Height of consolidation (H) is usually less than 5 m. Even when an apron and a vertical wall is provided, a head of water (dH) must be less than 3.5 to 4.5 m.

5.2.15 P15: Bridge

1) P15-1: Concrete Bridge

2) P15-2: Steel Bridge

Bridges are constructed as restoration measures for the roads where existing bridges and/or their approaches have been washed out. In addition, bridges are sometimes used to avoid damages due to debris flow by passing thereover.

Description on bridge design and construction is omitted in this Manual, since other references are available.

5.2.16 P16: Foot Protection Including Apron

Refer to APPENDIX II STANDARD DRAWINGS 16. GROUTED RIPRAP APRON

Foot protection is to protect the foot portion of structure from being scoured by stream of water and apron is to protect the foot portion of structure and/or frontage ground from being scoured by dropping water.

This work is classified according to the function and the material into concrete foot protection, gabion foot protection, and grouted riprap apron.

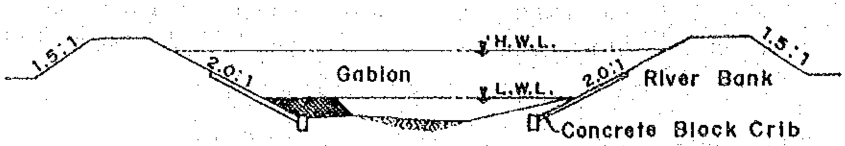
1) P16-1: Concrete Foot Protection

2) P16-2: Gabion Foot Protection

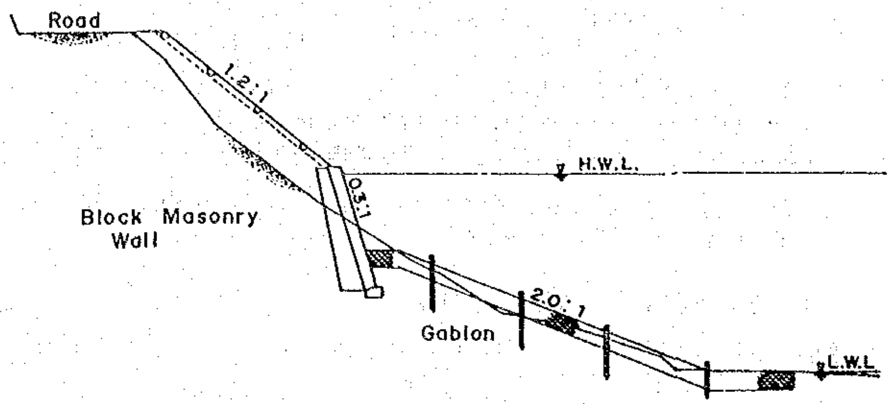
3) P16-3: Grouted Riprap Apron

Figure 5.2-84 presents two examples of gabion foot protection work, one for river bank restoration work and the other for road restoration work.

As shown in this figure, the upper level of foot protection must be higher than that of the foundation structure and generally is the same level as low water level of the river.



FOR RIVER BANK RESTORATION WORK



FOR ROAD RESTORATION WORK

FIGURE 5.2-84 GABION FOOT PROTECTION

5.2.17 P17: Spurdike

Spur is classified into two types as folows :

- longitudinal line of spurdike: parallel to the flow direction.
- transversal line of spurdike: at right or nearly right angle to the flows direction.

The function of the first type is almost same as that of river bank with low height. The second type of spurdike is constructed so as to change the flow direction and concentrate the flow into the center.

1) P17-1: Stone Spurdike

2) P17-2: Gabion Spurdike

The length, height, spacing distance of spurdike structure are determined in accordance with functional purpose, river and river bed movement, impact to up and downstream, oposite bank, strength of dike itself, etc.

The recommendable length, height and distance are :

Length : less than 10% of river width
Spacing: 2 to 4 times length
Height : above the mean low water level

Figure 5.2-85 shows an example of spurdike application.

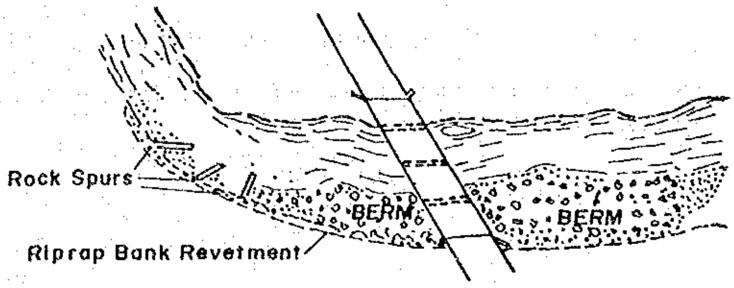


FIGURE 5.2-85 SPURDIKE APPLICATION

5.2.18 P18: Spillway

Refer to APPENDIX II STANDARD DRAWINGS 16. CONCRETE SPILLWAY

1) P18-1: Concrete Spillway

This is to place a concrete crossing the road in order to guide the concentrated water to smoothly flow down on the surface, instead of installing a culvert under the road, as see Figure 5.2-86.

A thickness of concrete plate is about 20 cm and its shape is as shown in Figure 5.2-86. The slope downstream of the spillway must be properly protected against scour.

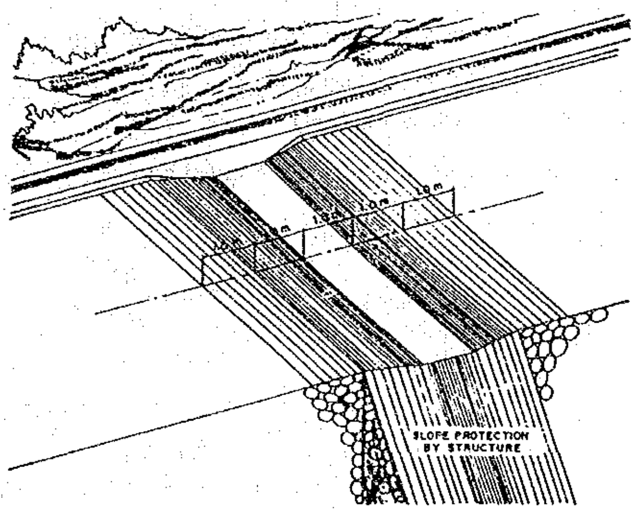


FIGURE 5.2-86 CONCRETE SPILLWAY

5.2.19 P19: Pavement Work

1) P19-1: Gravel Surfacing

2) P19-2: Bituminous Pavement

3) P19-3: Concrete Pavement

Pavement is constructed after the other restoration works are completed.

Description on pavement design and construction is omitted in this Manual, since other references are available.

5.2.20 P20: Reinforced Earth

Refer to APPENDIX II STANDARD DRAWINGS 17. REINFORCED EARTH WALL

Reinforced earth method is to increase the stability of earth structure by placing reinforcing materials therein. The increase in stability is due to frictional resistance between the earth and the reinforcing materials.

Reinforced earth method has two categories by the type of earth structure to be improved, i.e reinforcement of embankment and of natural slope.

1) P20-1: Reinforced Earth Wall

Reinforced earth wall method is to construct an embankment with a nearly vertical wall. Earth material is placed and compacted layer by layer with placement of reinforcing material each time.

This method has the following advantages :

- Constructable in narrow area
- Short construction time
- More economical than other methods in case of high wall

In the construction, backfilling work requires more careful and accurate quality control than in other methods.

Figure 5.2-87 shows a typical cross section of reinforced earth wall.

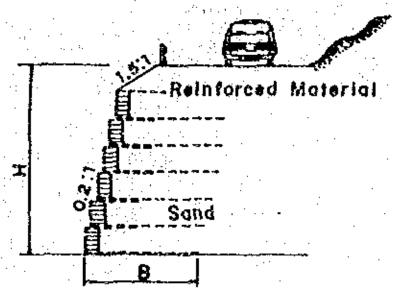


FIGURE 5.2-87 REINFORCED EARTH WALL

2) P20-2: Inserting of Reinforcing Bar

Inserting of reinforcing bar is to drive rigid bars into soil, or insert them into boreholes and then fill the gap with grout.

Figure 5.2-88 shows the typical cross sections of different types of this work.

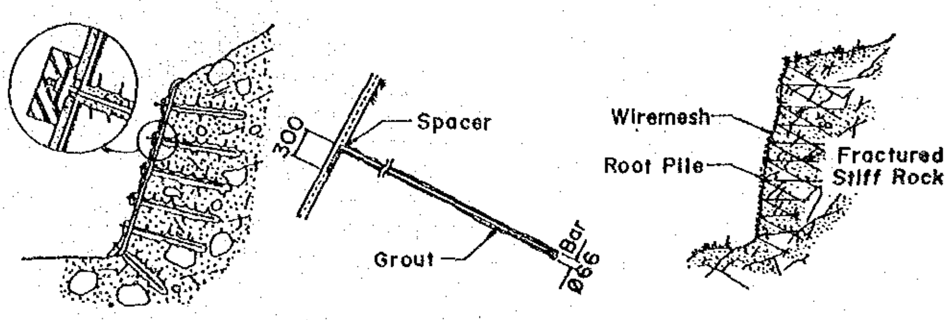


FIGURE 5.2-88 INSERTING OF REINFORCING BAR

This work can be undertaken with light construction equipment, and that at narrow and steep portion.

CHAPTER 6
SELECTION OF RESTORATION MEASURES

CHAPTER 6
SELECTION OF RESTORATION MEASURES

6.1 SELECTION OF URGENT RESTORATION MEASURES

Main purposes of urgent restoration are:

- To reopen the road to traffic,
- To remove materials endangering traffic, and
- To check the progress of damage

Therefore, urgent restoration measures should be selected depending on necessity of answering respective purpose.

The basic flow for the selection of the urgent restoration measures is shown in Figure 6.1-1.

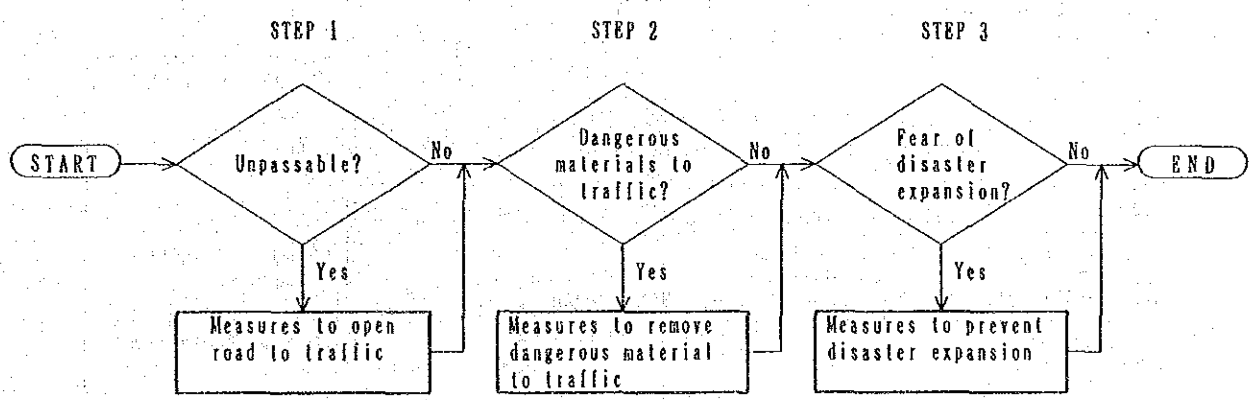


FIGURE 6-1.1 FLOW CHART FOR SELECTION OF URGENT RESTORATION MEASURES

The most important item to be achieved by the urgent restoration is to reopen the road to traffic. Therefore, in some cases, measures are selected from only step 1 neglecting step 2 and step 3.

Applicable measures corresponding to the purposes of the urgent restoration for each type of disaster are summarized in Table 6.1-1.

TABLE 6.1-1 APPLICATION OF URGENT RESTORATION MEASURES

Type of Disaster	Purposes		
	To Open Road to Traffic	To Remove Dangerous Material to Traffic	To Prevent Disaster Expansion
1. Cut Slope Failure (C-F)	U1-1 Removal of Deposit Materials	U1-2 Removal of Unstable Materials	U2 Surface Drainage U3 Slope Protection U4 Retaining Work
2. Embankment Slope Failure (E-F)	U1-4 Refilling/Embankment U4 Retaining Work	-	U2 Surface Drainage U3 Slope Protection
3. Rock Fall/ Debris Fall (FALL)	U1-1 Removal of Deposit Materials	U1-2 Removal of Unstable Materials	U2 Surface Drainage U3 Slope Protection U4 Retaining Work
4. Landslide (L-SL)	U1-1 Removal of Deposit Materials	U1-3 Removal of Head	U2 Surface Drainage U4 Retaining Work
5. Debris Flow (D-FL)	U1-1 Removal of Deposit Materials	-	-
6. Scour/Washout of Roadbed (Rd-D)	U1-4 Refilling/Embankment U4 Retaining Work	-	U2 Surface Drainage
7. Flooded/Muddy Road Surface (FM-Rd)	U1-4 Refilling/Embankment U4 Retaining Work	-	U2 Surface Drainage
8. Permanent Bridge Washout (PBr-W)	U6 Bridge	-	-
9. Permanent Br. Approach Wash-out (PBr-A)	U1-4 Refilling/Embankment U4 Retaining Work	-	U5 Foot Protection
10. Permanent Br. Other Damage (PBr-D)	-	-	U5 Foot Protection
11. Temporary Bridge Washout (TBr-W)	U6 Bridge	-	-
12. Temporary Br. Approach Washout (TBr-A)	U1-4 Refilling/Embankment U4 Retaining Work	-	U5 Foot Protection
13. Temporary Br. Other Damage (TBr-D)	-	-	U5 Foot Protection
14. Spillway Damage (SPW-D)	U1-5 Selected Material Fill U4 Retaining Work		U5 Foot Protection
15. Culvert Damage (CLV-D)	U1-4 Refilling/Embankment U4 Retaining Work		U3 Slope Protection
16. Seawall Damage (SW-D)	U1-4 Refilling/Embankment U4 Retaining Work	-	U2 Surface Drainage

6.2 SELECTION OF PERMANENT RESTORATION MEASURES

1) Cut Slope Failure (C-F)

Main measures which are generally applied to cut slope failure are:

- P1-1 Recutting
- P2 Surface Drainage
- P3 Subsurface Drainage
- P4 Slope Protection by Vegetation
- P5 Slope Protection by Structure
- P6 Retaining Wall

Figure 6.2-1 shows the general flow for the selection of restoration measures for cut slope failure.

Main points to be considered in the selection are as follows:

- Where the collapsed slope is still unstable, (P1-1) recutting until the appropriate slope gradient is recommended as the most reliable and cheap measure to stabilize the slope.
- In case that the recutting is long and high, (P6) retaining wall shall be planned at the toe of the slope in order to shorten the slope length. (P6-2) grouted riprap is recommended as a retaining wall placed at the toe of the cut slope.
- Whenever soil condition allows it, (P4) slope protection by vegetation shall be apllied to the recut slope. This is not only to prevent the slope from erosion but also to improve the surrounding environment by greening it. Slope protection by vegetation which requires no special equipment and materials such as (P4-1) hand seeding, (P4-2) hand seeding with mat, (P4-6) pick hole seeding and (P4-8) wattling are recommended, depending on topographic and soil conditions of the slope.
- For slope protection by structure to be applied to the recut slope, (P5-3) stone pitching, (P5-7) cast in- place concrete crib or (P5-5) gabion pitching is recommended because they require no special equipment and materials. Among them, (P5-5) gabion pitching is very effective to the slope with rich surface and subsurface water.
- For the slope with rich groundwater, (P3-1) subsurface drainer shall be planned at points where seepage water is concentrated.

