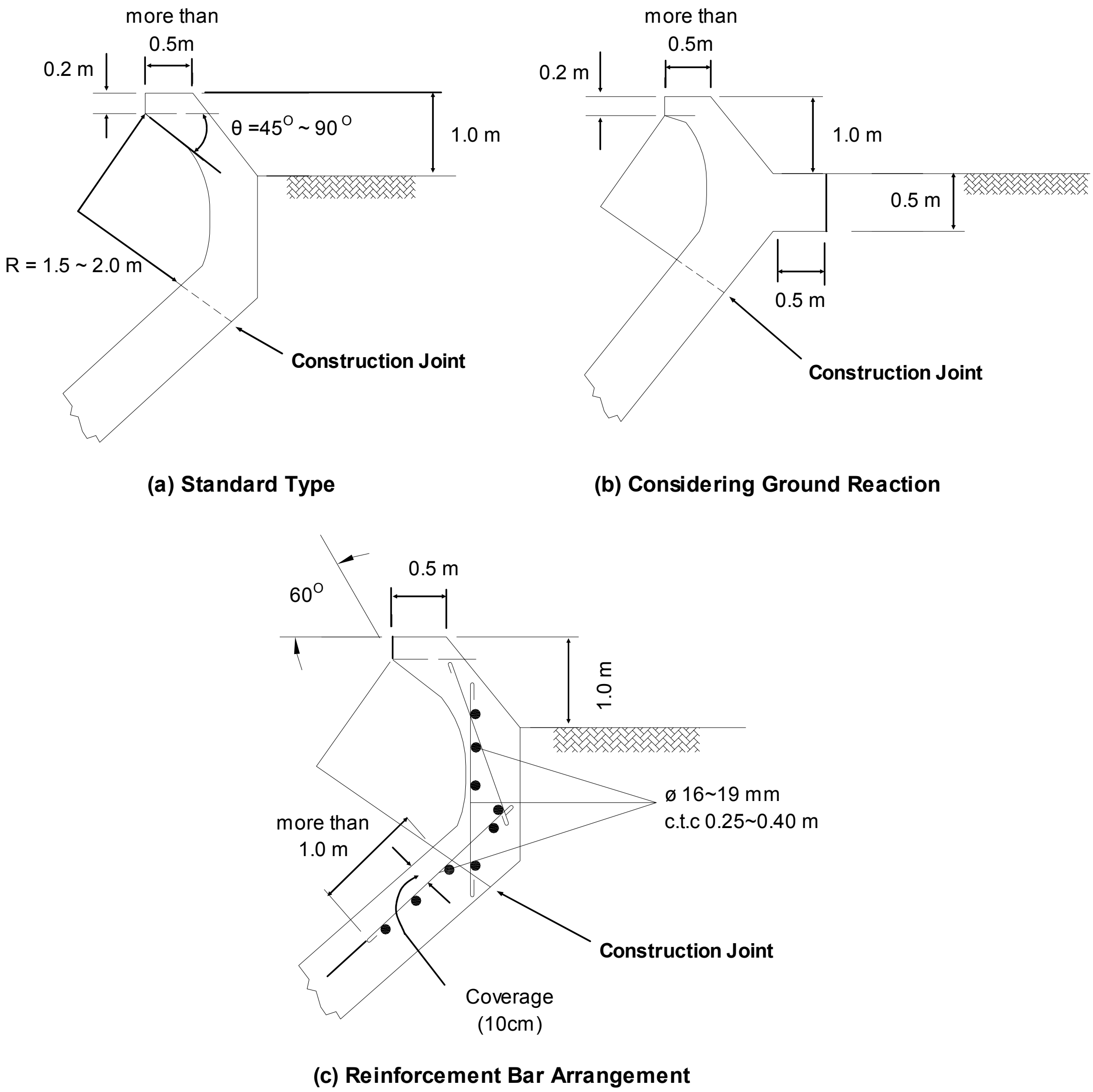


Figure 9.26 Examples of Recurved Parapet Wall Structures

Source: Modified from Reference No. 12 Costal Development Institute of Technology Japan 2004, Technical Criteria and Commentaries on Costal Protection Facilities

The advantage of installing the said parapet is that the crest elevation of the bank may be decreased and construction cost of the revetment may also be reduced. However, the structure may become unstable due to wave force, since the parapet wall has the thin section. From this viewpoint, the following notes should be considered in the design of recurved parapet walls;

- The height of the parapet should be approximately 1.0 m. To improve driver visibility or sight distance or for aesthetic reasons, the height of the parapet may reduced to less than 1.0 m, in which case 0.5 m to 0.8 m is common.
- The wall body should be built of reinforced concrete and the parapet should be firmly connected to the revetment body.
- Curve radius is 1.5 m to 2.0 m and the angle with a horizontal surface is 45^{o} to 90^{o} (angle of 60^{o} is most commonly used). It should be noted that recurved parapets with radii of only 0.5 m have been ineffective from past experience.
- The location of expansion joints in the recurved parapet should be arranged the same as the expansion joints in the slope revetment.

CHAPTER 10 EXAMPLES OF COUNTERMEASURE DESIGN

10.1 General

Several practical examples for which data was available from the disaster investigation stage to the restoration work design stage are presented in this chapter. These examples are given to provide a further understanding of the design procedure and process for the restoration of road slope disasters.

10.2 Examples of Countermeasure Design in the Philippines

10.2.1 Road Slips－Dalton Pass Road Km 211 (Region II)

(1) Background of Countermeasure

Km 221 at Daang-Maharlika(LZ), commonly called the Dalton Pass has incurred cracks and gaps in the pavement surface over a stretch approximately 80 m long for decades. Based on the boring investigation and site damage assessment, the following factors are assumed to be the cause of said damage;

- According to the topographic condition, the road slip seems to occur at the site where the road was constructed on an embankment, and the cracks are assumed to have developed at the boundary of the cut and fill as presented in Figure 10.1.

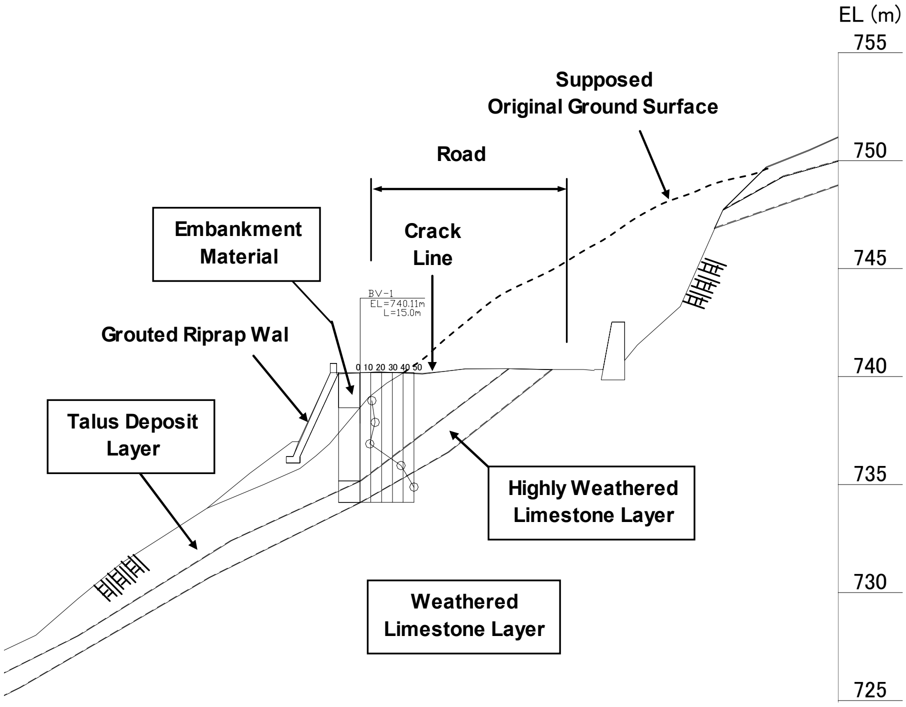


Figure 10.1 Engineering Geological Profiling at Dalton Pass Km211

- The grouted riprap was constructed on the slope surface of a talus deposit layer or fill layer. The said materials are relatively loose and soft since the N-value of SPT is approximately 10. One of the possible causes of the road slip may be that the bearing capacity of the foundation for grouted riprap is not sufficient, and consequently the road body embankment along with the riprap wall slid due to lost stability

- In addition, the circular slip stability analysis of this section resulted in a minimum safety factor of Fs=0.97, as shown in Figure 10.2, which means that the entire embankment slope was prone to slip.