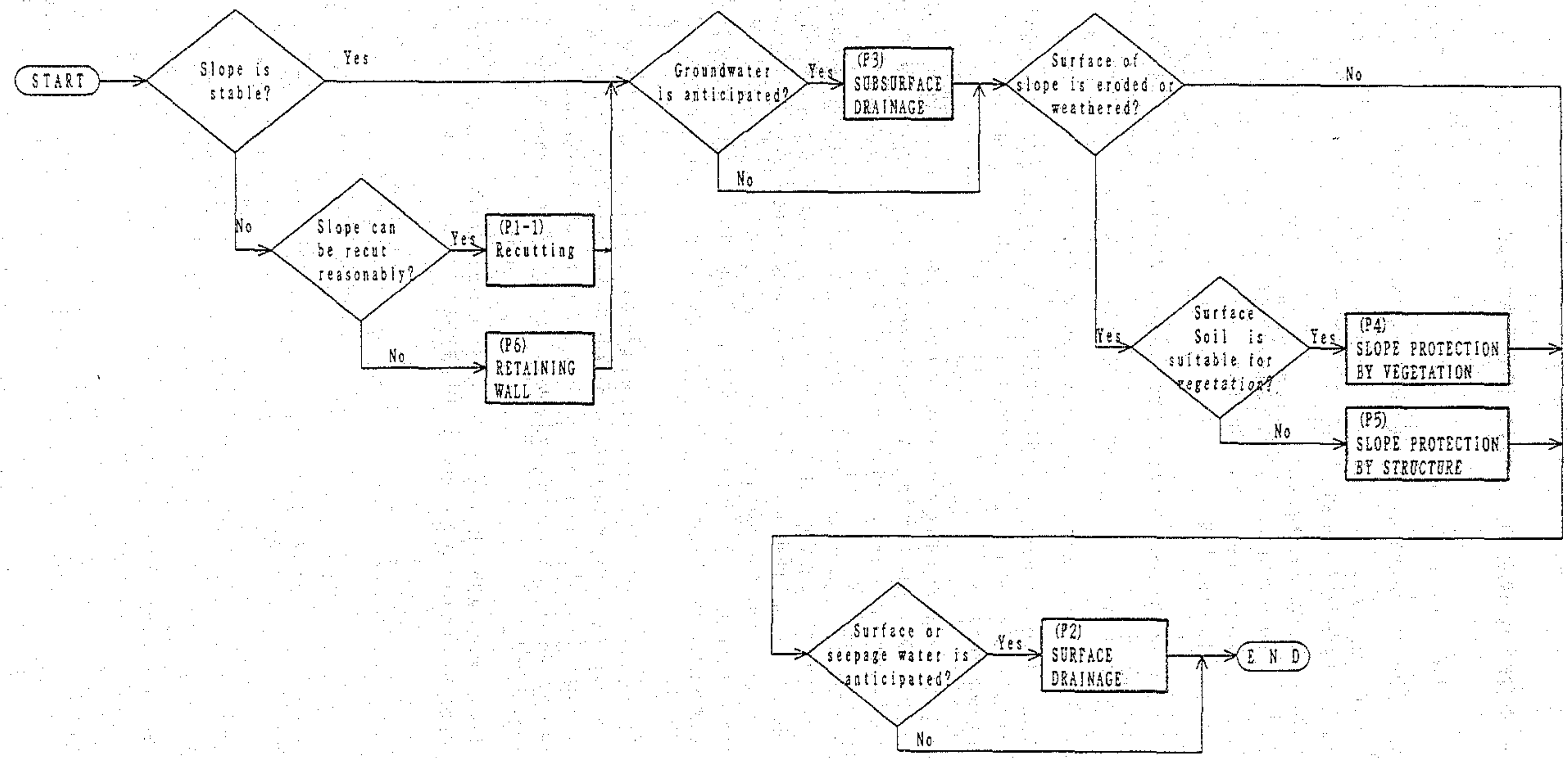


FIGURE 6.2-1 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR CUT SLOPE FAILURE (C-F)

- For a wide area of cut slope, (P2-1) slope ditch shall be planned to collect and rain surface water flowing down on the slope. Especially where surface water in surrounding area is expected to flow into the slope, top slope ditch shall be planned. As the slope ditch, grouted riprap ditch is commonly applied in Philippines.

2) Embankment Slope Failure (E-F)

Main measures which are generally applied to embankment slope failure are:

- P1-3 Refilling/Embankment
- P2 Surface drainage
- P3 Subsurface drainage
- P4 Slope protection by vegetation
- P5 Slope protection by structure
- P6 Retaining wall
- P20 Reinforced earth

Figure 6.2-2 shows the general flow for the selection of restoration measures.

Main points to be considered in the selection are as follows:

- When the groundwater is acknowledged in the failed spot and its surrounding, (P3-1) subsurface drainer is needed to release the groundwater pressure harmful to stability of embankment before taking major measures.
- Wherever possible, (P1-3) refilling/embankment is the most appropriate measure for embankment slope failure from the view points of construction speed and cost.
- When the refilled surface is susceptible to erosion, slope protection work is required. In case of the refilled slope is relatively gentle and stable by itself, (P4) slope protection by vegetation is recommendable. On the other hand, when the refilled slope is steep, (P5-3) stone pitching, (P5-5) gabion pitching, (P5-6) concrete block crib or (P5-7) cast-in-place concrete crib is suitable. Especially, (P5-5) gabion pitching is, due to its permeability, an effective measure for river or sea side embankment.
- When the refilled slope is not stable, (P6) retaining wall or (P20-1) reinforced earth wall is necessary.
- (P2-1) slope ditch is necessary to be provided on the slope surface which is of large scale and has surface or seepage water.

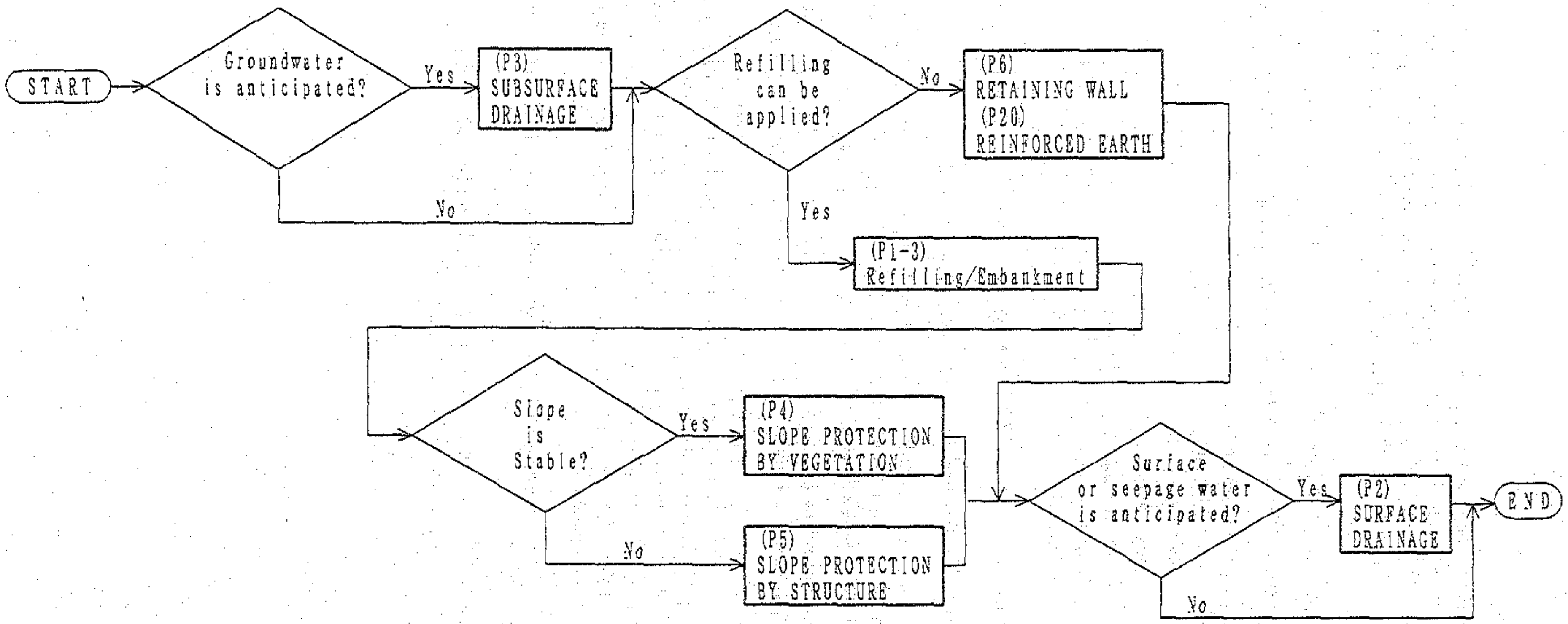


FIGURE 6.2-2 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR EMBANKMENT SLOPE FAILURE (E-F)

3) Rock Fall/Debris Fall (Fall)

Main measures which are generally applied to rock fall/debris fall are:

- P1-1 Recutting
- P1-2 Removal of Head
- P2 Surface drainage
- P4 Slope protection by vegetation
- P5 Slope protection by structure
- P8 Catch work
- P9 Supporting work
- P10 Rock shed

Figure 6.2-3 shows the general flow for the selection of restoration measures.

Main points to be considered in the selection are as follows:

– When the slope is stable and detached rocks or supportless stones still exist thereon, the prevention work against fall such as (P8-5) catch wire net, (P9) supporting work or (P1-2) removal of head is necessary.

– On the other hand, when the slope is unstable, (P1-1) recutting is selected if reasonably applicable.

– After construction of above-mentioned restoration measures, the surface of slope must be protected by slope protection works and surface drainage works. Selection procedure of these works is the same as that for cut slope failure (C-F).

– In case that recutting work is difficult to be applied, (P8) catch work or (P10) rock shed is required to protect the road from rock fall/debris fall. Among catch work, (P8-1) catch fill and ditch, (P8-2) catch gabion wall, (P8-3) catch concrete wall, and (P8-4) catch fence are applicable. The choice is made depending on the site condition.

4) Landslide (L-SL)

Main measures which are generally applied to landslide are:

- P1-2 Removal of head
- P1-4 Counterweight fill
- P2 Surface drainage
- P4 Slope protection by vegetation
- P6 Retaining wall
- P11 Prevention pile

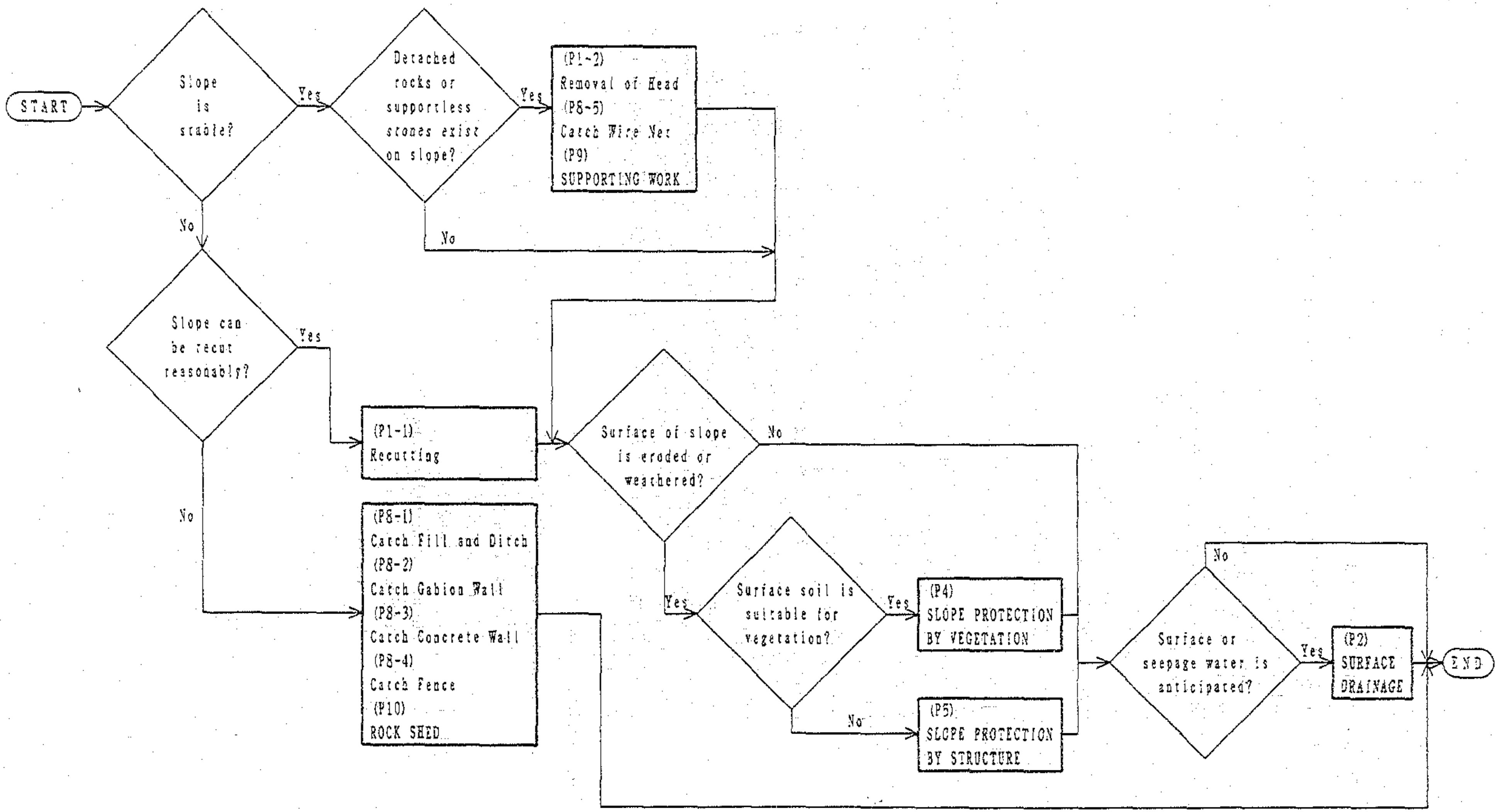


FIGURE 6.2-3 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR ROCK FALL/DEBRIS FALL (FALL)

Figure 6.2-4 shows the general flow for the selection of restoration measures.

Main points to be considered in the selection are as follows:

- When earthwork is considered to be applicable and more suitable than other methods from engineering and economic points of view, (P1-2) removal of head and/or (P1-4) counterweight fill are applied.
- In case that the earthwork is unsuitable or impossible to be applied, (P6) retaining wall or (P11) prevention pile is an effective restoration measure to stop the landslide.
- One of main causes of landslide is the excess water existing in and on the slope. Therefore, (P2) surface drainage and (P3) subsurface drainage are necessary in any case. Especially when landslide is large-scale and groundwater level is high, (P3-2) horizontal drain hole or (P3-3) deep well is necessary to drain the excess water.
- In almost all cases of landslide, there exist bare areas after the failure occured. For such bare areas, (P4) slope protection by vegetation together with (P2-3) water channel is in general required to protect the bare areas of slope from erosion, scour and weathering.

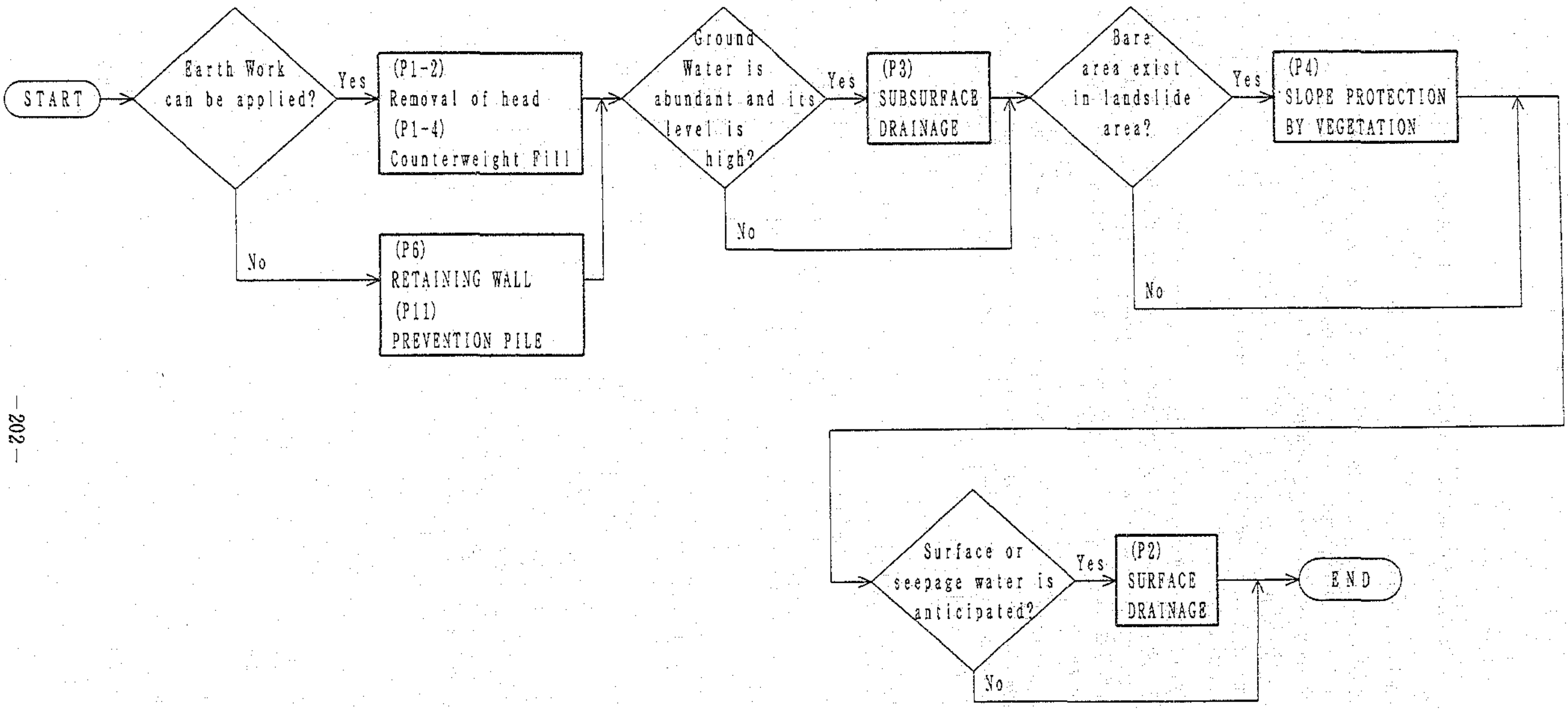


FIGURE 6.2-4 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR LANDSLIDE (L-SL)

5) Debris Flow (D-FL)

Main measures which are generally applied to debris flow are:

- P6 Retaining wall
- P13 Sabo dam
- P14 Consolidation
- P15 Bridge

Figure 6.2-5 shows the general flow for the selection of restoration measures.

Main points to be considered in the selection are as follows:

- Debris flow/mud flow generally affects a large extent of area extending over a long period. Therefore, restoration/prevention measures must be systematic. Systematic prevention works include training dike, sand deposit pond, mud reservoir, afforestation, etc. other than (P6) retaining wall, (P8) catch work, (P13) Sabo dam and (P14) consolidation.
- For debris flow on the gentle hillside. (P8-2) catch gabion wall is recommended as a simple and economical measure.
- For debris or mud flow on the river, which is usually a large- size disaster, a sabo system is necessary to be provided with (P13) Sabo dam, (P14) consolidation or a combination of the two.
- When these prevention works are judged to be inapplicable from the engineering and economic points of view, (P15) bridge is selected as a measure to avoid debris or mud flow directly attacking the road.

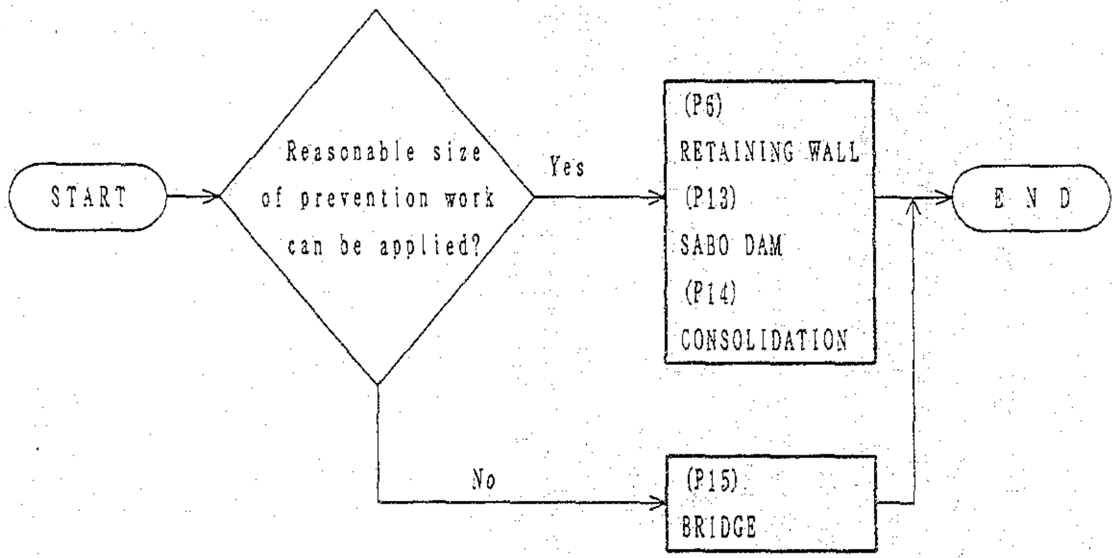


FIGURE 6.2-5 FLOW CHART FOR SELECTION OF RESTORTION MEASURES FOR DEBRIS FLOW (D-FL)

6) Scour/Washout of Roadbed (Rd-D)

Main measures which are generally applied to scour/washout of roadbed are:

- P1-3 Refilling/embankment
- P4 Slope protection by vegetation
- P5 Slope protection by structure
- P6 Retaining wall
- P16 Foot protection including apron
- P17 Spurdike

Figure 6.2-6 shows the general flow for the selection of restoration measures.

Main points to be constructed in the selection are as follows:

- At first, possibility of application of refilling/embankment to the damaged spot is technically evaluated especially on stability of earth structure. When judged to be possible, (P1-3) refilling/embankment is selected to recover the damaged portion.
- In case that the refilled earth structure is expected to be washed out or scoured again, (P5) slope protection by structure and/or (P16) foot protection including apron are required to directly protect the slope. (P17) spurdike is also applied as one of the indirect measures as necessary. (P5) slope protection by structure is used mostly together with (P16) foot protection. In such case, same materials are advisable to be used. Among others, (P5-5) gabion pitching and (P16-2) gabion foot protection are recommended because gabion is excellently permeable quickly following the change of water level.
- Where the filled earth structure is free from scour, (P4) slope protection by vegetation is recommended.
- When refilling is judged to be unapplicable, (P6) retaining wall is the only possible measure. (P6-9) gabion wall is highly recommended from the same viewpoint as previously mentioned.

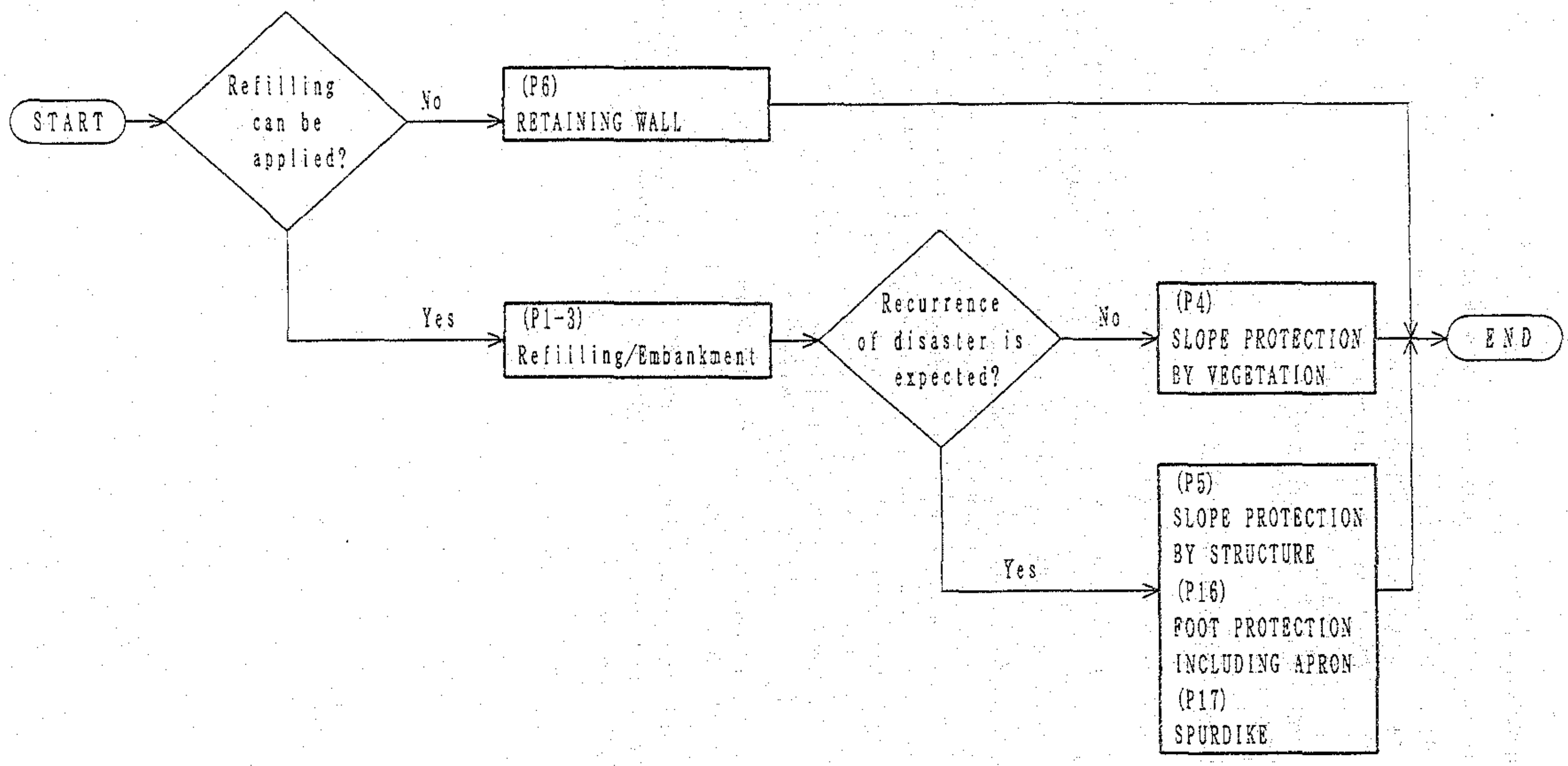


FIGURE 6.2-6 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR SCOUR/WASHOUT OF ROADBED (Rd-D)

7) Flooded/Muddly Road Surface (FM-Rd)

Main measures which are generally applied to flooded/muddy road surface are:

- P1-3 Refilling/embankment
- P2 Surface drainage
- P19 Pavement work

Figure 6.2-7 shows the general flow for the selection of restoration measures.

Main points to be considered in the selection are as follows:

- When the road suffers from flood, (P1-3) refilling/embankment is constructed to avoid it.
- (P2) surface drainage, especially (P2-2) side ditch and (P2-4) culvert are required in most cases. They should be of enough capacity and installed at proper locations.
- (P19) pavement work is also required in most cases. Usually the same type as that in neighboring section of the road is selected.

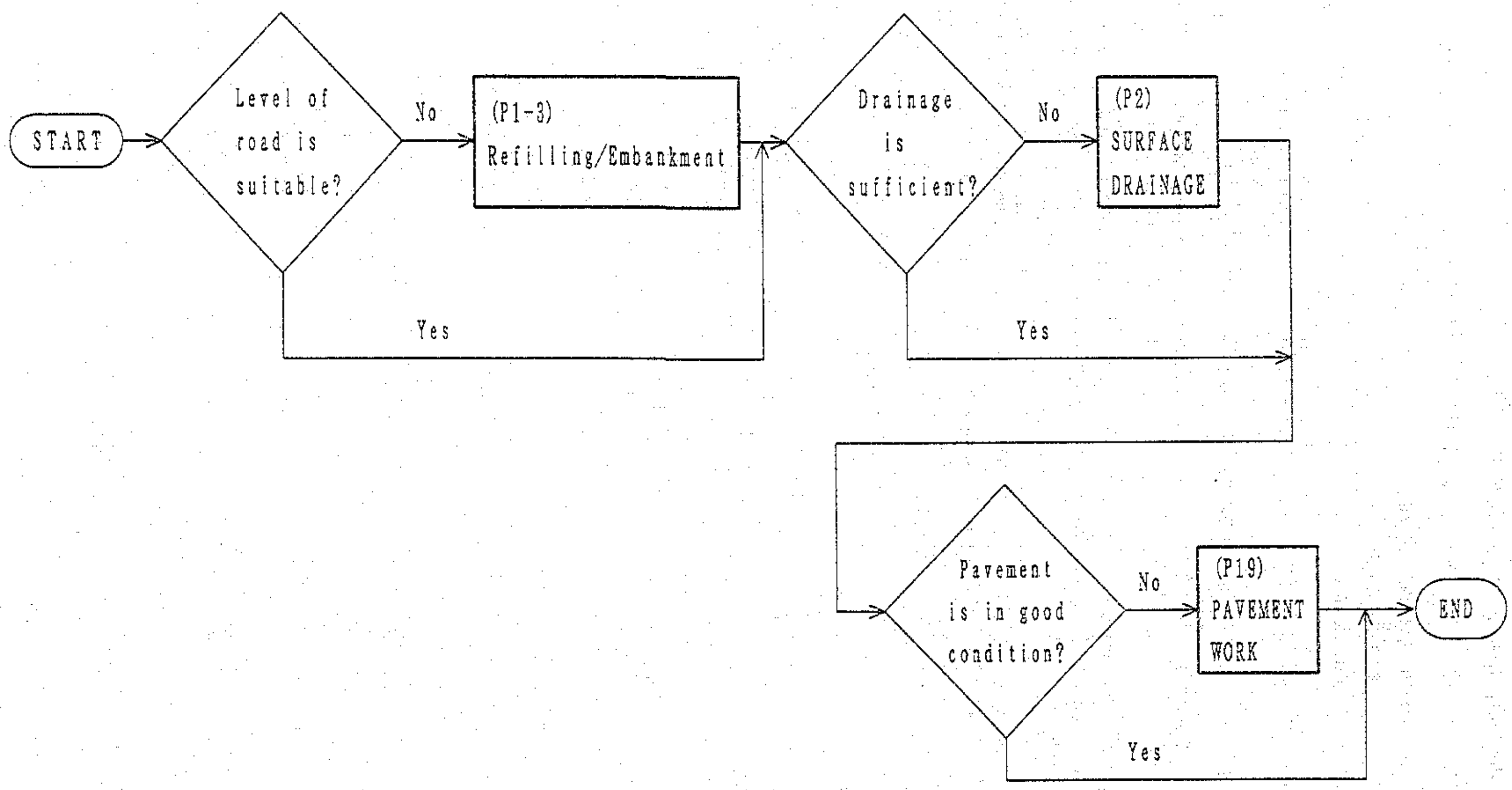


FIGURE 6.2-7 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR FLOODED/MUDDY ROAD SURFACE (FM-Rd)

8) Permanent Bridge Washout (PBr-W)

This section is applicable also to temporary bridge washout.

Figure 6.2-8 shows the general flow for the selection of restoration measures.

For bridge washout, temporary bridge should be constructed first as urgent measure to open the road to traffic urgently. The temporary bridge is replaced with (P15) bridge, if feasible judging from economic and other considerations.

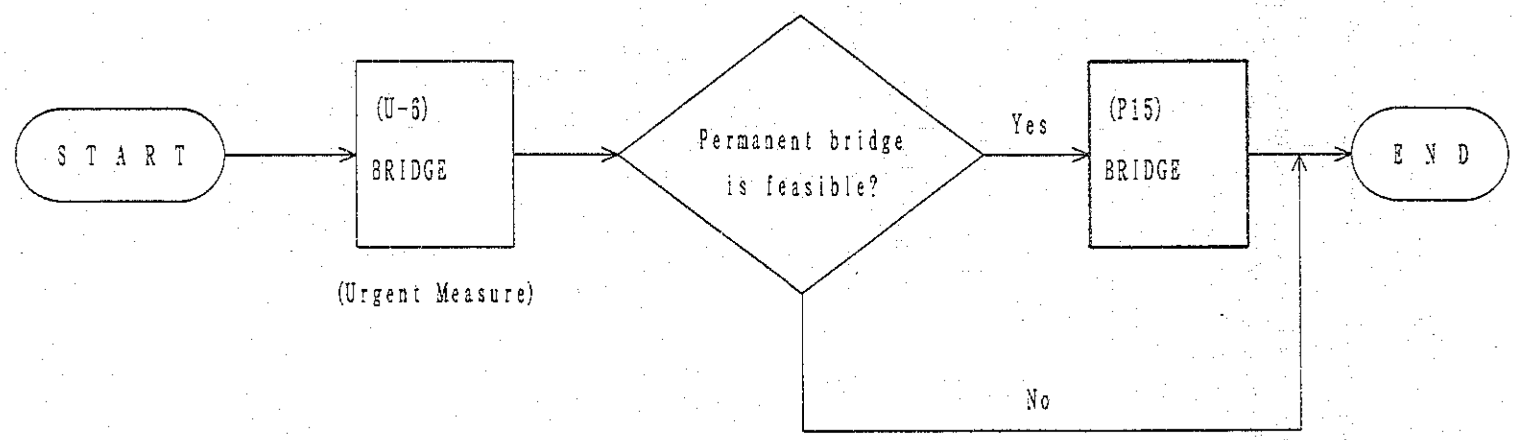


FIGURE 6.2-8 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR PERMANENT/TEMPORARY BRIDGE WASHOUT (PBr-W, TBr-W)

9) Permanent Bridge Approach Washout (PBr-A)

This section is applicable also to temporary bridge approach washout.

Figure 6.2-9 shows the general flow for the selection of restoration measures.

At first, it is judged technically whether the damaged approach can reasonably be restored with earth structure or not. If the approach has encroached or would encroach on the stream, the answer is no. In such case, the bridge should be expanded with (P15) bridge or (U6) bridge as the case may be.

If the damaged approach can reasonably be restored with earth structure, restoration measures are selected in the same way as scour/washout of roadbed. Refer to 6) of this chapter.