Layer		Unit Weight	Shear Strength	
		γ (kN/m^3)	c (kN/m^2)	ϕ (deg.)
1	Embankment	19	0	25
2	Talus Deposit	18	0	30
3	Highly Weathered Limestone	20	180	0
4	Weathered Limestone (Base Rock)	21	-	-

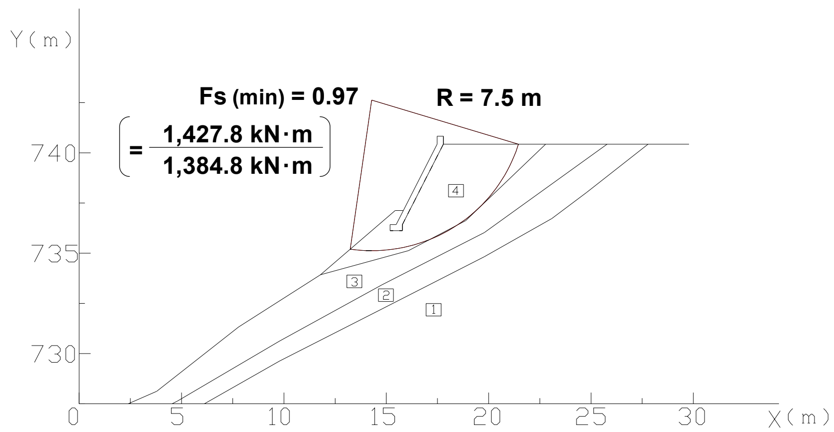


Figure 10.2 Result of Circular Slip Stability Analysis for Existing Condition at Dalton Pass Km 211

(2) Structure to be Applied and Design Procedure for the Countermeasure

The structure to be applied as a countermeasure is a double retaining wall combining a leaning concrete wall on the upper slope and a concrete block masonry wall, as illustrated in Figure 10.3 and Figure 10.4. Shape and dimensions of the retaining structures are as follows;

Wall Type : Leaning Concrete Wall (Upper Wall)
Concrete Block Masonry Wall (Lower Wall)

Height of Wall : H = 5.0 m (Upper Wall), H = 5.0 m (Lower Wall)

Gradient of Wall : 1:0.45 (Upper Wall), 1:0.50 (Lower Wall)

Foundation Type : Concrete Pile φ150 (Upper and Lower Walls)

Figure 10.5 shows the design procedure for the above structures, however, it should be noted that details of the design calculations for the concrete block masonry wall have been omitted since a standard structure type was adopted for the lower slope retaining wall.

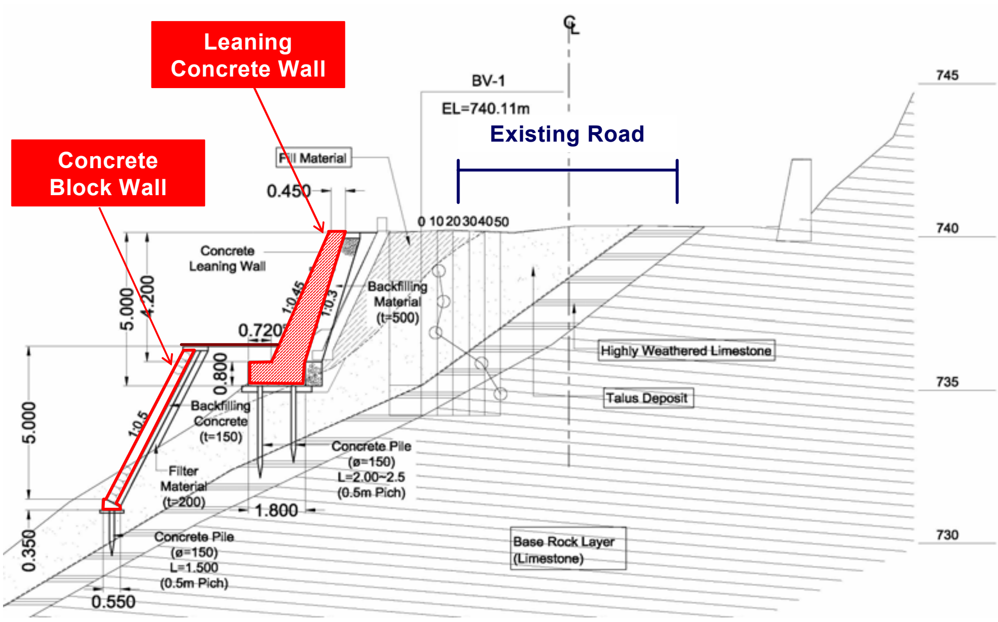


Figure 10.3 Typical Cross Section of the Countermeasure

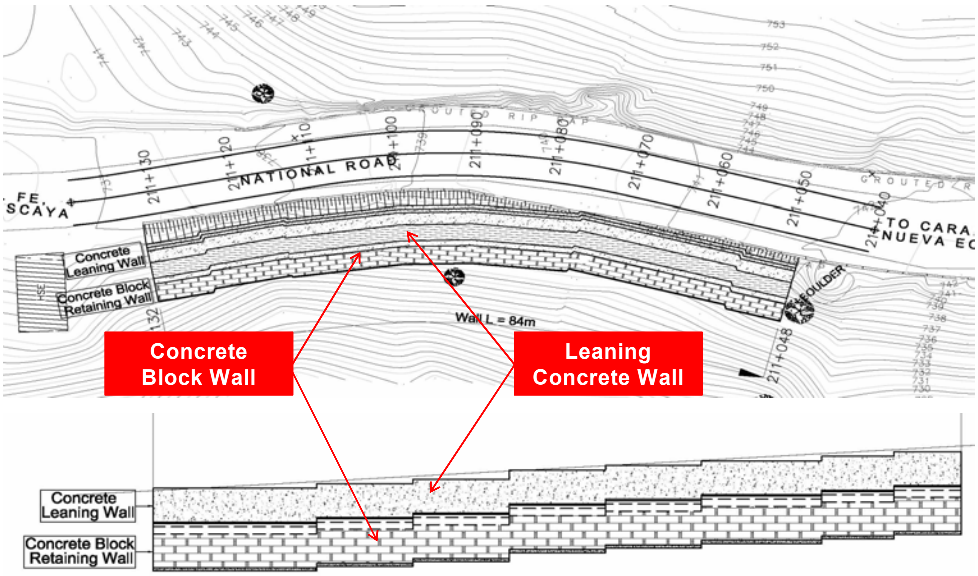


Figure 10.4 Plan and Side-view of the Countermeasure

Setting of Soil Design Parameters

↓

Setting of Design External Force : Dead Load, Earth Pressure, etc.

↓

Stability Design Calculation of Wall Body : 1) Tumble, 2) Slip, 3) Bearing Capacity

↓

Wall Foundation Design

↓

Circular Sliding Analysis for Double Wall Slope

↓

Concrete Stress Intensity Design Calculation

↓

End

Figure 10.5 Design Procedure for Double Retaining Wall Structure

(3) Design Conditions and Criteria

(a) Soil Design Parameters

Soil parameters for the design including unit weight and shear strength (c and φ) may be set up as shown in Figure 10.2 based on a boring investigation.

(b) External Force

[Dead Load of Concrete]

Unit Weight of Concrete: 23.5 kN/m^3 (Design Strength σ_{ck} = 16 N/mm^2)

[Earth Pressure]

The following Coulomb's earth pressure formula was adapted;

$$P_A = \frac{1}{2} K_A \cdot \gamma \cdot H^2$$

$$K_A = \frac{\cos^2(\phi - \alpha)}{\cos^2\alpha \cdot \cos(\alpha + \delta)\left(1 + \sqrt{\dfrac{\sin(\phi + \delta)\cdot \sin(\phi + \beta)}{\cos(\alpha + \delta)\cdot \cos(\alpha - \delta)}}\right)^2}$$

where, P_A =Active earth pressure (kN/m)

K_A =Coefficient of active earth pressure

γ =Unit weight of soil for back side of wall (kN/m^3)

ϕ =Internal friction angle of soil for back side of wall (deg.)

δ =Friction angle of soil with wall surface (= 2/3 ϕ) (deg.)

H =Height of Retaining Wall (m)

α =Angle of wall as shown in Figure 10.6 (deg.)

β =Angle of slope as shown in Figure 10.6 (deg.)

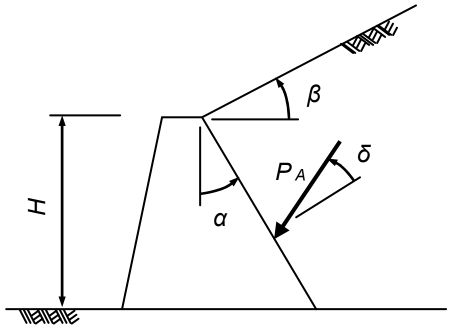


Figure 10.6 Concept of Coulomb's Active Earth Pressure

[Groundwater Pressure]

Since groundwater was not observed during the boring investigation, groundwater pressure was not considered.