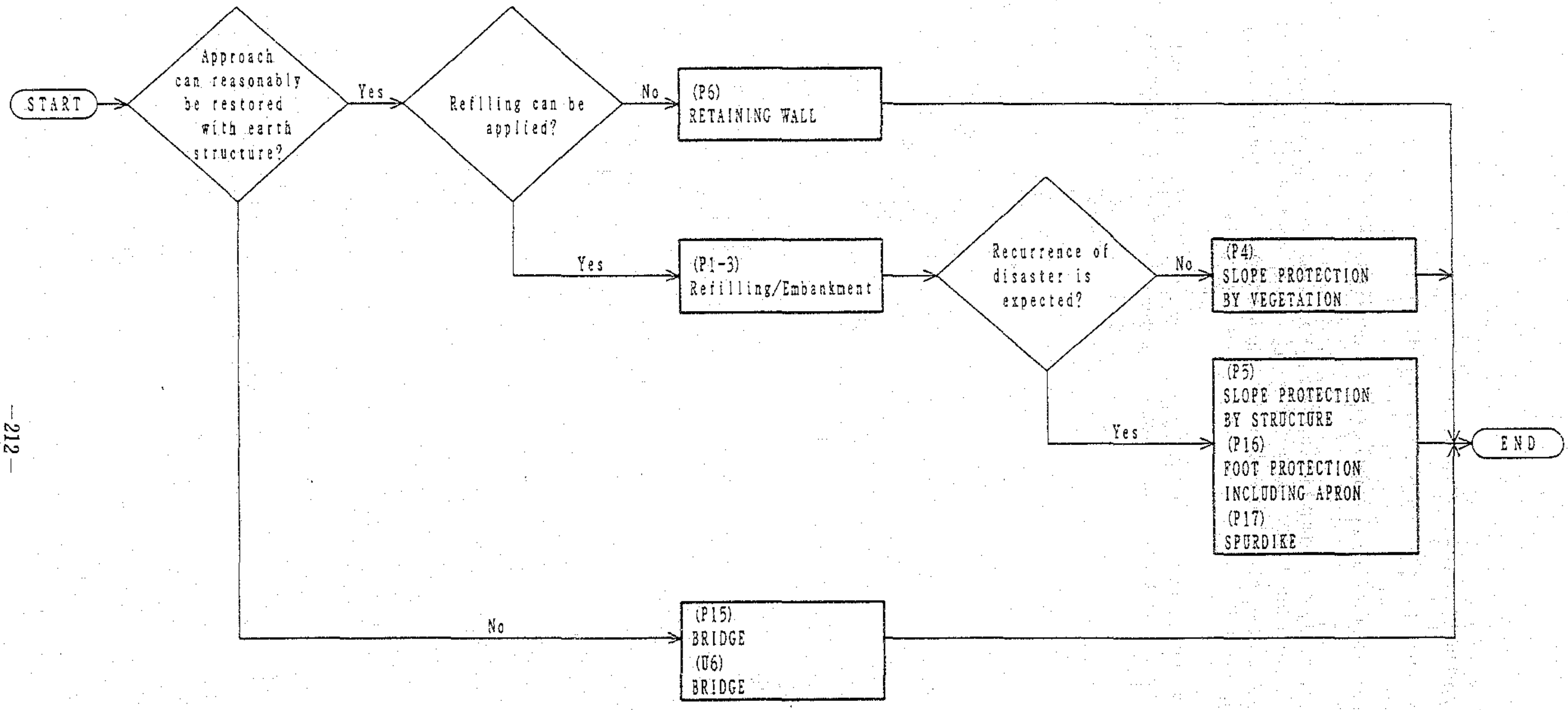


FIGURE 6.2-9 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR PERMANENT/TEMPORARY BRIDGE APPROACH WASHOUT (PBr-A, TBr-A)

10) Permanent Bridge Other Damage (PBr-D)

This section is applicable also to temporary bridge other damage.

Main measures which are generally applied to permanent/temporary bridge other damage are:

- P6 Retaining wall
- P13 Sabo dam
- P14 Consolidation
- P16-1 Concrete foot protection
- P16-2 Gabion foot protection
- P17 Spurdike

Figure 6.2-10 shows the general flow for the selection of restoration measures.

Main points to be considered in the selection are as follows:

- When pier foundation is being scoured, it should be protected with (P16-1) concrete foot protection or (P16-2) gabion foot protection. Another solution is to construct downstream (P14) consolidation aiming at aggradation of riverbed, unless danger of collapse is imminent. Both measures are sometimes combined for more effect.
- When riverbed is aggrading, it should be controlled by proper means, for example, by constructing upstream (P13) sabo dam or (P14) consolidation.
- When abutment and/or river bank are being scoured, they should be protected with (P6) retaining wall or (P16) foot protection, or both. If the direction of stream must be controlled to prevent further erosion of bank, (P17) spurdike is required.

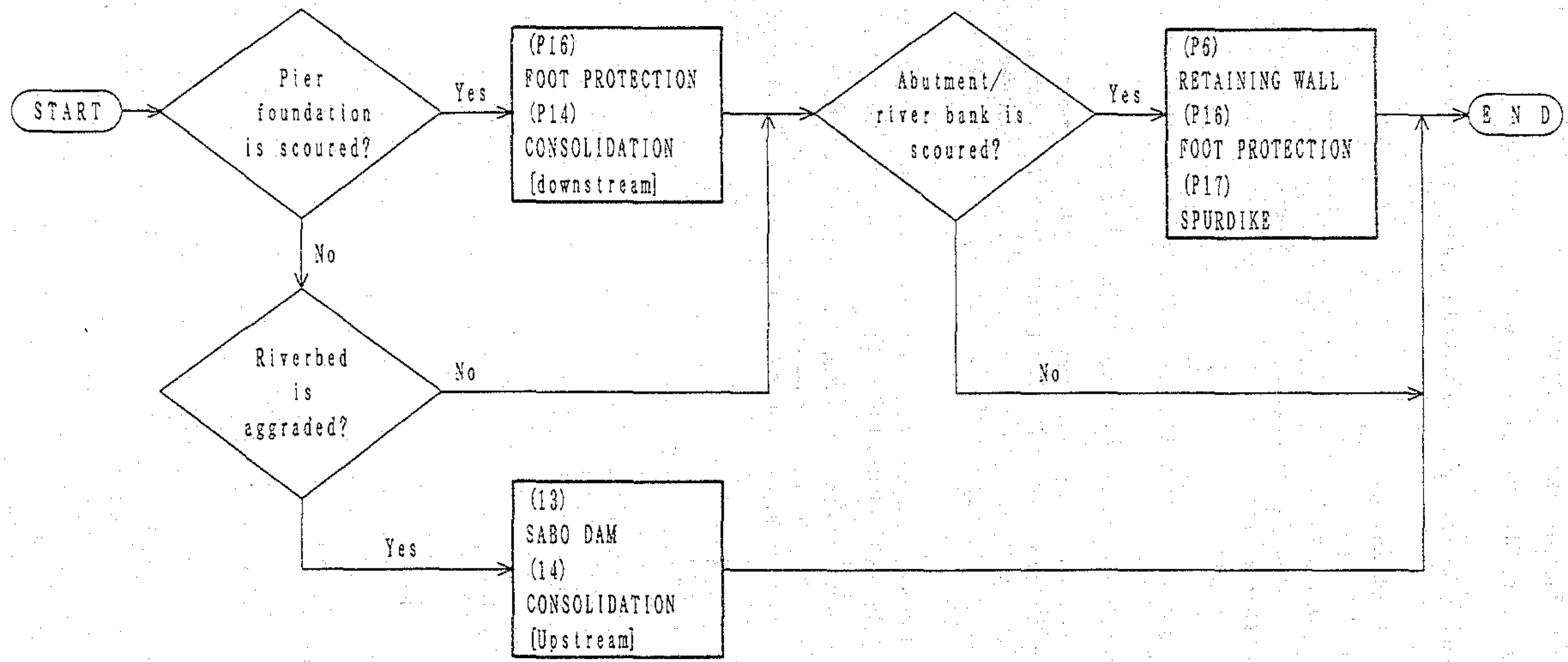


FIGURE 6.2-10 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR PERMANENT/TEMPORARY BRIDGE OTHER DAMAGE (PBr-D, TBr-D)

11) Temporary Bridge Washout (TBr-W)

Refer to 8) of this chapter.

12) Temporary Bridge Approach Washout (TBr-A)

Refer to 9) of this chapter.

13) Temporary Bridge Other Damage (TBr-D)

Refer to 10) of this chapter.

14) Spillway Damage (SPW-D)

Main measures which are generally applied to spillway damage are:

- P1-5 Selected material fill
- P2-4 Culvert
- P6 Retaining wall
- P13 Sabo dam
- P14 Consolidation

Figure 6.2-11 shows the general flow for the selection of restoration measures.

Main points to be considered in the selection are as follows:

- Spillway damage is classified into 2 types according to the cause as follows:
  - Body of spillway and/or its approach are scoured or washed out.
  - Culverts in the spillway are clogged with debris.
- In the first case, spillway should be reconstructed and or/extended. For this, the use of (P1-5) selected material fill protected by (P6) retaining wall such as (P6-6) supported type concrete wall or (P6-9) gabion wall is recommended.
- In the second case, removal of sediment is a solution. If recurrence of such sedimentation is anticipated, a fundamental preventive measure such as (P13) sabo dam or (P14) consolidation is needed.

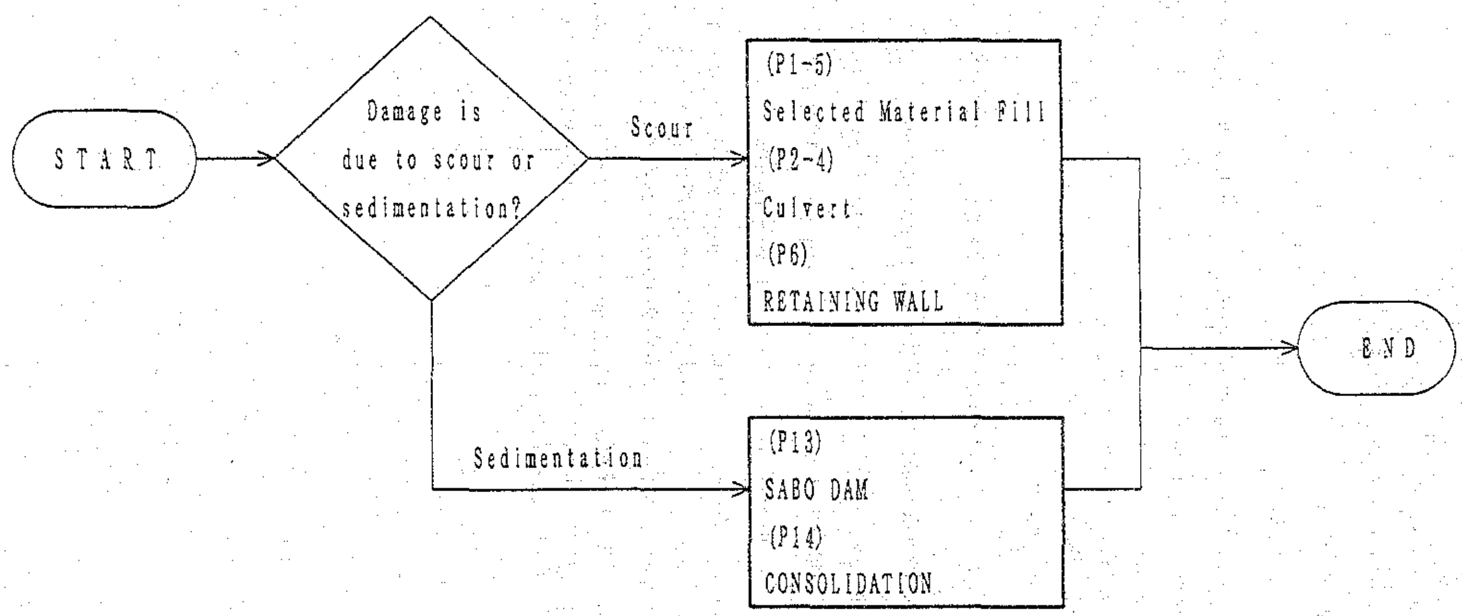


FIGURE 6.2-11. FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR SPILLWAY DAMAGE (SPW-D)

15) Culvert Damage (CLV-D)

Main measures which are generally applied to culvert damage are:

- P2-4 Culvert
- P2-5 Catch basin
- P16-3 Grouted riprap apron
- Measures for embankment slope failure

Figure 6.2-12 shows the general flow for the selection of restoration measures.

Main points to be considered in the selection are as follows:

- When the existing culvert has a defect such as insufficient capacity, damage, and insufficient length, it should be corrected with proper-sized (P2-4) culvert: by replacement or addition against insufficient capacity, replacement against damage, and extension against insufficient length.
- When culvert inlet is clogged with debris, it should be removed. The installation of (P2-5) catch basin is sometimes effective for debris problem. When the portion near the culvert outlet is subjected to scour, proper protection such as (P16-3) grouted riprap apron is required.
- When embankment slope is damaged, it should be restored. See 2) of this chapter.

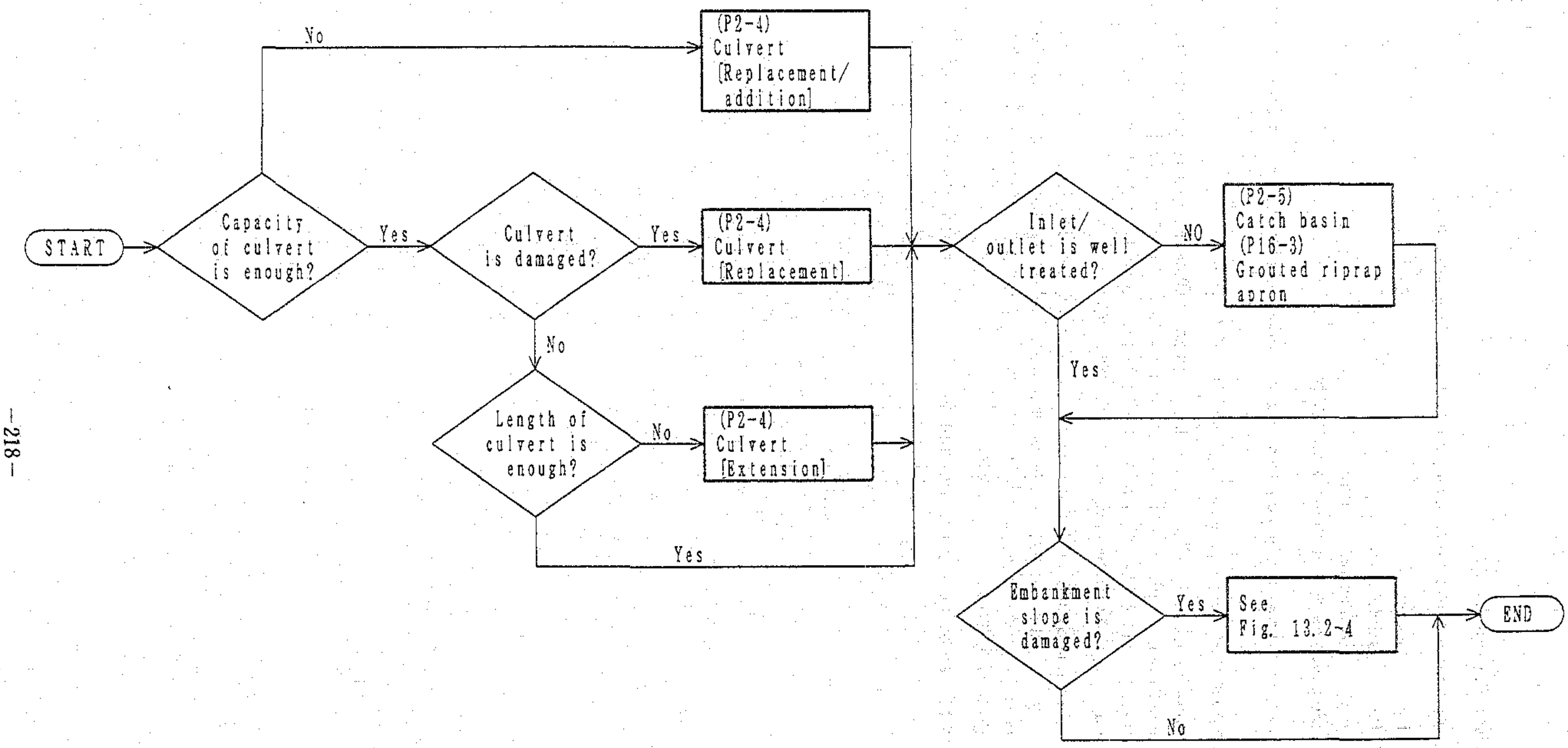


FIGURE 6.2-12 FLOW CHART FOR SELECTION OF RESTORATION MEASURES FOR CULVERT DAMAGE (CLV-D)

16) Seawall Damage (SW-D)

Permanent restoration measure for seawall damage is mainly (P6) retaining wall. Proper type of retaining wall should be selected in consideration of height, soil property, construction conditions, etc.

APPENDIX I
ANALYSIS METHODS

APPENDIX I
ANALYSIS METHODS

TABLE OF CONTENTS

PAGE

1. SLOPE STABILITY

1.1 GENERAL

The calculation method and an example presenting in this chapter can be applied mainly to estimating slope stability for embankment structure.

Table 1-1 indicates suitable combination of embankment height and slope gradient by different type of embankment materials, being based on empirical experiences, for the study of slope stability.

TABLE 1-1 SUITABLE EMBANKMENT AND SLOPE GRADIENT BY MATERIALS

Embankment Materials	Height	Slope Gradient
Well grained sand, gravel	> 5 m 5 m > 15 m	1.5:1 ~ 1.8:1 1.8:1 ~ 2.0:1
Poorly grained sand	> 10 m	1.8:1 ~ 2.0:1
Crashed Rock	> 10 m 10 m > 20 m	1.5:1 ~ 1.8:1 1.8:1 ~ 2.0:1
Sandy soil, Stiff clayey soil,	> 5 m	1.5:1 ~ 1.8:1
Stiff clay	5 m ~ 10 m	1.8:1 ~ 2.0:1
Soft Clayey Soil	> 5 m	1.8:1 ~ 2.0:1

Table 1.1 can be practically applicable for almost studies of restoration measures in embankment failure spots. In the following cases, however, estimating and evaluating slope stability will be required to proposed embankment structure including foundation ground:

- embankment materials have high water content and poor permeability, thus high pore pressure remain in the materials.
- spring water appears on the slope of foundation ground.
- foundation ground is composed of soft soil like silt, silty clay, clay, clayey soil, etc.

1.2 SOIL PROPERTIES

Physical and mechanical soil properties of embankment and foundation ground, which are required for slope stability analysis, are evaluated as shown in Table 1-2.

TABLE 1-2 SOIL PROPERTIES BY CLASSIFICATION

	Classification	Condition		Unit Weight (t/m^3)	Internal Friction Angle	Cohesion (kg/cm^2)
Embankment	Gravel, Sand with Gravel	Well Compacted		2.0	40	0
	Sand	Well Compacted	Well Grained	2.0	35	0
			Poorly Grained	1.9	30	0
	Sandy Soil	Well Compacted		1.8	25	<0.3
	Cohesive Soil	Well Compacted		1.7	15	<0.5
Natural Ground	Gravel	Dense or well grained		2.0	40	0
		Not dense or poorly grained		1.8	35	0
	Sand with Gravel	Dense		2.1	40	0
		Not dense		1.9	35	0
	Sand	Dense or well grained		2.0	35	0
		Not dense or poorly grained		1.8	30	0
	Sandy Soil	Dense		1.9	30	<0.3
		Not dense		1.7	25	0
	Cohesive Soil	Stiff		1.8	25	<0.5
		Soft		1.6	15	<0.15
	Clay, Silt	Stiff		1.6~1.7	20	<5
		Soft		1.4~1.5	10	<1.5

When the data of unconfined compression test on undisturbed soil sample or standard penetration test are obtained, the strength of soil can be evaluated as follows :

Unconfined compression test

$$C = qu/2$$

where, c: cohesion
qu: unconfirmed compressive strength

N-Value

Silty Clay	c	=	1/2 (0.1 + 0.15 N)
Clay (N < 10)	c	=	1/2 (0.2 + 0.15 N)
Cohesive Soil	c	=	1/2 (0.1 + 0.14 N)
Diluvial Clay	c	=	1/2 (N/5 - N/6)

where c: cohesion in kg/cm^2
n: N-value from SPT

1.3 CALCULATION METHOD

For the stability analysis of embankment, the slice method of circular slop surface may be applicable.

A mass on a circular arc is divided into several slices with appropriate width as shown in Figure 1-1. The shearing force and resisting force of each slice along the circular slope surface are totalled separately. Then, the factor of safety is determined by ratio of both shearing and resisting forces. Normally, the number of slices is more than 6 or 7.

There are two kinds of method, the effective stress method and the total stress method which require different types of tests. The former is generally adopted in analysis of design while the latter is used to check the stability of embankment during and just after the construction of embankment which was quickly constructed with fine-grained soil.

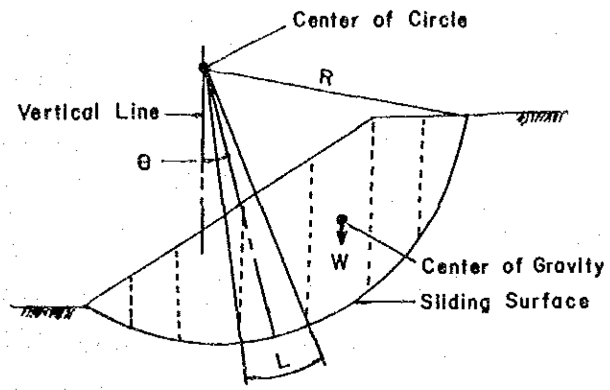


FIGURE 1-1 STABILITY CALCULATION BY SLICE METHOD OF CIRCULAR SLIP SURFACE

Calculation equations for both method are eq. (1) and eq (2).

Effective Stress Method

$$F_s = \frac{\Sigma \{c' \cdot L + (W \cdot \cos\theta - u \cdot L) \cdot \tan \phi'\}}{\Sigma W \cdot \sin\theta} \quad \text{- - - (1)}$$

Where shearing stress S is given by

$$S = c' + (\sigma - \mu) \tan \phi'$$

Total Stress Method

$$F_s = \frac{\Sigma (c \cdot L + W \cdot \cos\theta \cdot \tan \phi)}{\Sigma W \cdot \sin\theta} \quad \text{- - - - (2)}$$

Where, shearing stress is given by

$$S = c + \sigma \cdot \tan \phi$$

$$\delta = \frac{P}{L}$$

$$P = W \cdot \cos\theta$$

Where for eq. (1) and (2):

F_s : Factor of Safety

σ : Normal stress (t/m2)

P : Normal reaction acting to the bottom plane of slice (t/m)

W : Weight of slice (t/m)

L : Arc length of a slice of sliding surface (m)

c : Cohesion (t/m2)

ϕ : Internal friction angle of soil (degree)

μ : Pore water pressure (t/m2)

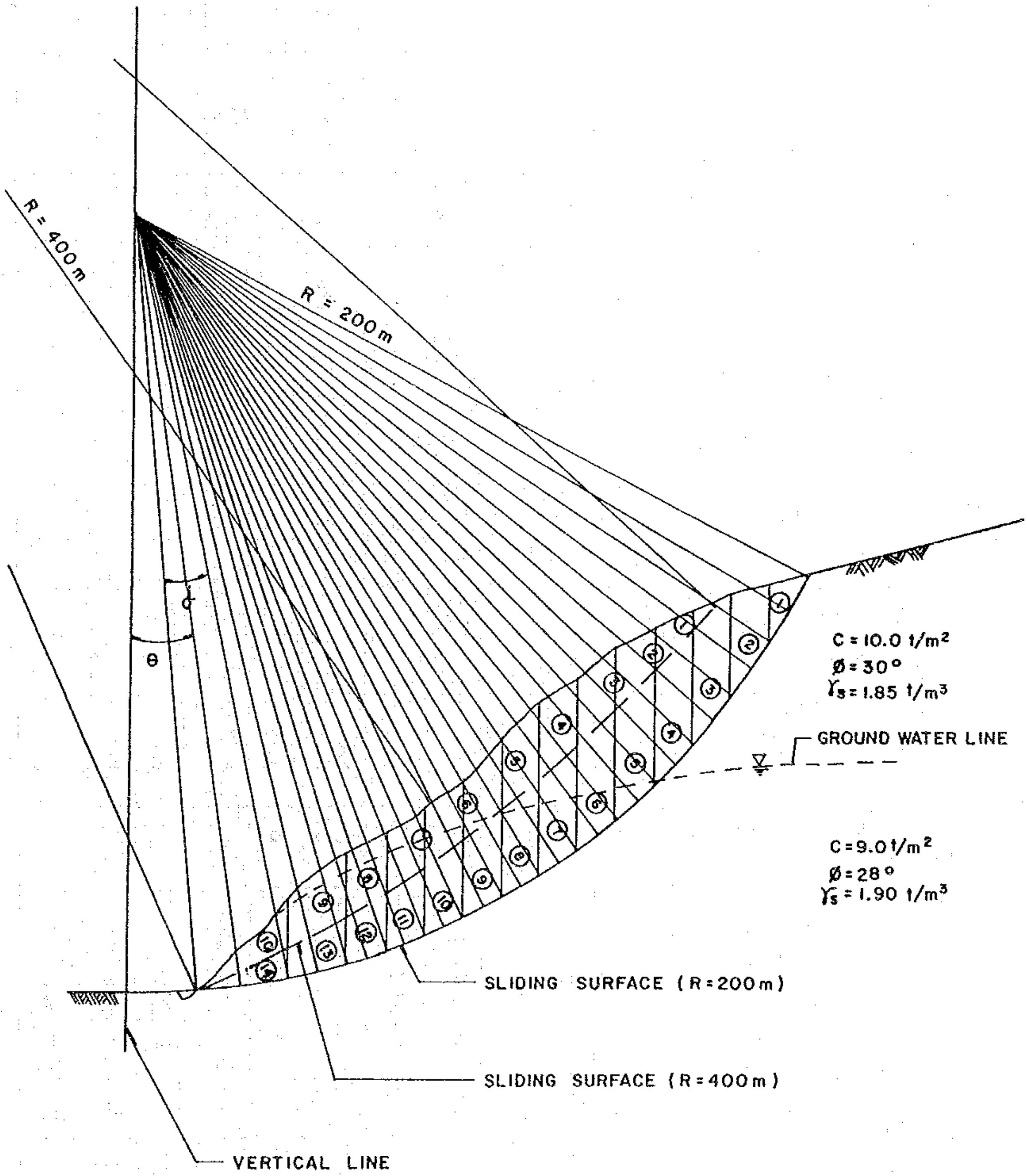
c' : Cohesion of soil for effective stress (t/m2)

ϕ' : Internal friction angle of soil for effective stress (degree)

θ : Refer to Figure 1-1 (degree)

1.4 EXAMPLE CALCULATION OF EMBANKMENT SLOPE

Case-1 R = 200 m



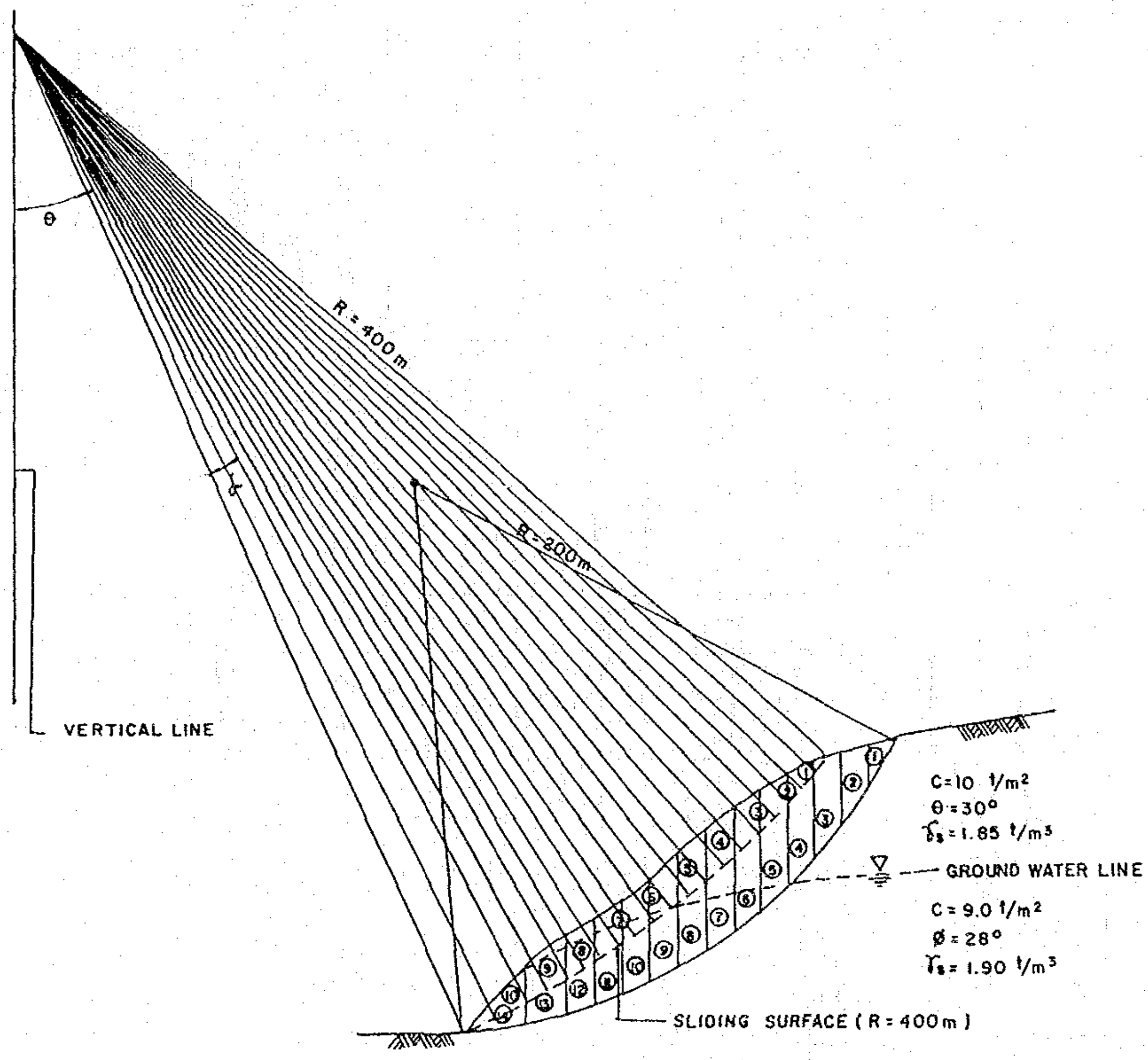
R = 200 M

Slice No.	(1) Area of Trapezium A (m^2)	(2) Unit Weight of Soil Γ (t/m^3)	(3) Weight of Slices W = Γ • A (t/m)	(4) θ (degree)	(5) cos θ	(6) sin θ	(7) W cos θ	(8) μ • L (t/m)	(9) W cos θ - μ • L	(10) (W cos θ - μ • L) x tan θ	(11) W sin θ	(12) α	(13) L (m) = $\frac{\alpha (2 \times r)}{360}$	(14) c • L (t/m)
1	1/2 x 10 x 15 = 75	1.85	138.75	59.5	0.5075	0.8616	70.415	-	70.415	40.654	119.547	5.5	19.199	191.99
2	$\frac{10 + 26}{2}$ x 10 = 180	1.85	333.00	54.5	0.5807	0.8141	193.373	-	193.373	111.646	271.095	5.0	17.453	174.53
3	$\frac{26 + 34}{2}$ x 10 = 300	1.85	555.0	49.5	0.6494	0.7604	360.417	-	360.417	208.086	422.022	4.5	15.708	157.08
4	$\frac{34 + 40}{2}$ x 10 = 370	1.85	684.50	45.5	0.7009	0.7132	479.766	-	479.766	276.993	488.185	4.0	13.963	139.963
5	$\frac{40 + 44}{2}$ x 10 - 52.361 = 367.639	1.85	680.132	41.5	0.7489	0.6626	583.856	$\frac{0 + 75}{2}$ L = 52.361	531.495	282.601	516.575	4.0	13.963	125.667
	1/2 x (13.963) x 7.5 = 52.361	1.90	99.486											
6	$\frac{44 + 44}{2}$ x 10 - 107.50 = 332.50	1.85	615.125	37.5	0.7933	0.6087	650.010	$\frac{7.5 + 14}{2}$ L = 131.333	518.677	275.785	498.753	3.5	12.217	109.953
	$\frac{7.5 + 14}{2}$ x 10 = 107.50	1.90	204.250											
7	$\frac{44 + 54}{2}$ x 10 - 107.50 = 325.00	1.85	601.250	34	0.8290	0.5592	758.327	$\frac{14 + 19}{2}$ L = 201.581	556.746	296.027	511.528	3.5	12.217	109.953
	$\frac{14 + 19}{2}$ x 10 = 165.00	1.90	313.50											
8	$\frac{54 + 42}{2}$ x 10 - 205.00 = 275.00	1.85	508.75	31	0.8571	0.5150	769.89	$\frac{19 + 22}{2}$ L = 250.448	519.442	276.192	462.598	3.5	12.217	109.953
	$\frac{19 + 22}{2}$ x 10 = 205.00	1.90	389.50											
9	$\frac{42 + 36}{2}$ x 10 - 230.00 = 160.00	1.85	296.00	27.5	0.8870	0.4617	650.171	$\frac{22 + 24}{2}$ L = 240.856	409.315	217.636	338.426	3.0	10.472	94.248
	$\frac{22 + 24}{2}$ x 10 = 230.00	1.90	437.00											
10	$\frac{36 + 33}{2}$ x 10 - 250.00 = 95.00	1.85	175.75	24.5	0.9099	0.4146	592.117	$\frac{24 + 26}{2}$ L = 261.80	330.317	175.633	269.801	3.0	10.472	94.248
	$\frac{24 + 26}{2}$ x 10 = 250.00	1.90	475.00											

R = 200 M (Continued)

Slice No.	(1) Area of Trapezium A (m^2)	(2) Unit Weight of Soil Γ (t/m^3)	(3) Weight of Slices $W = \Gamma \cdot A$ (t/m)	(4) θ (degree)	(5) cos θ	(6) sin θ	(7) W cos θ	(8) $\mu \cdot L$ (t/m)	(9) W cos θ − $\mu \cdot L$	(10) (W cos θ − $\mu \cdot L$) x tan θ	(11) W sin θ	(12) α	(13) (m) = $\frac{\alpha (2 \pi r)}{360}$	(14) c • L (t/m)
11	$\frac{33 + 31}{2}$ x 10 - 260.00 = 60.00	1.85	111.00	21.5	0.9304	0.3665	562.692	$\frac{26 + 26}{2}$ L = 272.272	290.620	154.525	221.733	3.0	10.472	94.248
	$\frac{26 + 26}{2}$ x 10 = 260.00	1.90	494.00											
12	$\frac{31 + 29}{2}$ x 10 - 250.00 = 50.00	1.85	92.50	18	0.9510	0.3090	539.692	$\frac{26 + 24}{2}$ L = 305.425	234.267	124.562	175.357	3.5	12.217	109.953
	$\frac{26 + 24}{2}$ x 10 = 250.00	1.90	475.00											
13	$\frac{29 + 21}{2}$ x 15 - 250.00 = 50.00	1.85	97.125	14.5	0.9681	0.2503	687.23	$\frac{24 + 19}{2}$ L = 300.205	387.025	205.785	177.682	4.0	13.963	125.667
	$\frac{24 + 19}{2}$ x 10 = 322.50	1.90	612.75											
14	1/2 (26.18) - 248.71 = 26.18	1.85	48.433	8.5	0.9890	0.1478	515.251	$\frac{19 + 0}{2}$ L = 248.71	266.541	141.722	77.001	7.5	26.18	235.62
	1/2 (26.18) (19) = 248.71	1.90	472.549											
	Fs = (1873.073 + 2787.845)/4550.303 = 1.024									2787.815	4550.303			1873.073