[Live Loading of Road Surface]

Since the retaining wall is to be installed far enough from the carriageway, the live load on the road surface was not considered.

(b) Stability Design of Wall Body

[Tumble]

The eccentric distance of the resultant force acting on the bottom of the wall shall be within the following range;

$$|e| < \frac{B}{6} \qquad (e = \frac{B}{2} - d)$$

where, e = Eccentric distance of resultant force (m)

d = Acting distance of resultant force (m)

B = Width of foundation (m)

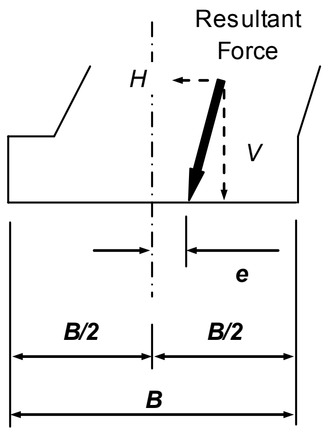


Figure 10.7 Concept of Eccentric Distance of Resultant Force

[Slip]

$$Fs = \frac{V \cdot \mu}{H} > 1.5$$

where, Fs =Safety factor against slip of wall body

V =Vertical force from the wall body at the bottom (kN/m)

μ = Coefficient of friction between bottom of concrete slab and ground (μ = 0.6 : sandy soil)

H =Lateral force at the bottom from the wall body (kN/m)

[Bearing Capacity of Ground Foundation]

The following Meyerhof's formula was adopted for calculation of the ultimate bearing capacity of the ground foundation;

$$q_a = \frac{q_a}{F}$$

$$q_d = i_c \cdot c \cdot N_c + i_q \cdot \gamma_1 \cdot D_f \cdot N_q + i_r \cdot \frac{\gamma_2 \cdot B}{2} \cdot N_\gamma$$

$$i_c = i_q = (1 - \frac{\theta}{90})^2, \quad i_\gamma = (1 - \frac{\theta}{\phi})^2$$

where, q_a =Allowable bearing capacity (kN/m^2)

F = Safety Factor (= 3)

q_d = Ultimate bearing capacity (kN/m^2)

c = Cohesion of soil under bottom slab (kN/m^2)

ϕ = Internal friction angle of soil under bottom slab (deg.)

γ_1 = Unit weight of upper soil from bottom slab (kN/m^3)

γ_2 = Unit weight of lower soil from bottom slab (kN/m^3)

D_f = Depth of embedment (m)

B = Width of Foundation (m)

N_c, N_q, N_γ = Coefficient of bearing capacity

i_c, i_q, i_γ = Compensating coefficient of inclining force

θ = Inclination angle of force (deg.)

Ground reaction force under the bottom slab is calculated as follows;

$$\left.\begin{matrix} q_1 \\ q_2 \end{matrix}\right\} = \frac{V}{B}(1 \pm \frac{6e}{B})$$

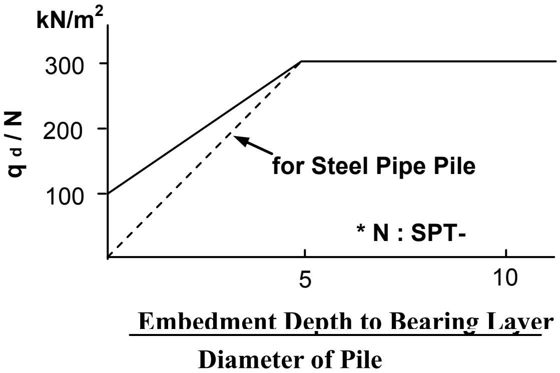
where, q_1, q_2 = Ground reaction force at edges of bottom slab (kN/m^2)

[Bearing Capacity of Pile Foundation]

Bearing capacity of the pile foundation is calculated based on "Specifications for Highway Bridges" published by Japan Road Association" as follows;

$$R_a = \frac{R_u}{3}$$

$$R_u = q_d \cdot A + U \cdot \sum (L_i \cdot f_i)$$



Source : Specifications for Highway Bridges (March 2002), Japan Road Association

Figure 10.8 Bearing Capacity on Pile Toe

where, R_a = Allowable bearing capacity of pile (kN)

R_u = Ultimate bearing capacity of pile (kN)

F = Safety factor (=3)

q_d = Intensity of bearing power on toe of pile (kN/m^2)

(In case of driven piles, Figure 10.7 may be applied.)

A = Area of toe of pile (m^2)

U = Skin length of pile (m)

L_i = Layer thickness to consider friction power (m)

f_i = Intensity of skin friction of pile (kN/m^2)

Sandy soil : f = 2 N < 100 kN/m^2 (N : SPT-value)

Cohesive soil : f = c < 150 kN/m^2 (c : cohesion of soil)

[Stress Intensity of Non-reinforced Concrete]

Stress intensities of non-reinforced concrete are calculated from the following formulas;

$$\left.\begin{array}{l}\sigma_{c1}\\ \sigma_{c2}\end{array}\right\} = \frac{N}{b \cdot h} \pm \frac{6M}{b \cdot h^2}$$

$$\tau_c = \frac{S}{b \cdot h}$$

where, σ_{c1}, σ_{c2} = Bending stress intensity of concrete (N/mm^2)

τ_c = Bending stress intensity of concrete (N/mm^2)

b = Effective width of concrete member (m)

h = *Intensity* of bearing power on toe of pile (kN/m^2)

N = Axis force to act at design section (kN)

S = Shear force to act at design section (kN)

M = Bending moment to act at design section (kN・m)

With reference to "Manual for Retaining Walls, published by Japan Road Association, March 1999", the allowable intensity for different type of stresses on non-reinforced concrete may be established as shown in Table 10.1.

Table 10.1 Allowable Intensity of Stress on Non-reinforced Concrete

Type of Stress	Allowable Intensity of Stress (N/ mm²)	Note
Compression Stress	$\frac{\sigma_{ck}}{4} \leq 5.5$	Design Strength = σ_{ck} = 16 N/mm²
Bending Tensile Stress	$\frac{\sigma_{ck}}{80} \leq 0.3$	
Shear Stress	$\frac{\sigma_{ck}}{100} + 0.15$	
Bearing Stress	$0.3\sigma_{ck} \leq 6.0$	

Source : Manual for Slope Protection, Japan Road Association (November 1999)

(4) Design Calculations

(a) External Force

The external forces to be considered in the stability calculations, as illustrated in Figure 10.9, are summarized in Table 10.2.

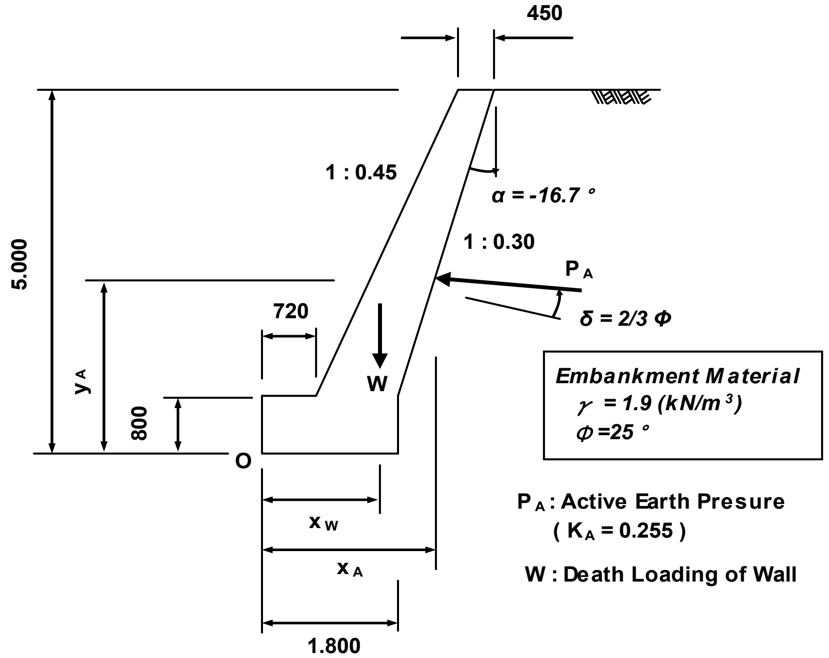


Figure 10.9 External Forces for Stability Calculations of Wall Body

Table 10.2 Summary of External Forces

Loading	Vertical Force V (kN/m)	Lateral Force H (kN/m)	Distance (m)		Moment (kN・m/m)	
			x	y	V・x	H・y
Dead Load	117.7	0.0	1.60	-	188.3	0.0
Earth Pressure	0.0	60.6	2.06	1.67	0.0	101.2
Σ	117.7	60.6	-	-	188.3	101.2

(b) Stability Calculations

The stability calculations relative to tumble, slip and bearing capacity of the wall body are summarized in Table 10.3.

Table 10.3 Summary of Stability Calculation Results

Tumble	d (m) $\left(=\frac{\Sigma(V\cdot x)-\Sigma(H\cdot y)}{V}\right)$	$B/6$ (m)	e (m)	**Judgment or Remarks**	
	0.74	0.26	0.04	$\lvert e\rvert < B/6$	-O.K.-

Slip	V (kN/m)	H (kN/m)	μ	**Judgment or Remarks**	
	117.7	60.6	0.6	Fs = 1.17 < 1.5	-OUT -

Bearing Capacity of the Ground	N_c	N_q	N_γ	θ (deg)	i_c, i_q	i_γ	
	20.72	10.66	10.88	27	0.49	0.00	
	q_d (kN/m^2)	F	q_a (kN/m^2)	q_1 (kN/m^2)	q_2 (kN/m^2)	**Judgment or Remarks**	
	129.0	3	43.0	56.7	74.1	$q_1, q_2 > q_a$	-OUT-

Slip and bearing capacity of the ground are out of the acceptable range of technical requirements, therefore, the following pile foundation was applied to the leaning concrete retaining wall from the bearing capacity calculation of the piles as shown in Table 10.4;

- Pile type : Concrete Pile φ150
- Penetration method : by Driving
- Pile arrangement : 2-lines@0.5m

Table 10.4 Summary of Bearing Capacity Calculations for the Foundation Piles

<table>
<tr><td rowspan="4">Bearing Capacity of Piles
&
Required Number</td><td>Diameter</td><td>Length (m)</td><td>N-value at Toe</td><td>$q_d \cdot A$ (kN)</td><td>N-value at skin of pile</td><td>$U \cdot \Sigma(L \cdot f)$ (kN)</td></tr>
<tr><td>φ150</td><td>2.0</td><td>N = 35</td><td>86.6</td><td>N = 10</td><td>18.8</td></tr>
<tr><td rowspan="1">R_u (kN)</td><td>F</td><td>R_a (kN)</td><td>V (kN/m)</td><td colspan="2">Number of Piles
Required | Design</td></tr>
<tr><td>105.4</td><td>3</td><td>35.1</td><td>117.7</td><td>3.4 piles/m</td><td>4 piles/m</td></tr>
</table>

Concerning stability against slip, the intensify of shear stress for concrete piles (τ_{ca} = 0.3 N/mm^2) is added to the resistance force as follows;

[Safety Factor for Slip]

$$Fs = \frac{V \cdot \mu + \tau_{ca} \cdot A}{H} = \frac{117.7\ x\ 0.6 + 300\ x\ 0.018\ x\ 4}{60.6} = 1.52 > 1.5 \quad \text{-O.K.-}$$

(b) Circular Sliding Analysis for Slope Countermeasure

The stability concerning circular sliding for the slope countermeasure is satisfied with the design requirement of more than Fs=1.20 as shown in Figure 10.10.

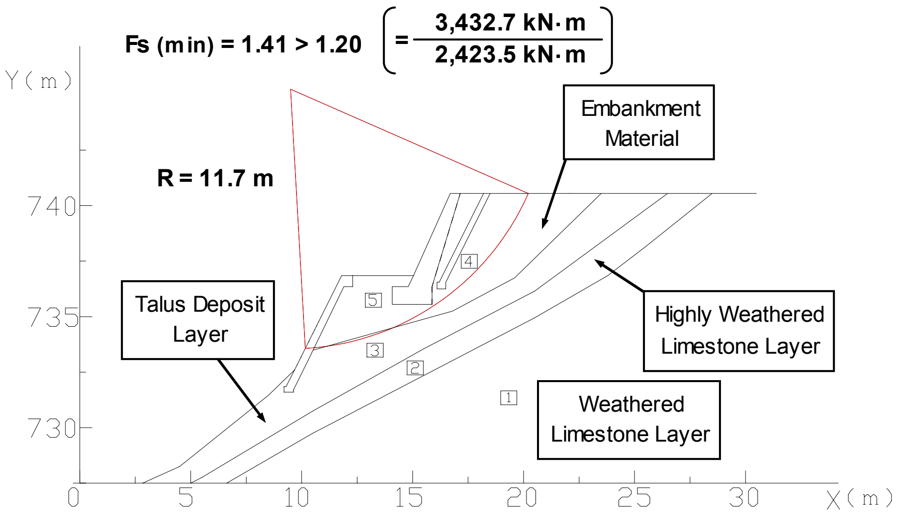


Figure 10.10 Circular Slip Stability Analysis for Slope Countermeasure

(d) Intensity of Concrete Stress

Examinations of the intensity of the stress for leaning concrete were conducted on the front wall (at Section I-I in Figure 10.11) and the toe slab (at Section II-II in Figure 10.12), and the stress intensities were calculated to be within the required range as summarized in Table 10.5.

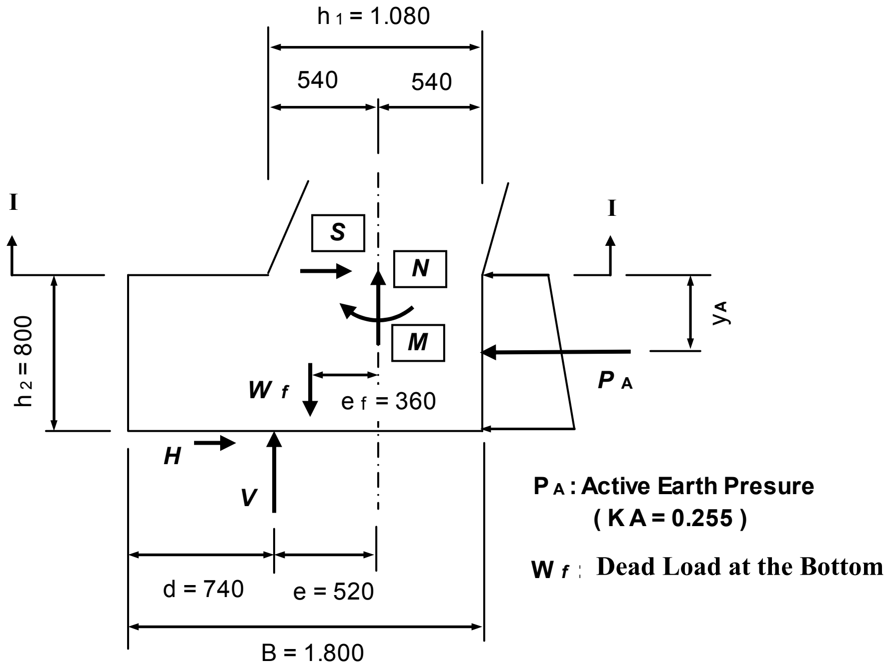


Figure 10.11 Section Force for Concrete Stress Calculation on Front Wall

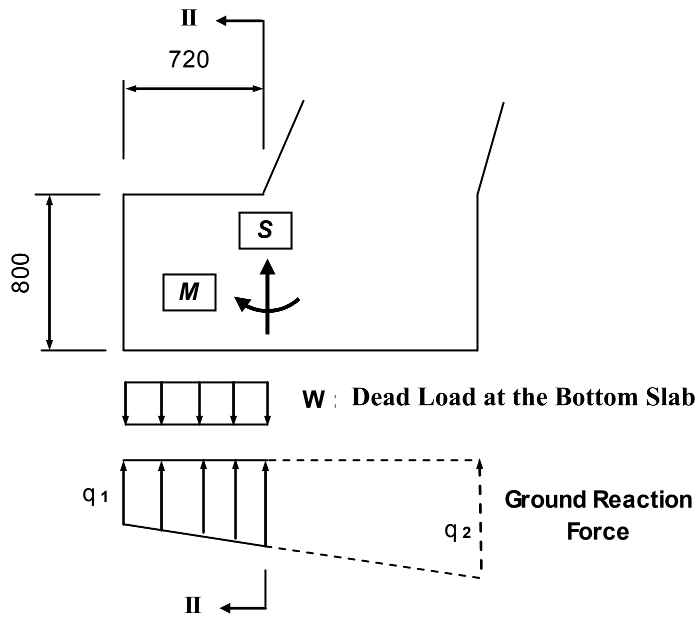


Figure 10.12 Section Force for Concrete Stress Calculation on Toe Slab

Table 10.5 Summary of Concrete Stress Intensity Calculation Results

a) Front Wall at Section I-I in Figure 10.11

Section Force	N (kN/m) ($= V - W_f$)	S (kN/m) ($= H - P_A$)	M (kN·m/m) ($= V \cdot e + P_A \cdot y - H \cdot h_2 - W_f \cdot e_f$)
	83.9	42.8	7.9