Case-2 R = 400 m



θ
R = 400 m
δ
R = 200 m
VERTICAL LINE
C=10 t/m²
θ=30°
γs=1.85 t/m³
GROUND WATER LINE
C=9.0 t/m²
ø=28°
γs=1.90 t/m³
SLIDING SURFACE (R=400m)

R = 400 M

Slice No.	(1) Area of Trapezium A (m^2)	(2) Unit Weight of Soil Γ (t/m^3)	(3) Weight of Slices $W = \Gamma \cdot A$ (t/m)	(4) θ (degree)	(5) $\cos\theta$	(6) $\sin\theta$	(7) $W\cos\theta$	(8) $\mu \cdot L$ (t/m)	(9) $W\cos\theta - \mu \cdot L$	(10) $(W\cos\theta - \mu \cdot L) \times \tan\theta$	(11) $W\sin\theta$	(12) α	(13) $L(m) = \frac{\alpha(2\pi r)}{360}$	(14) $c \cdot L$ (t/m)
1	$1/2 \times 8 \times 11 = 44.00$	1.85	81.40	48	0.6691	0.7431	54.464	-	54.464	31.444	60.488	2.0	13.963	139.63
2	$\frac{8+13}{2} \times 10 = 105.00$	1.85	194.25	46	0.6946	0.7193	134.926	-	134.926	77.899	139.724	2.25	15.708	157.08
3	$\frac{13+16}{2} \times 12.50 = 181.25$	1.85	335.31	43.50	0.7254	0.6883	243.233	-	243.233	140.431	230.793	2.25	15.708	157.08
4	$\frac{16+19}{2} \times 12.50 = 218.75$	1.85	404.68	41.25	0.7518	0.6593	304.238	-	304.238	175.652	266.806	2.50	17.453	174.53
5	$\frac{19+18}{2} \times 12.50 = 231.25$	1.85	427.81	38.75	0.7798	0.6259	333.606	-	333.606	192.607	267.765	2.25	15.708	157.08
6	$\frac{18+10}{2} \times 12.50 = 175.00$	1.85	323.75	36.50	0.8038	0.5948	317.501	$\frac{6}{2} \cdot L = 47.124$	270.377	143.762	234.946	2.25	15.708	141.372
	$1/2 \times (6)(12.50) = 37.50$	1.90	71.25											
7	$\frac{10+6}{2} \times 12.50 = 100.00$	1.85	185.00	34.25	0.8266	0.5628	300.159	$\frac{6+9}{2} \cdot L = 1117.81$	182.349	96.956	204.367	2.25	15.708	141.372
	$\frac{6+9}{2} \times 12.50 = 93.75$	1.90	178.125											
8	$\frac{6+5}{2} \times 12.50 = 68.75$	1.85	127.187	32.25	0.8457	0.5336	318.458	$\frac{9+12}{2} \cdot L = 164.934$	153.524	81.630	200.934	2.25	15.708	141.372
	$\frac{9+12}{2} \times 12.50 = 131.25$	1.90	249.375											
9	$\frac{5+1}{2} \times 15.00 = 45.00$	1.85	83.25	30	0.8660	0.5000	368.266	$\frac{12+12}{2} \cdot L = 209.436$	158.79	84.430	212.625	2.50	17.453	157.077
	$\frac{12+12}{2} \times 15.00 = 180.00$	1.90	342.00											
10	$1/2 \times (1) \times (5) = 2.50$	1.85	4.625	26.75	0.8929	0.4501	252.804	$\frac{12}{2} \cdot L = 157.08$	95.724	50.897	127.4354	3.75	26.18	235.62
	$1/2 \times (24.43) \times 12 = 146.58$	1.90	278.502											
	$F_s = (1602.213 + 1075.708)/1945.884 = 1.376$									1075.708	1945.884			1602.213

2. DRAINAGE

2.1 DESIGN YEAR OF RAINFALL PROBABILITY

The factor influencing the design of drainage facilities is, of course, run-off due to rainfall, characteristics of which shall therefore be carefully examined. Similarly, other factors to be considered are the importance of the road and the anticipated degree of damage when actual run-off exceeds expected design discharge. Therefore, the design year of rainfall probability shall be determined giving considerations to topographic characteristics aside from the factors mentioned above. Table 2-1 presents the recommended design year of rainfall probability.

TABLE 2-1 DESIGN YEAR OF RAINFALL PROBABILITY

Required Level of Drainage	Design Year of Rainfall Probability	
	Road Surface and Small Scale Slope	Important Drainage Facility
High	3 years	more than 10 years
Average	2 years	7 years
Low	1 year	5 years

Required level of drainage may be decided in accordance with the importance of the road.

2.2 CALCULATION OF RUN-OFF

Run-off is calculated by the following Rationale Formula.

$$Q = (1/3.6 \times 10^6) \cdot C \cdot I \cdot A$$

Where:

Q = Run-off (m3/sec)
C = Coefficient of run-off
I = Rainfall Intensity within time of concentration (mm/h)
A = Catchment Area (m2)

2.2.1 Coefficient of Run-Off

The coefficient of run-off varies on the condition of ground surface, slant, soil, duration of rainfall, etc. The standard value of coefficient of run-off shown in Table 2-2 may be used for the calculation of run-off.

The "C" value in the Rationale Formula reflects this variation in the terrain.

2.2.2 *Rainfall Intensity*

The value of rainfall intensity (mm/h) is found from the Rainfall Intensity Curve. Time of concentration for the different surface characteristics of the catchment is shown in Figure 2-1 (1) to 2- 1 (4).

The catchment should be divided into separate areas, a_1 to a_n, where the corresponding value of Γ will be constant, hence:

$$= \Gamma \times (c_1 \cdot a_1 + c_2 \cdot a_2 + c_3 \cdot a_3 \ldots\ldots + c_n \cdot a_n)$$

Where a_1 to a_n are the number of each sub-areas

c_1 to c_n are the corresponding coefficients of run-off

2.2.3 Time of Concentration

$$t = t1 + t2$$

Where:

t = Time of Concentration (min)

Travel time in minutes of water from the farthest point to the point where run-off is to be calculated.

t1 = Inlet time from slope to water course (min) (Refer to Figure 2-1).

t2 = travel time from water course to the point where run-off is to be calculated (min).

$$t2 = L/60 \cdot V$$

L = Horizontal length of water course (m)

v = Average velocity in water course (m/sec)

2.2.4 Catchment Area

The catchment area to be considered may be determined by one of the following methods:

1) Direct field survey using conventional survey instruments;
2) Use of topographical maps together with field surveys to check details, e.g., artificial barriers such as terraces, ponds, etc;
3) Aerial photography.

TABLE 2-2 COEFFICIENT OF RUN-OFF

Kind of Ground Surface		Coefficient of Run-Off
Surface of road	Pavement	0.70 to 0.95
	Gravel road	0.30 to 0.70
Shoulder, slope, etc.	Fine-grained soil	0.40 to 0.65
	Coarse-grained soil	0.10 to 0.30
	Hard rock	0.70 to 0.85
	Soft rock	0.50 to 0.75
Lawns on sandy soil	Gradient 0 to 2%	0.05 to 0.10
	2 to 7%	0.10 to 0.15
	More than 7%	0.15 to 0.20
Lawns on cohesive soil	Gradient 0 to 2%	0.13 to 0.17
	2 to 7%	0.18 to 0.22
	More than 7%	0.25 to 0.35
Ridge		0.75 to 0.95
Intermediate area		0.20 to 0.40
Park with lawns and many trees and forest		0.10 to 0.25
Mountain with gentle slope		0.20 to 0.40
Mountain with steep slope		0.40 to 0.60
Paddy field, water surface		0.70 to 0.80
Field		0.10 to 0.30

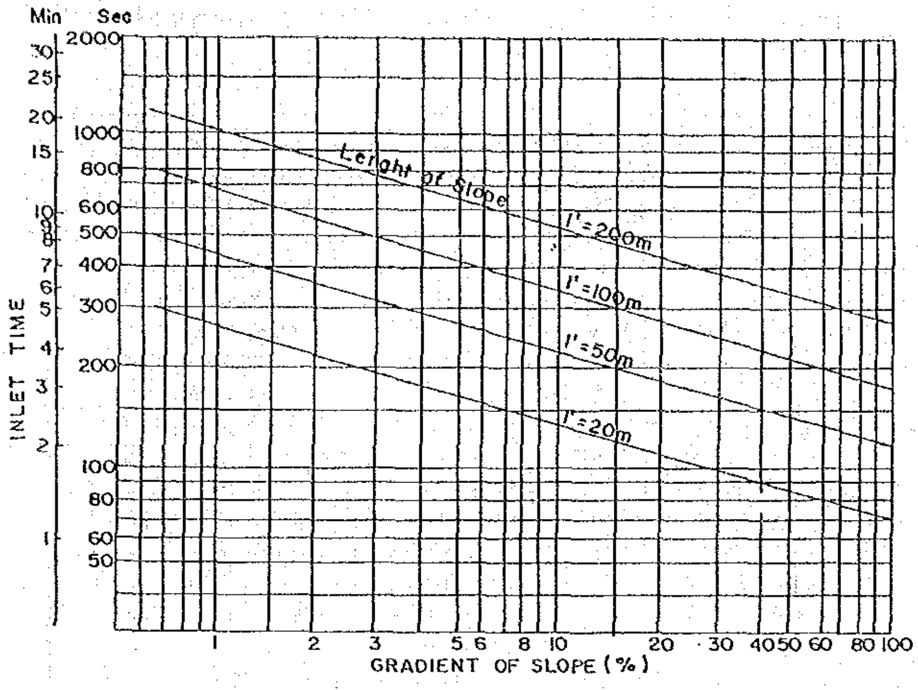


FIGURE 2-1 (1) INLET TIME (SMOOTH SURFACE, n = 0.02)

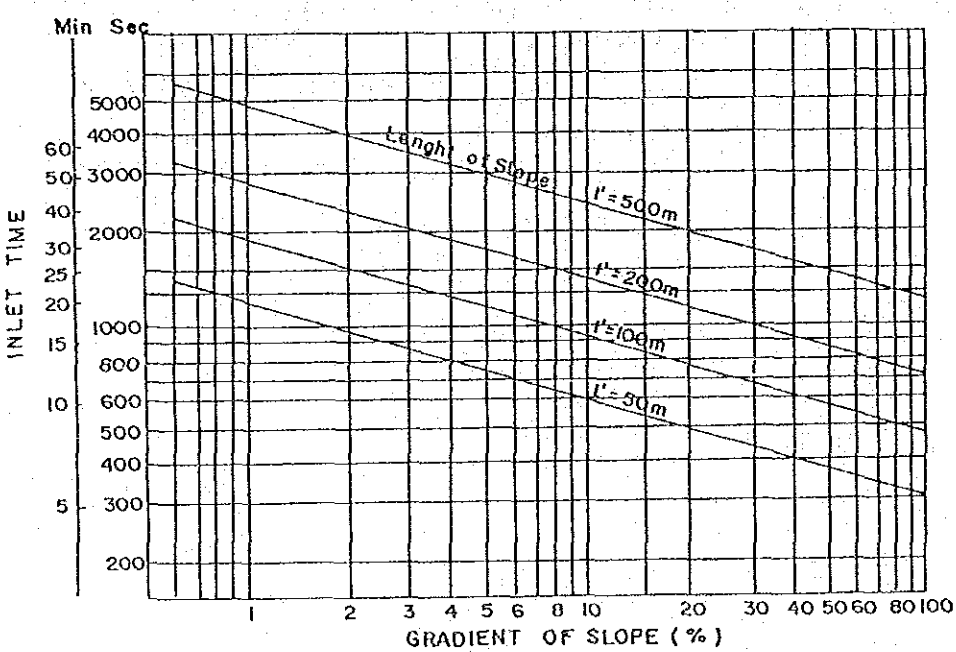


FIGURE 2-1 (2) INLET TIME (BARE AREA WITH NO STONE n = 0.10)

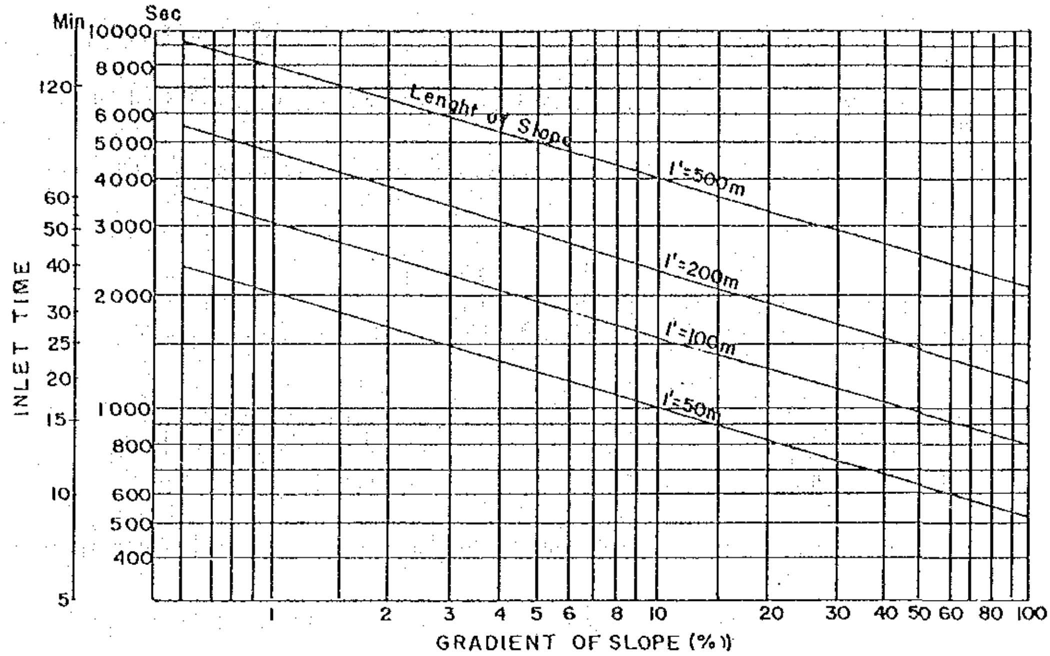


FIGURE 2-1 (3) INLET TIME (AREA WITH FEW GRASSES, BARE AREA WITH n = 0.2)

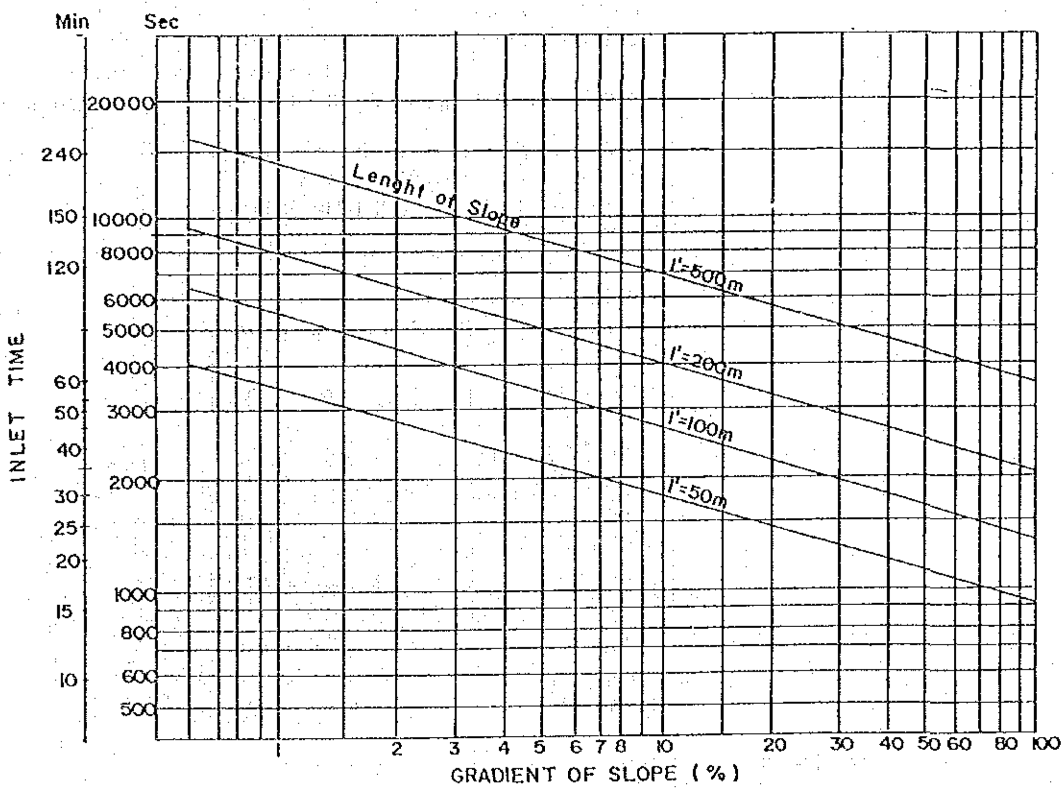


FIGURE 2-1 (4) INLET TIME (ORDINARY GRASSLAND n = 0.4)

2.3 RUNNING WATER VELOCITY

The running water velocity is calculated using Manning's Formula.

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

Where;

n ; Coefficient of roughness ($sec/m^{1/3}$)
i ; Hydraulic Gradient
R ; Hydraulic Radius R = A/P (m)
A ; Area of running water (m^2)
P ; Wetted perimeter (m)

Travel time of water flows in water course to the point under consideration may be calculated using the estimated velocity.

Required cross sectional area of water course (side ditch) is calculated using the following formula.

$$Q = A\ V$$

Where;

Q ; Discharge of side ditch (m^3/sec)
A ; Cross-sectional area of side ditch (m^2)
V ; Mean velocity of stream (m/sec)

Table 2-3 shows the coefficient of roughness generally adopted for different types of drainage.

TABLE 2-3 COFFICIENT OF ROUGHNESS

Type of Drainage		Coefficient of Roughness (n)
Earth and Gravel	Earth	0.020 ~ 0.025
	Sand and Gravel	0.025 ~ 0.040
	Rock	0.025 ~ 0.035
Cast-in-Place	Cement Mortar	0.010 ~ 0.013
	Concrete	0.013 ~ 0.018
	Stone Pitching	0.015 ~ 0.030
Fabricated	Concrete Pipe	0.012 ~ 0.016

3. CONCRETE RETAINING WALL

3.1 GENERAL

Concrete retaining walls are structures to support and retain earth in order to prevent failure of sediment in the places where stability of slope can not be assured by normal ground conditions (earth alone) or by other slope protection works.

Concrete retaining walls are classified into the following types in accordance with the shapes and characteristics.

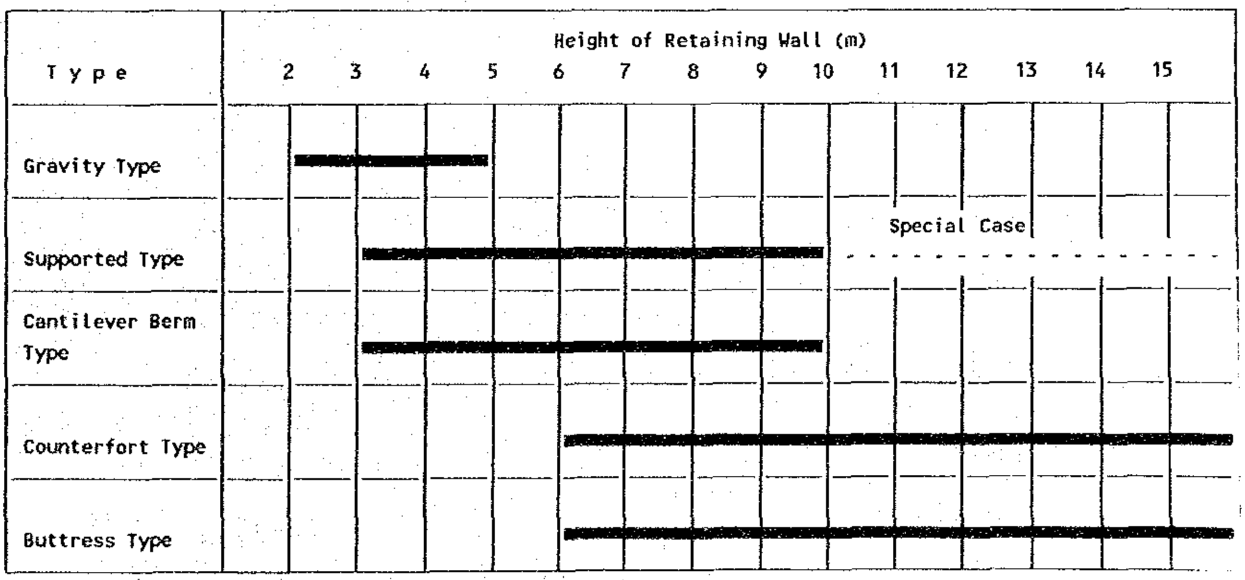
- Gravity Type Retaining Wall
- Supported Type Retaining Wall
- Cantilever Beam Type Retaining Wall
- Counterfort Type Retaining Wall
- Buttress Type Retaining Wall

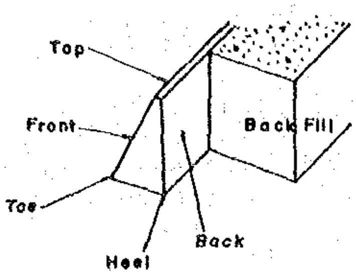
Figure 3-1 show the (conceptional) shapes of each type of retaining wall.

To select a most appropriate type of retaining wall, a comparative analysis is recommended among several alternative types. In the analysis, topography, geology and conditions for construction should be taken into consideration.

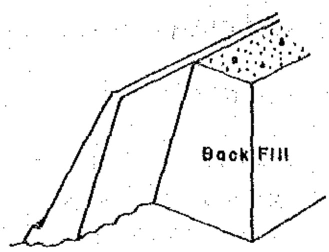
In Table 3-1, types of retaining walls generally adopted are presented in accordance with the height. The typical example and the general application of each type are discussed this chapter.

TABLE 3-1 RECOMMENDED TYPES OF RETAINING WALL IN ACCORDANCE WITH HEIGHT

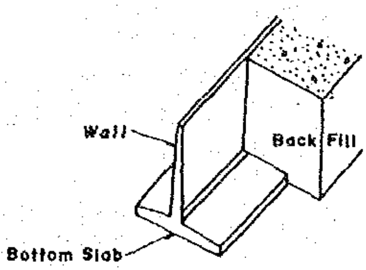




(1) Gravity Type

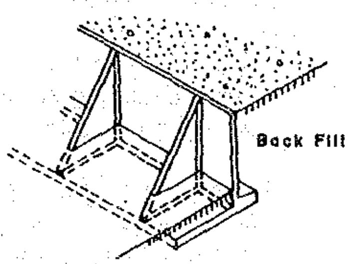


(2) Supported Type

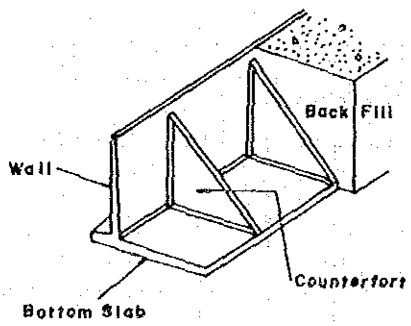


Inverted T Type

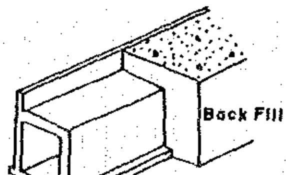
(3) Cantilever Beam Type



(4) Buttless Type



Wall Type



Box Type

(5) Counterfort Type

FIGURE 3-1 TYPES OF RETAINING WALL

3.2 LOADS

Generally, loads acting to retaining wall are dead load, surcharge, earth pressure, bouyancy, impact, water pressure and earthquake. Loads to be considered in the design are a combination of dead load, surcharge and earth pressure.

Design analysis for earthquake in ordinary retaining wall on embankment is not required considering the fact that the increase of load due to seismic force will be compensated by a safety factor which may be assessed slightly bigger than the normal case and by resistance forces against which are not taken into account in the design.

However, retaining wall higher than 8.0 m or failure of which may cause serious damages should be designed taking into consideration effect of earthquake.

3.2.1 Dead Load

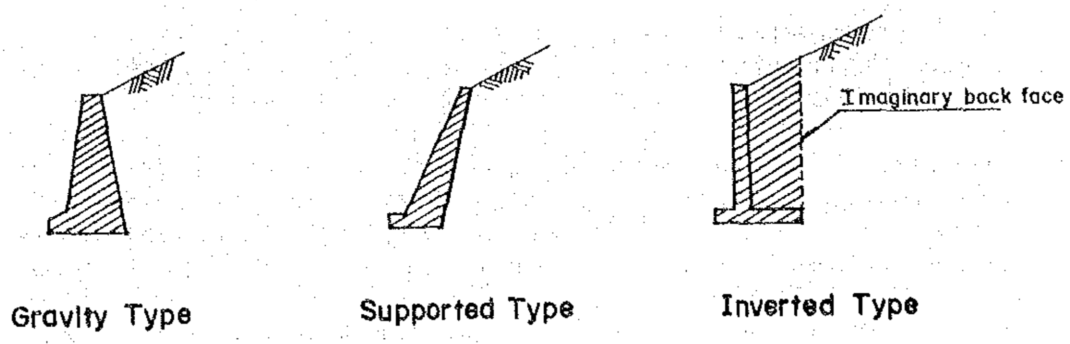
The dead load acting on a retaining wall is made up of weight of the structure itself and the weight of the earth directly on of the footing slab, as shown in Figure 3-2.

The unit weight of each material to be used in design is indicated in Table 3-2.

TABLE 3-2 UNIT WEIGHT OF MATERIAL

Material	Unit Weight (t/m^3)
Reinforced Concrete	2.5
Concrete	2.35
Gravel, Gravelly Soil, Sand	2.0
Sandy Soil	1.9
Silt, Cohesive Soil	1.8

FIGURE 3-2 DEAD LOAD OF RETAINING WALL



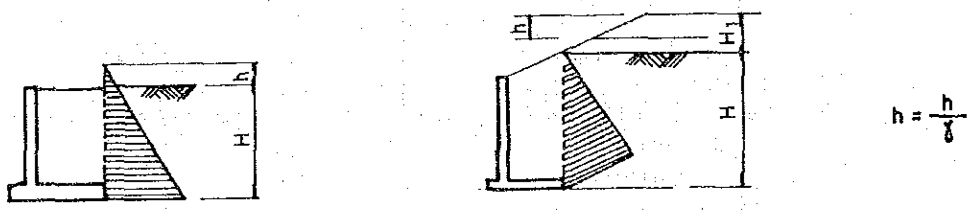
3.2.2 Surcharge

The live load acting on a retaining wall which is considered as surcharge is given by the equation.

$q = 1.0$ t/m2

where: q = Live load acting on retaining wall

In actual design procedure, the level of ground behind retaining wall can be increased by its height equivalent to the amount of the live load. See Figure 3-3.



Note: h: Height of earth equivalent to surcharge

q: 1.0 t/m²

Γ: Unit weight of soil (t/m3)

FIGURE 3-3 SURCHARGE

3.2.3 Earth Pressure for Embankment Section

Earth pressure component which is acting on an ordinary retaining wall is calculated by Trial Wedge Shape Method.

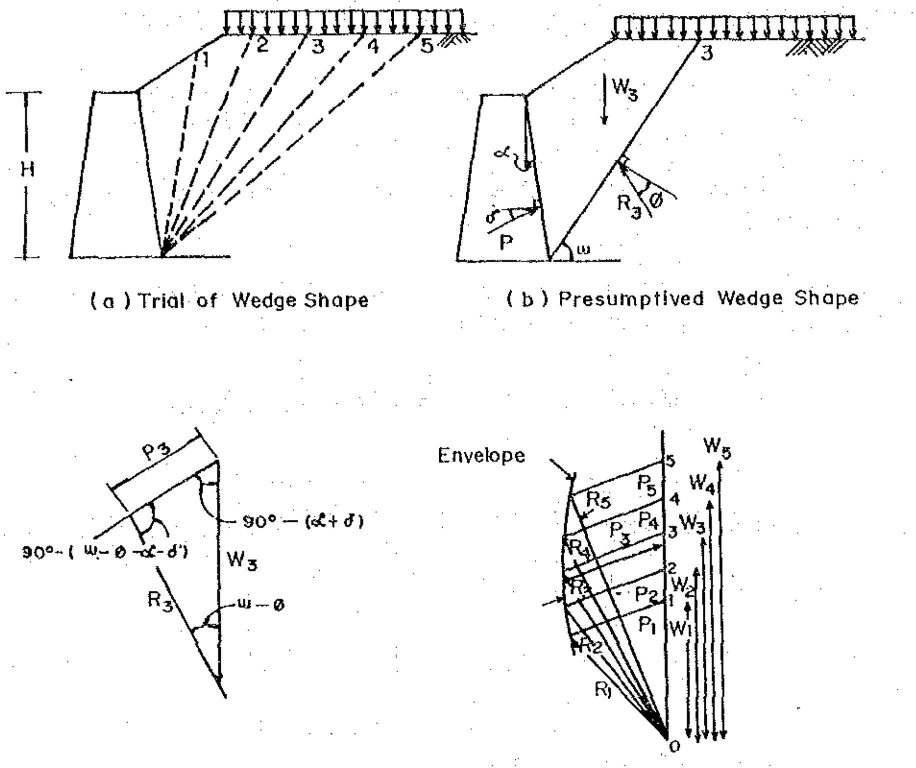
Characteristics of backfill which will be used in the calculation is preferably determined by soil test.

For the design of retaining wall with less than 8.0 m in height, however, the values of internal friction angle shown in Table 3-3 can be available.

TABLE 3-3 INTERNAL FRICTION ANGLE

Type of Backfill	ϕ (degree)
Gravel with Sand	35
Sandy Soil	30
Silt	25

The concept of Trial Wedge Shape Method is shown in Figure 3-4, Vertical and Horizontal element of earth pressure is shown in Figure 3-5.



(a) Trial of Wedge Shape

(b) Presumptived Wedge Shape

(c) Funicular Polygon

(d) Combination of Funicular Polygon to ESTIMATE Max Pa

where:

H : Height of wall for calculation of earth pressure (m)

W : Weight of soil within the wedge shape (including surcharge) (t/m)

R : *Reaction of active pressure to sliding face (t/m)*

P : Resultant of active earth pressure (t/m)

α : Angle between imaginary back face of wall and vertical line

Ø : Internal friction angle

δ : Friction angle between Imaginary Back Face of Wall and Soil (refer to Table 3-4).

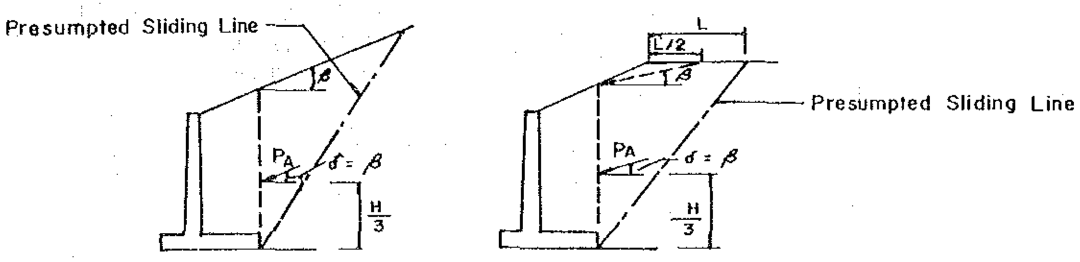
θ : *Angle between Imaginary Sliding Line and Horizontal Line*

FIGURE 3-4 CONCEPT OF TRIAL METHOD WITH WEDGE SHAPE

TABLE 3-4 FRICTION ANGLE BETWEEN SOIL AND IMAGINARY BACK FACE OF WALL (δ)

Type of Retaining Wall	Condition	δ
Gravity Type Supported Type	Soil to Concrete	$2/3 \cdot \phi$
Cantilever Berm Type Buttress Type	Soil to Soil	β

Note: β: Refer to Figure 3-5



Horizontal element $P_H = P_A \cos(\alpha + \delta)$
Vertical element $P_V = P_A \sin(\alpha + \delta)$

FIGURE 3-5 VERTICAL AND HORIZONTAL ELEMENT OF EARTH PRESSURE

Coefficient of earth pressure which is calculated by Trial Wedge Shape Method is shown in Figure 3-6.