Stress Intensity	Bending Stress(N/mm^2)				Shear Stress(N/mm^2)		
	σ_{c1}	σ_{c2}	σ_{ca}	**Judgment**	τ_c	τ_{ca}	**Judgment**
	0.09	0.10	4.00	$\sigma_{c1}, \sigma_{c2} < \sigma_{ca}$ -OK-	0.05	0.30	$\tau_c < \tau_{ca}$ -OK-

b) Toe Slab at Section II- II in Figure 10.12

Section Force	S (kN/m) ($= \frac{l}{2}(p_1 + p_2)$)	M (kN·m/m) ($= \frac{l^2}{6}(2p_1 + p_2)$)	
	29.4	10.03	

Stress Intensity	Bending Stress(N/mm^2)					Shear Stress(N/mm^2)		
	σ_{c1}	σ_{c2}	σ_{ca}	σ_{ta}	**Judgment**	τ_c	τ_{ca}	**Judgment**
	0.10	-0.10	4.00	0.26	$\sigma_{c1} < \sigma_{ca}$ -OK- $\sigma_{c2} < \sigma_{ta}$ -OK-	0.04	0.30	$\tau_c < \tau_{ca}$ -OK-

10.2.2 Road Slips-Kennon Road Km 232 (Region CAR)

(1) Existing Conditions on the Site and Countermeasure Planning

This failure was classified as a Road Slip (RS), and half of the carriageway lane for approximately 70 m in length, had been closed to traffic as shown in Photo 10.1. As per information from the DPWH District Engineering Office, the original road alignment was along the valley side. After it slipped down, the road was re-aligned to the mountain side and this is now the existing alignment.

(a) Front View

(b) Side View

Photo 10.1 Conditions of the Collapsed Road Shoulder

Based on the topographic features of the site, the road slip incident is assumed to have occurred due to the following mechanisms;

- The surface water from higher on the mountain converged as it approached the road slip section as shown in Photo 10.2.
- The groundwater increased and concentrated at the slip section.
- Since the collapsed grouted riprap had not been provided with sufficient weep holes or drain filter when the back filling was done, pore water pressure was generated behind the riprap.

Photo 10.2 Surface Water Flow Route from Upslope

- Also, the embedment of the riprap wall foundation did not meet the requirements and was not able to withstand the increasing ground water pressure. These factors caused the collapse of the retaining wall and consequently, the road slip occurred.

The following viewpoints were taken into consideration in the countermeasure planning;

- Since residential houses are located on the mountain behind the slip and along the road on the Baguio City side, the negative impact of resettlement and land acquisition to the said residents should be minimized.
- Kennon Road is one of the major trunk roads in CAR Region and functions as an important tourism route, therefore, a continuous smooth flow of traffic should be maintained even during the restoration work.
- The foundation of the new retaining wall should be placed on the andesite rock layer to ensure the stability of the wall.
- Since the new retaining wall will stand on a very steep cliff, a vertical wall or a high gradient wall should be planned/desighed.
- The road should have a carriageway width of 6.2 m and road shoulders of 1.5 m on each side for a total width of 9.2 m.

(2) Structure to be Applied and Outline of the Design

(a) Structure to be Applied

The structure to be applied as the countermeasure for the site is a Reinforced Embankment (Terre Armee) as illustrated in Figure 10.13, Figure 10.14 and Figure 10.15, and as summarized below;

Wall Type	: Reinforced Embankment (Terre Armee)
Height of Wall	: H = 1.5 to 7.5 m
Gradient of Wall	: Vertical Wall (1:0)
Foundation Type	: Concrete Gravity Wall (H = 2 m)

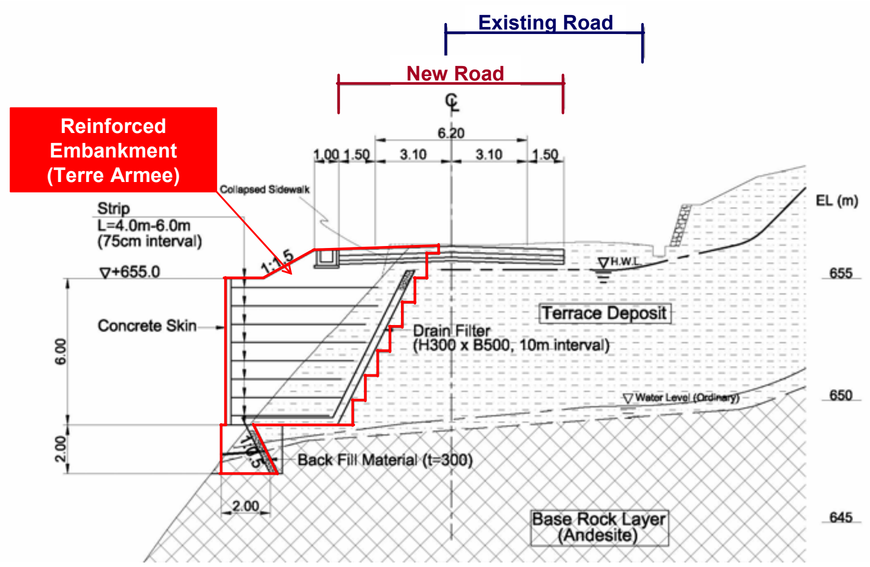


Figure 10.13 Typical Cross Section of Reinforced Embankment (Terre Armee)

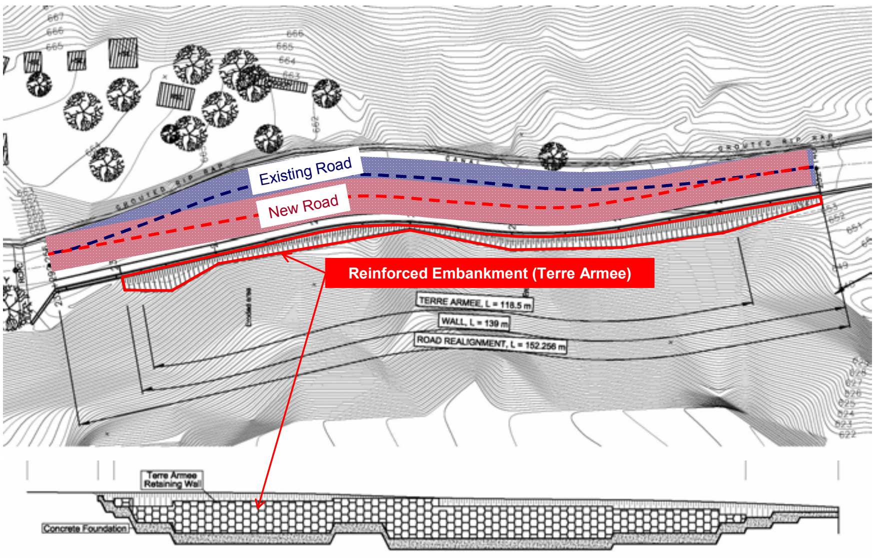
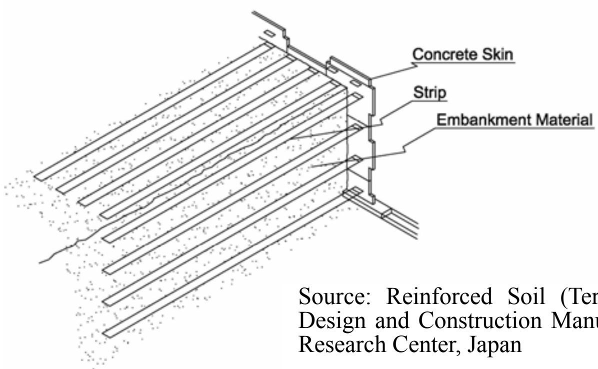


Figure 10.14 Plan and Side View of Reinforced Embankment (Terre Armee)



Source: Reinforced Soil (Terre Armee) Wall, Design and Construction Manual, Public Works Research Center, Japan

Figure 10.15 Conceptual Illustration of Reinforced Embankment (Terre Armee)

(b) Outline of the Design

In the general procedure of the design of a reinforced embankment (Terre Armee), the following four (4) items are examined.

1) Displacement of the Embankment due to the slippage of the Strips
2) Breaking of the Strips and the Bolts
3) Stability against sliding of the Embankment
4) Settlement of the ground foundation of the Embankment

The descriptions in the next sub-section focus only on items 1) and 2) above, since items 3) and 4) are common check points in the general design of retaining walls and they are shown in other section of this chapter (see calculation procedure for the retaining wall for Dalton Pass).

(3) Design Procedure for Reinforced Embankment (Terre Armee)

Figure 10.16 shows the design procedure for a reinforced embankment (Terre Armee).

Start

↓

(a) Setting of the Soil Parameters for the Embankment Material

↓

(b) Setting of the Design Live Load

↓

(c) Setting of the Allowable Stresses and Safety Factors

↓

(d) Setting of the Margin for Corrosion in the Strips

↓

(e) Setting of the Shape and Size of the Structure

↓

(f Calculation of the Active Earth Pressure Area

↓

(g) Calculation of the Coefficient of Active Earth Pressure and Friction Coefficient

↓

(h) Calculation of the Loads

↓

(i) Calculation of the Earth Pressure

↓

(j) Determination of the Horizontal Arrangement of the Strips

↓

(k) Calculation of the Horizontal Force acting on the Strips

↓

(l) Calculation of the Necessary Strip Length

↓

(m) Determination of the Strip Length

↓

End

Figure 10.16 Design Procedure for a Reinforced Embankment (Terre Armee)

(a) Setting of the Soil Parameters for the Embankment Material

Establish the parameters for the soil that can be used as the embankment material at the site. It is mentioned in "Reinforced Soil (Terre Armee) Wall, Design and Construction Manual, Civil Research Center Japan", that it is desirable to use the following two kinds of soils for embankment material:

1) Soil material of which the fine fraction content is less than 25%

2) Rock material that can be well compacted and that does not contain stones of a size over 250mm and of which the fine fraction content is less than 25% among the contents that pass a 75mm sieve

In this case, the soil parameters for the embankment material are set as follows:

Unit Weight γ = 18.6 kN/m^3

Internal Friction Angle ϕ = 35 °

(b) Setting of the Design Live Load

With reference to "Reinforced Soil (Terre Armee) Wall, Design and Construction Manual, Public Works Research Center, Japan)", it is common to adopt 9.8 kN/m^2 for the design live load where the road is planned to be developed on a reinforced embankment.

Live Load q_L = 9.8 kN/m^2

Distribution Width B_L = 6.2 m

(c) Setting of the Allowable Stresses and Safety Factors

The following were adopted, in this case, as the allowable stresses and the safety factors.

Table 10.6 Allowable Stresses and Safety Factor

Item		Allowable Stresses & Safety Factor
Safety Factor against the slippage of the Strips		2.0
Allowable Stress	Tensile Stress of the Strips (σ_T)	13.7 kN/cm^2
	Shear Stress of the Bolts (τ_a)	8.83 kN/cm^2

Source: Reinforced Soil (Terre Armee) Wall, Design and Construction Manual, Public Works Research Center, Japan

(d) Setting of the Margin for Corrosion in the Strips

Since the strips are the main components that bear the tensile force and sustain the whole reinforced embankment, the margin for corrosion should be taken into consideration in the calculation, although the strips are ordinarily given anticorrosion treatment.

Margin for Corrosion in the Strips = c_m = 1.0 mm

(e) Setting of the Shape and Size of the Structure

The shape of the structure was set, in this case, as shown in Figure 10.17.

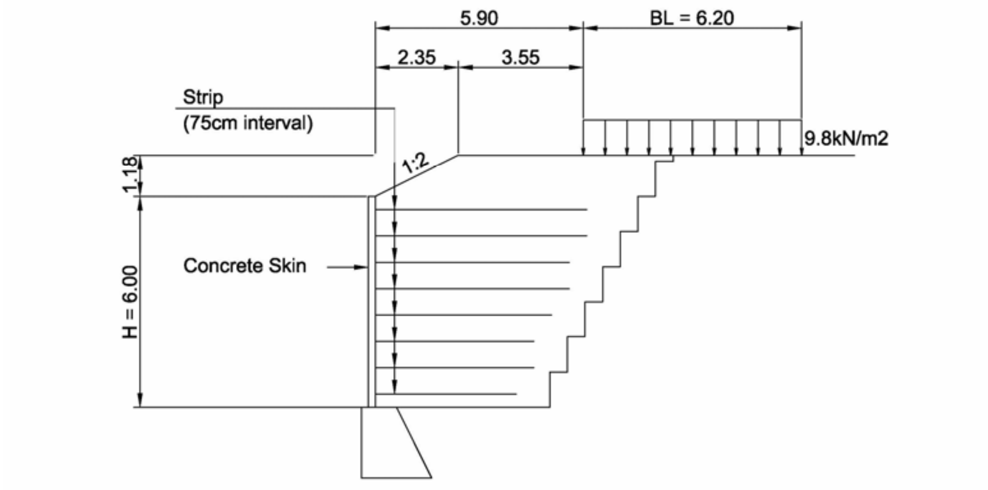


Figure 10.17 Shape of the Structure

[Virtual Wall Height]

The virtual wall height (H_a) is the simulated height of the wall that is used in the calculation. That can be given as shown in Figure 10.18 and calculated as follows.

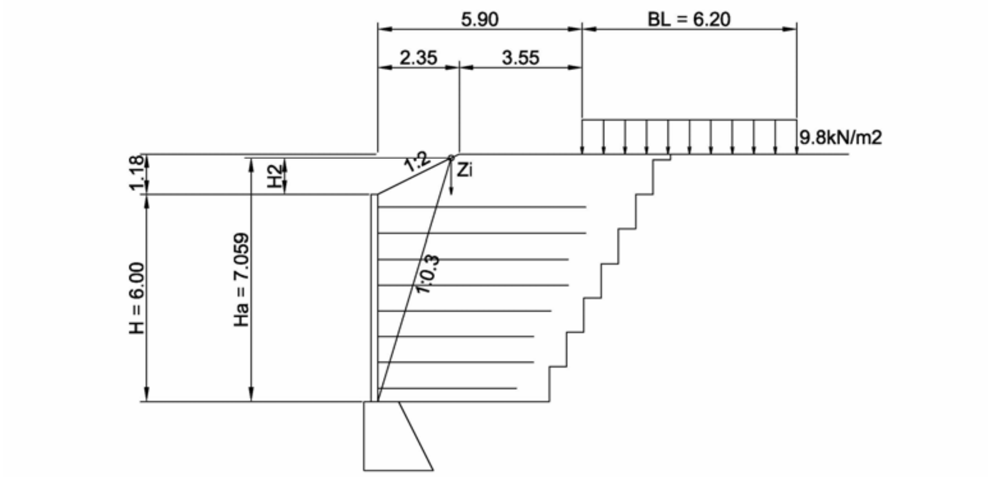


Figure 10.18 Virtual Wall Height

$H_2 = (0.3 \text{ x } H) / (n - 0.3) = (0.3 \text{ x } 6.0) / (2.0 - 0.3) = 1.059$ m

$H_a = H + H_2 = 6.0 + 1.059 = 7.059$ m

The depth of the Strips for each step, Step i, measured from the Virtual Wall Height (z_i) can be calculated and summarized as follows.

$z_i = {}_{\Delta}H \times (i - 1/2) + H_2$ (m)

Where:

${}_{\Delta}H$=Vertical Intervals of the Strips = 0.75 m

z_i	Depth (m)
1	1.434
2	2.184
3	2.934
4	3.684
5	4.434
6	5.184
7	5.934
8	6.684

(f) Calculation of the Active Earth Pressure Area

The active earth pressure area can be calculated by simplifying the shape as shown in Figure 10.19 based on the results of empirical measurements of actual structures and the results of experiments.

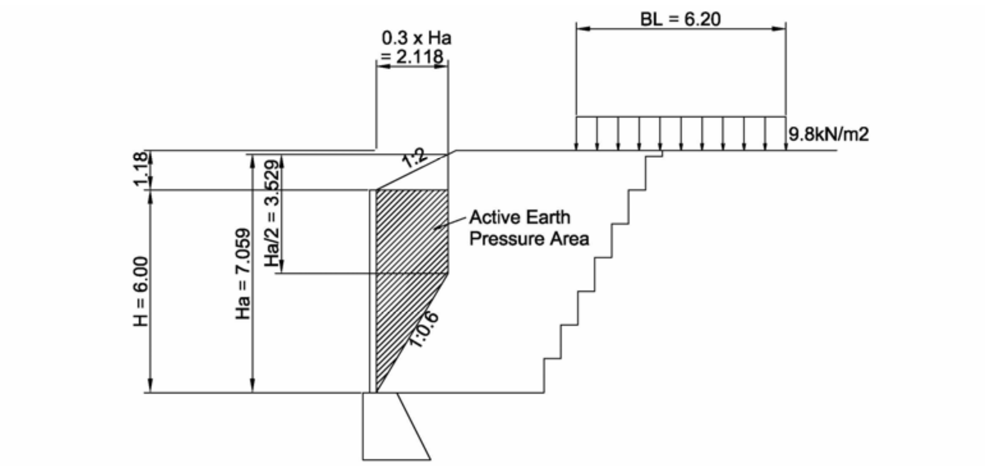


Figure 10.19 Active Earth Pressure Area

$L_{oi} = 0.3 \times H_a$ $(z_i \le H_a/2)$

$L_{oi} = 0.6 \times (H_a - z_i)$ $(z_i > H_a/2)$

i	z_i (m)	L_{oi} (m)
1	1.434	2.118
2	2.184	2.118
3	2.934	2.118
4	3.684	2.025
5	4.434	1.575
6	5.184	1.125
7	5.934	0.675
8	6.684	0.225

(g) Calculation of the Coefficient of Active Earth Pressure and Friction Coefficient

[Coefficient of Active Earth Pressure]

The coefficient of earth pressure used to calculate earth pressure can be assumed to change from the Coefficient of Earth Pressure at Rest (K_0) to the Coefficient of Active Earth Pressure (K_A) linearly from the top of the virtual wall height to 6.0m in depth, and K_A can be applied in areas deeper than 6.0m. Distribution of the Coefficient of Earth Pressure is shown in Figure 10.20.

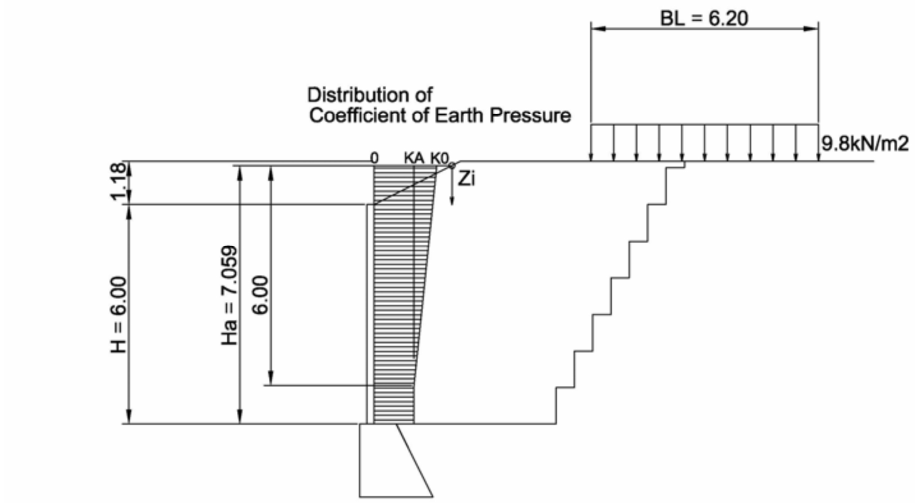


Figure 10.20 Coefficient of Earth Pressure

$K_i = K_0 \times (1 - z_i/z_0) + K_A \times z_i/z_0$ $(z_i \le z_0 = 6.0\text{m})$

$K_i = K_A$ $(z_i > z_0 = 6.0\text{m})$

Where;

K_i =Coefficient of Earth Pressure at the Strips of Step i

K_0 = Coefficient of Earth Pressure at Rest = $1 - \sin\phi$ = 0.426

K_A =Coefficient of Active Earth Pressure = $\tan^2(\pi/4 - \phi/2)$ = 0.271

z_i = Depth from the Top of the Virtual Wall Height to the Strips of Step i (m)

z_0 = Depth from the Top of the Virtual Wall Height to the Changing Point of the Coefficient of Earth Pressure (m) = 6.0 m

i	z_i (m)	$K_0 \times (1-z_i/z_0)$	$K_A \times z_i/z_0$	K_i
1	1.434	0.325	0.065	0.389
2	2.184	0.271	0.099	0.370
3	2.934	0.218	0.133	0.350
4	3.684	0.165	0.166	0.331
5	4.434	0.111	0.200	0.312
6	5.184	0.058	0.234	0.292
7	5.934	0.005	0.268	0.273
8	6.684			0.271

[Friction Coefficient]

Friction Coefficient (f_i^*) between the soil and the strips can be given as follows.

$f_i^* = f_0^* \times (1 - z_i/z_0) + \tan\psi \times z_i/z_0$ $(z_i \leq z_0 = 6.0m)$

$f_i^* = \tan\psi$ $(z_i > z_0 = 6.0m)$

Where;

f_0^* =1.5

ψ = Fiction Angle between Soil and Strips=36°

z_i = Depth from the Top of the Virtual Wall Height to the Strips of Step i (m)

z_0 = Depth from the Top of the Virtual Wall Height to the Changing Point of the Coefficient of Earth Pressure (m) = 6.0 m

i	z_i (m)	$f_0^* \times (1 - z_i/z_0)$	$\tan\psi \times z_i/z_0$	f_i^*
1	1.434	1.142	0.174	1.315
2	2.184	0.954	0.264	1.218
3	2.934	0.767	0.355	1.122
4	3.684	0.579	0.446	1.025
5	4.434	0.392	0.537	0.928
6	5.184	0.204	0.628	0.832
7	5.934	0.017	0.719	0.735
8	6.684			0.727

(h) Calculation of the Load

[Live Load]

Live Load should be considered only on the Strips that are located below the intersection point of the Virtual Boundary and the Live Load Affected Area.

In this case, since the Live Load Affected Area does not intrude into the Active Earth Pressure Area as shown in Figure 10.21, Live Load is not considered.

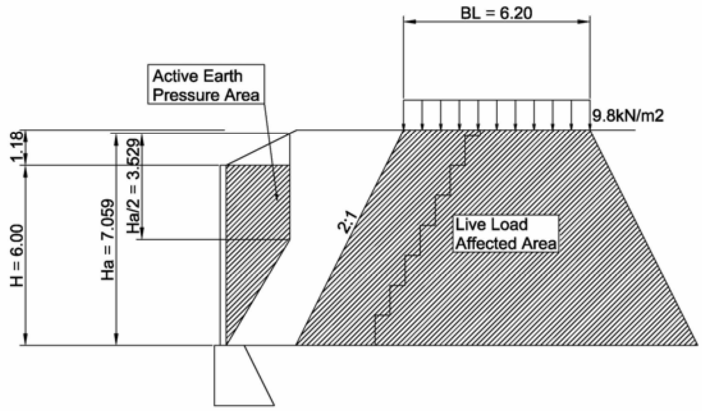


Figure 10.21 Live Load Affected Area

i	z_i (m)	L_{oi} (m)	Live Load Affected Area (m)
1	1.434	2.118	5.612
2	2.184	2.118	5.237
3	2.934	2.118	4.862
4	3.684	2.025	4.487
5	4.434	1.575	4.112
6	5.184	1.125	3.737
7	5.934	0.675	3.362
8	6.684	0.225	2.987

[Overburden Soil Load]

Overburden Soil Load can be converted into a uniformly distributed load as shown in Figure 10.22. Converted height (H_3) is the height of the overburden soil at the distance of H/2 away from the top of the wall.

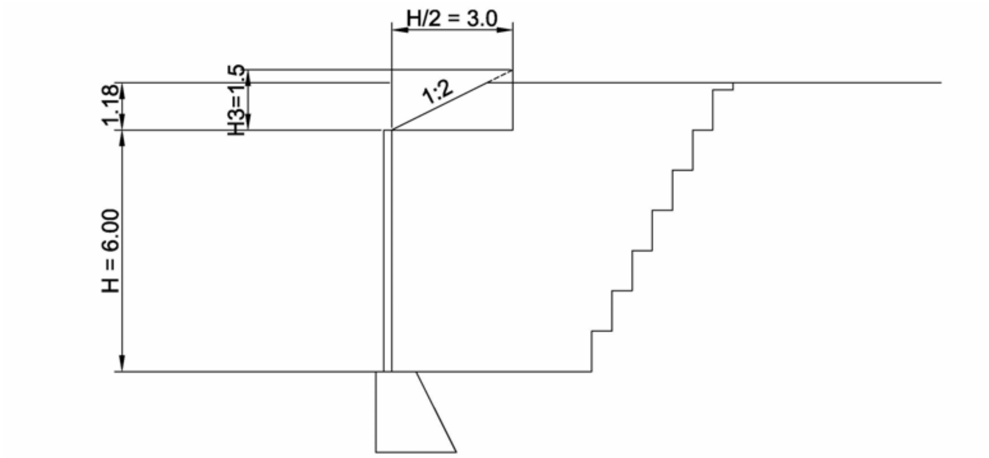


Figure 10.22 Overburden Soil Load

$q_d = \gamma \times H_3$ (kN/m^2)

Where;

$H_3 = 1/n \times (H/2) = 1.5$ m

$q_d = \gamma \times H_3 = 27.9$ kN/m^2

(i) Calculation of the Earth Pressure

$P_i = K_i \times {}_{\Delta}H \times (\gamma \times {}_{\Delta}H \times (i - 1/2) + q_d + q_{L\text{-}i})$

Where;

γ =Unit Weight of Soil = 18.6 kN/m^3

${}_{\Delta}H$=Vertical Intervals of the Strips = 0.75 m

i = The number of the Step of the Strip from the top

q_d = Overburden Soil Load = 27.9kN/m^2

q_{L-i} = Live Load = 0kN/m^2

i	K_i	γ x $_{\Delta}H$ x (i - 1/2)	$q_d + q_{L-i}$	P_i (kN/m)
1	0.389	6.975	27.900	10.182
2	0.370	20.925	27.900	13.543
3	0.350	34.875	27.900	16.498
4	0.331	48.825	27.900	19.046
5	0.312	62.775	27.900	21.187
6	0.292	76.725	27.900	22.922
7	0.273	90.675	27.900	24.250
8	0.271	104.625	27.900	26.933

(j) Determination of the Horizontal Arrangement of the Strips

[Strips]

Size 5.0 x 60.0 mm

$A_g = (t - c_m) \times b = (0.50 - 0.10) \times 6.0 = 2.40$ cm^2

$A_n = (t - c_m) \times (b - (d + 0.3)) = (0.50 - 0.10) \times (6.0 - (1.60 + 0.3)) = 1.64$ cm^2

Where;

A_g =Sectional Area of Strips after Deducting Margin for Corrosion (cm^2)

A_n =Sectional Area of Strips after Deducting Margin for Corrosion and Bolt Holes (cm^2)

$\sigma_T \times A_g$ = 32.9 kN

$\sigma_T \times A_n \times 1/0.75$ = 30.0 kN

[Bolts]

Size M16 x 40

$n \times \tau_a \times A_e \times 1/0.75 = 37.0$ kN

Where;

n =Number of Bolts at 1 a Connecting Point n = 2

A_e =Effective Sectional Area of each Bolt A_e = 1.57 cm^2

Horizontal Intervals of the Strips can be decided by using the following formula.

$\sigma_T \times A_n \times 1/0.75 < \sigma_T \times A_g < n \times \tau_{aT} \times A_e \times 1/0.75$

$P_i \times {}_{\Delta}B_i < \sigma_T \times A_g = 32.9$ kN

${}_{\Delta}B_i < 32.9/P_i$

i	P_i (kN/m)	Necessary $_\Delta B_i$ (m)	Determined $_\Delta B_i$ (m)
1	10.182	3.229	0.750
2	13.543	2.428	0.750
3	16.498	1.993	0.750
4	19.046	1.726	0.750
5	21.187	1.552	0.750
6	22.922	1.434	0.750
7	24.250	1.356	0.750
8	26.933	1.221	0.750

(k) Calculation of the Horizontal Force acting on the Strips

$T_i = P_i \times {}_\Delta B_i$ (kN)

Where;

P_i =Earth Pressure acting on the Strip at Step i

$_\Delta B_i$ = Horizontal Intervals of the Strips (m)

i	P_i (kN/m)	$_\Delta B_i$ (m)	T_i (kN)
1	10.182	0.750	7.637
2	13.543	0.750	10.158
3	16.498	0.750	12.373
4	19.046	0.750	14.284
5	21.187	0.750	15.890
6	22.922	0.750	17.191
7	24.250	0.750	18.187
8	26.933	0.750	20.200

(l) Calculation of the Necessary Strip Length

$L_{ei} = (Fs \times T_i) / (2 \times f_i^* \times \sigma_{vi} \times b)$

Where;

L_{ei} =Necessary Strip Length (m)

Fs =Safety Factor against the slippage of the Strips : 2.0

T_i =Horizontal Force acting on the Strips (kN)

f_i^* =Friction Angle of the Strip at Step i

σ_{vi} =Vertical Stress of the Soil at Step i (kN/m2)

$\sigma_{vi} = \gamma \times {}_\Delta H \times (i-1/2) + q_d + q_{Li}$ (kN/m2)

b =Width of the Strips (m)

i	T_i (kN)	f_i^*	γ x $_\Delta H$ x (i - 1/2) + q_d + q_{Li}	L_{ei} (m)
1	7.637	1.315	34.875	2.775
2	10.158	1.218	48.825	2.846
3	12.373	1.122	62.775	2.929
4	14.284	1.025	76.725	3.027
5	15.890	0.928	90.675	3.146
6	17.191	0.832	104.625	3.293
7	18.187	0.735	118.575	3.478
8	20.200	0.727	132.525	3.497

(m) Determination of the Strip Length

$L = L_{oi} + L_{ei}$ (m)

Where;

L =Necessary Strip Length (m)

L_{oi} =Strip Length within the Active Earth Pressure Area at Step i (m)

L_{ei} =Effective Strip Length at Step i (m)

i	L_{oi} (m)	L_{ei} (m)	$L_{oi} + L_{ei}$ (m)	L_i (m)	Judgment
1	2.118	2.775	4.893	6.000	OK
2	2.118	2.846	4.963	6.000	OK
3	2.118	2.929	5.046	5.500	OK
4	2.025	3.027	5.052	5.500	OK
5	1.575	3.146	4.721	5.000	OK
6	1.125	3.293	4.418	4.500	OK
7	0.675	3.478	4.153	4.500	OK
8	0.225	3.497	3.722	4.000	OK

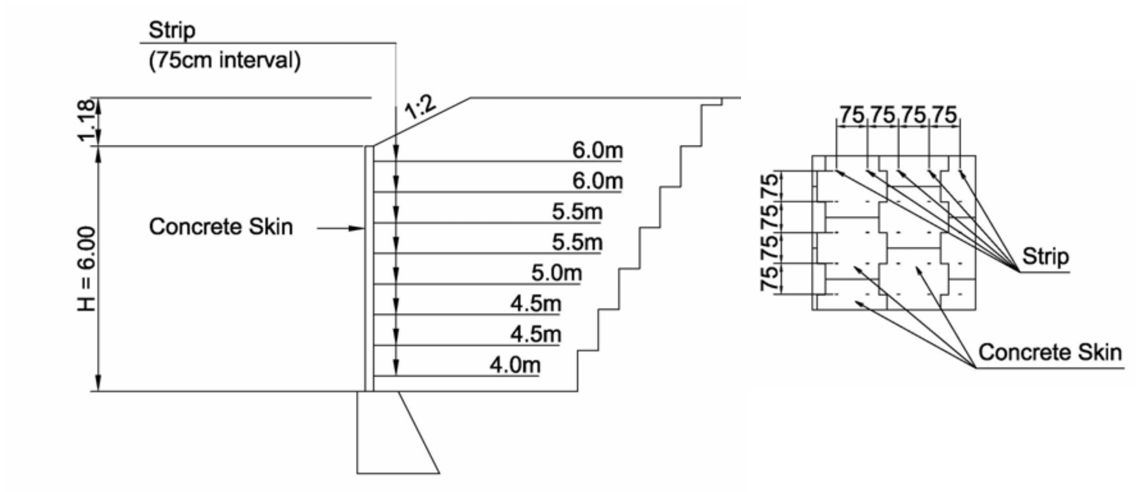


Figure 10.23 Shape of the Structure

(4) Stability Calculation for a Concrete Gravity Wall

(a) External Forces

The external forces required as input for the stability calculations are as illustrated in Figure 10.24 and are summarized in Table 10.7. And the results of the stability calculations for tumble, slip and bearing capacity are also summarized in Table 10.8.

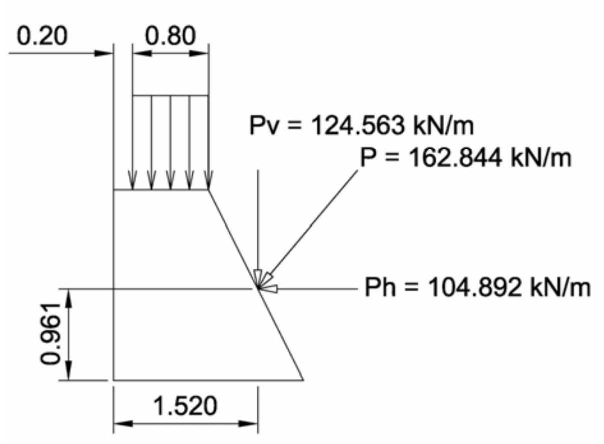


Figure 10.24 External Forces for Stability Calculations for a Gravity Wall

Table 10.7 Summary of External Forces for Stability Calculations

Loading	Vertical Force V (kN/m)	Horizontal Force H (kN/m)	Distance (m)		Moment (kN･m/m)	
			X	y	V･x	H･y
Dead Load of the Foundation	70.5	-	0.78	-	54.9	-
Dead Load of the Embankment	111.6	-	0.60	-	67.0	-
Earth Pressure	124.6	104.9	1.50	0.96	189.0	100.8
Total	306.7	104.9			311.0	100.8

Table 10.8 Stability Calculation Results

Tumble	d (m)	B/6 (m)	e (m)	Judgment
	0.67	0.33	0.31	|e| < B/6 -OK-

Slip	V (kN/m)	H (kN/m)	μ	Judgment
	306.7	104.9	0.7	Fs=2.05 >1.5 -OK-

Bearing Capacity of Ground	q (kN/m²)		qu (kN/m²)	Judgment
	297.8	8.9	300.0	q < qu -OK-