The earth pressure can be calculated by following equations after obtaining coefficient of earth pressure on Figure 3-6.

$$Ph = 1/2 \cdot kh \cdot \Gamma \cdot H^2$$

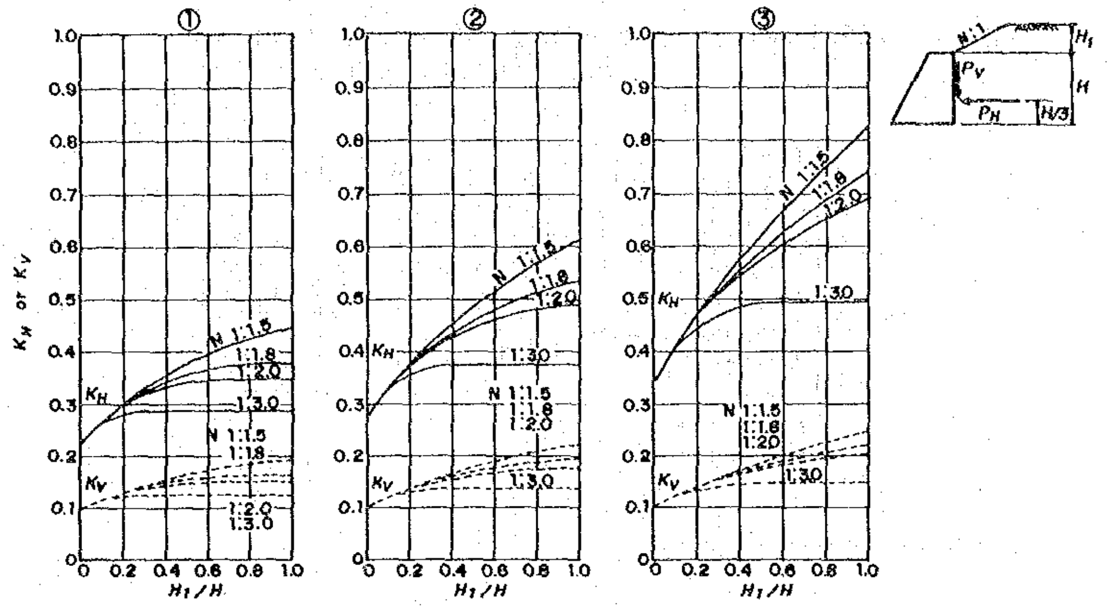
$$Pv = 1/2 \cdot kv \cdot \Gamma \cdot H^2$$

Where:

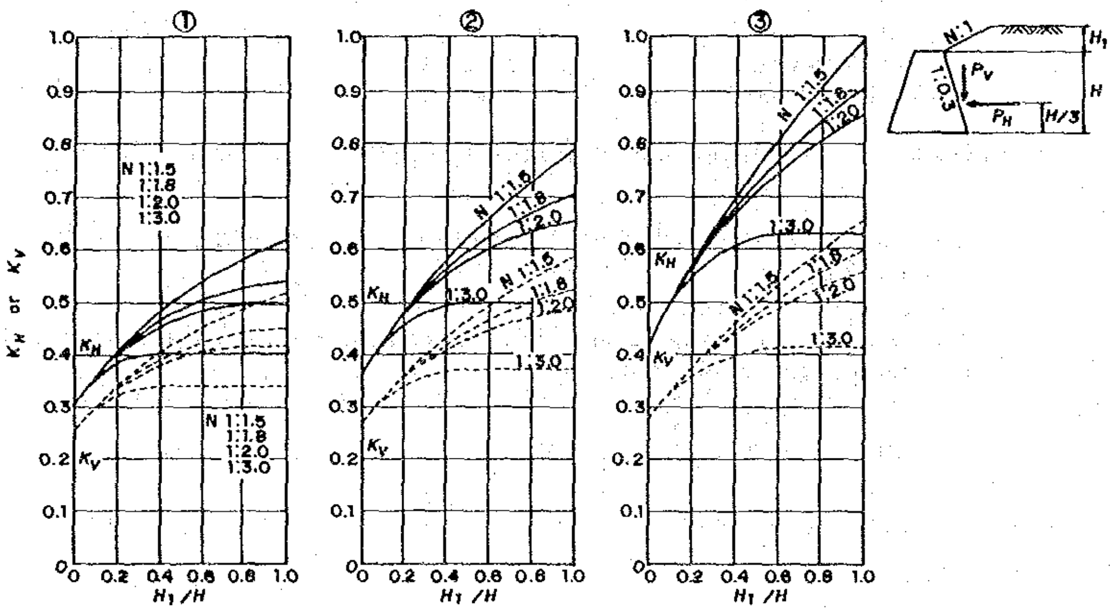
Kh : Coefficient of horizontal earth pressure

Kv : Coefficient of vertical earth pressure

Γ : Unit weight of backfill (t/m3)



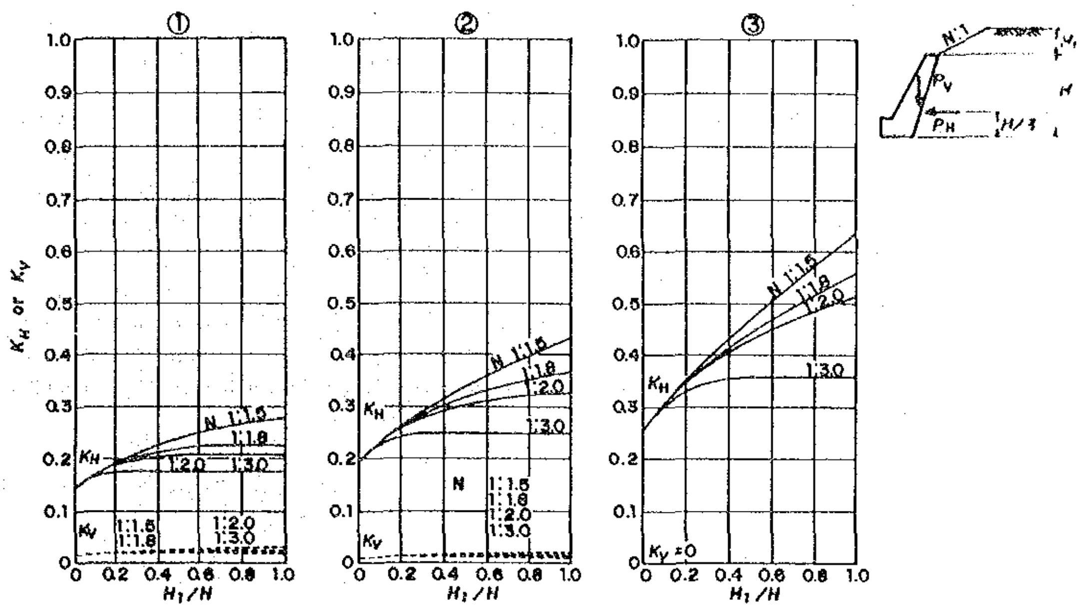
Gravity Type: Back Face is Vertical



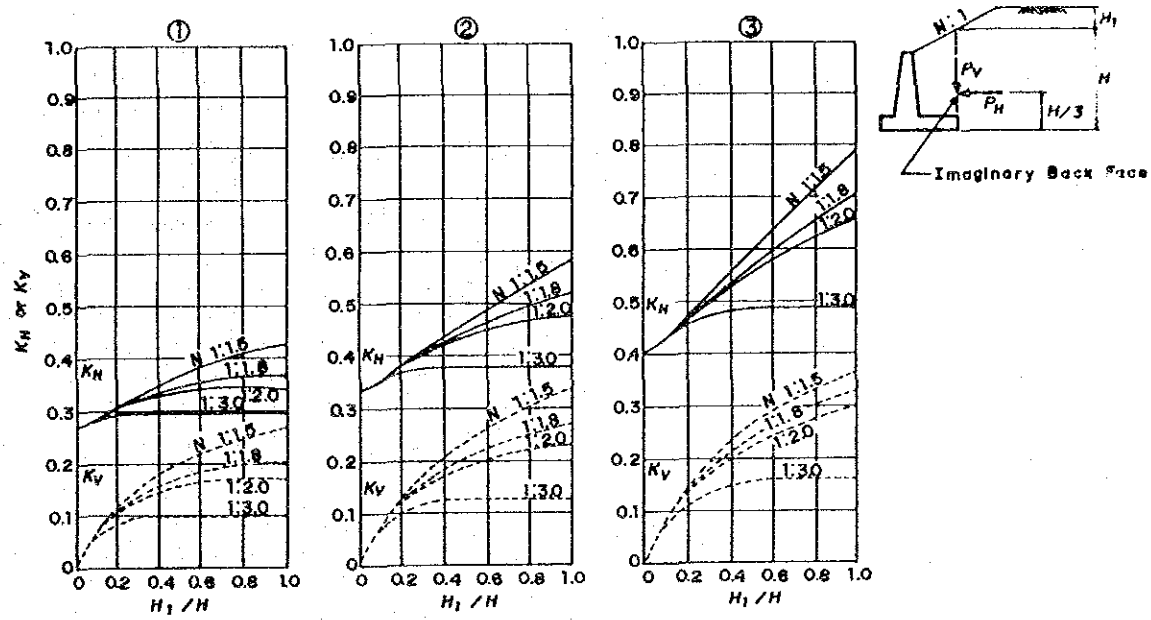
Gravity Type: Back Face is 0.3:1

Note: Figures on curve show kind of soil used for backfilling as follows:
(1) Gravel, Gravelly Soil and Sand
(2) Sandy Soil
(3) Silt and Cohesive Soil (WL 50%)

FIGURE 3-6 (1) COEFFICIENT OF EARTH PRESSURE FOR DESIGN (GRAVITY TYPE RETAINING WALL)



Supported Type: Back Face is 0.3:1



Inverted T Type: Imaginary Back Face is Vertical

Note: Figures on curve show kind of soil used for backfilling as follows:
(1) Gravel, Gravelly Soil and Sand
(2) Sandy Soil
(3) Silt and Cohesive Soil (W_L 50%)

FIGURE 3-6 (2) COEFFICIENT OF EARTH PRESSURE FOR DESIGN (SUPPORTED TYPE AND INVERTED T TYPE RETAINING WALL WITH A HEIGHT LESS THAN 8 M)

3.2.4 Earth Pressure for Cut Slope

In case of backfilling on a cut section calculation procedure for earth pressure acting on retaining wall is different from that of embankment section.

When a cut slope is unstable, the soil movement at the back of retaining wall may be considered the same in the case of an embankment.

On the other hand, in case of stable cut slope, earth pressure should be calculated by the the Trial Wedge Shape Method considering the gradient of cut slope and roughness of surface. Slide may be assumed to occur along the surface of cut slope, as shown in Figure 3-7.

With Funicular Polygon Analysis, earth pressure for stable cut slope is given by the following formula.

$$P = \frac{W \cdot \mathrm{Sin}\,(\Theta - \delta')}{\mathrm{Cos}\,(\Theta - \delta' - \alpha - \delta)}$$

Where:

- P : Earth pressure
- W : Weight of backfilled Soil (t/m)
- δ' : Internal friction angle of Cut Slope ($\delta = 2/3\ \emptyset \sim \emptyset$)
- δ : Friction angle between soil and imaginary back face of wall
- α : Angle between imaginary back face of wall and vertical line
- Θ : Gradient of cut slope (degree)

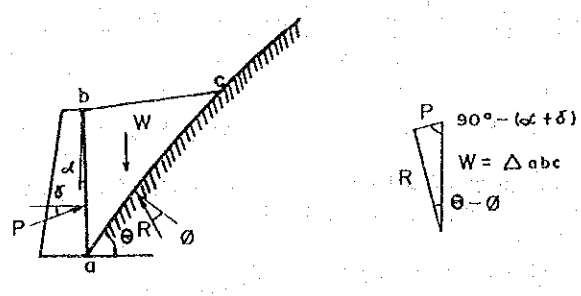
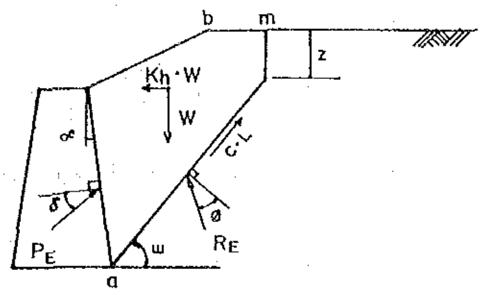
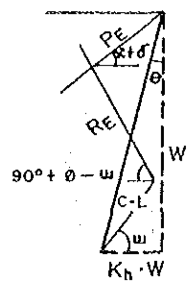


FIGURE 3-7 ANALYSIS OF EARTH PRESSURE FOR CUT SECTION

3.2.5 Earth Pressure during Earthquake

When analysis on the effect of earthquake is required, horizontal seismic intensity should be carefully determined based on seismic characteristics of area, condition of foundation, etc. During earthquake, earth pressure is determined by the the Funicular Polygon Method as in Figure 3.8.





(a) Presumptived Wedge Shape (b) Funicular Polygon

Where:

Kh: Coefficient of seismic force
θ: $\tan^{-1}$ kh
C: Cohesion (t/m2)
L: Presumed length of sliding line
Z: Depth of tension crack
Z: $2.c/\Gamma .\tan(45°+\varnothing/2)$
Γ: Unit weight of backfill (t/m3)
Ø: Internal friction angle

FIGURE 3-8 ANALYSIS OF EARTH PRESSURE DURING EARTHQUAKE

1) Inertial Force due to Earthquake

Inertial force due to earthquake can be estimated by the following formula:

$Fe = K_h \times W$

Where:

Fe : Inertial force due to earthquake

W : Weight of retaining wall which shall be considered as hatched portion in Figure 3-9.

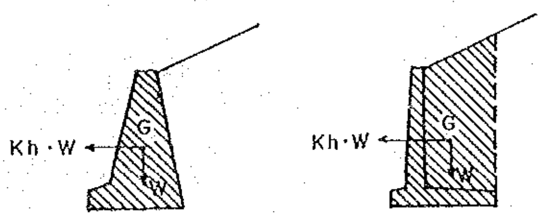


FIGURE 3-9 INERTIAL FORCE OF RETAINING WALL DUE TO EARTHQUAKE

3.3 CALCULATION OF STABILITY

Stability of retaining wall should be analyzed on the basis of the five consideration shown below. 1), 2) and 3) of them should be studied for calculation of stability while 4) and 5) will be analyzed depending on scale of structures and soil conditions.

1) Stability on sliding
2) Stability on overturning
3) Stability on bearing capacity of bearing ground
4) Stability during earthquake
5) Stability as whole system including embankment and foundation

3.3.1 Stability on Sliding

The horizontal component of earth pressure creates sliding of retaining wall along the bottom slab and friction force between the bearing ground and bottom of the slab bears that sliding force. The passive earth pressure in front of the retaining wall can be also considered as resisting force, but its reliability cannot be expected in long term, thus the passive earth pressure is normally neglected in design.

The factor of safety F_s against sliding is calculated by the following equation:

$$Fs = \frac{\text{Resisting force against sliding}}{\text{Sliding Force}}$$

$$= \frac{(W + P_V) \tan \delta + C \cdot B}{P_H} \geq 1.5$$

Where:

Fs : Factor of safety

Pv : Vertical component of earth pressure (t/m)

P_H : Horizontal component of earth pressure (t/m)

$\tan \delta$: Coefficient of friction between the bearing ground and bottom slab

$\delta = \emptyset$ for cast-in-place concrete, and

$\delta = 2/3\ \emptyset$ for other cases. However, the value of $\tan \delta$ should not exceed 0.6 when the bearing ground is the soil.
Normally, Table 3-5 can be practically used.

Ø : Internal friction angle of foundation ground

C : Cohesion between the bearing ground and bottom slab (t/m). But C should be zero (0) if the coefficient of friction tan δ is obtained from Table 3-5.

B : Width of bottom slab of retaining wall (m).

TABLE 3-5 DESIGN CONSTANT OF BEARING GROUND

Kind of Bearing Ground		Allowable Bearing Capacity (t/m^2)	Coefficient of Friction between Bearing Ground and Bottom Slab	Remarks	
				q_μ (t/m^2)	N-Value
Rock	Hard Rock with few cracks	100	0.7	over 1000	-
	Hard Rock with many cracks	60	0.7	over 1000	-
	Soft Rock	30	0.7	over 100	-
Gravel	Dense	30	0.6	-	-
	Not Dense	30	0.6	-	-
Sandy	Dense	30	0.6	-	30 ~ 50
	Reasonable Dense	20	0.6	-	15 ~ 30
Clay	Very firm	20	0.5	20 ~ 40	15 ~ 30
	Firm	10	0.45	10 ~ 20	8 ~ 15
	Reasonably firm	5	-	5 ~ 10	4 ~ 8

If the factor of safety is less than 1.5, the width of bottom slab should be reviewed. However, when widening of bottom slab is impracticable due to site conditions, engineering considerations, etc., the bottom slab should be placed in enough depth so that the passive earth pressure at the front of the structure can be effective. The following equation can be used, when the passive earth pressure is effective: See Figure 3-10.

$$Fs = \frac{(W + Pv) \cdot \tan \delta + c \cdot B + Pp}{Ph} \geq 1.5$$

Where:

Pp : Horizontal component of passive earth pressure

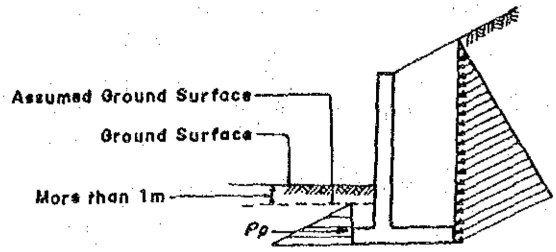


FIGURE 3-10 PASSIVE EARTH PRESSURE AGAINST SLIDING

Where the bearing ground is composed of firm soil or rock, a protuberant can be provided at the bottom of slab to increase resisting force against sliding. by the following equation. The equation stability is as follow: See Figure 3-11.

$$Hk = \frac{q1 + q3}{2} \cdot L1 \cdot \tan \varnothing + \frac{q2 + q3}{2} \cdot L2 \cdot \tan \delta + c \cdot L1$$

$$Fs = Hk/Ph \geq 1.5$$

Where:

Hk : Resisting Force against sliding

q1, q2, q3 : Ground Reaction (t/m2) (Refer to Figure 3-11)

L1, L2, : Width of bottom slab (m) (Refer to Figure 3-11)

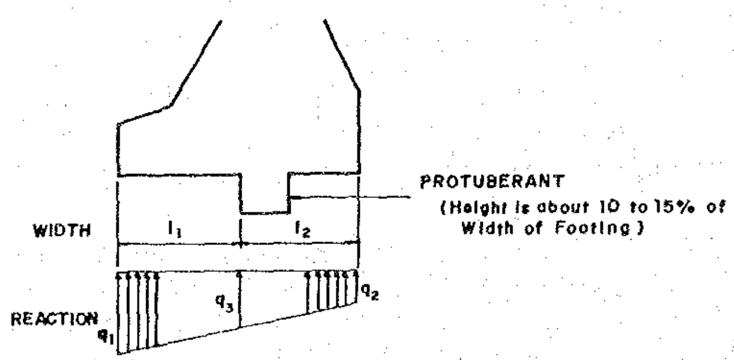


FIGURE 3-11 RESISTING FORCE AGAINST SLIDING WITH PROTUBERANT

3.3.2 Stability on Overturning

Stability on overturning is conditioned by the distance of eccentricity and the width of bottom slab.

The distance "d" from the toe to the acting point of resultant "R" can be expressed by the following equation.

$$d = \frac{W \cdot a + Pv \cdot b - Ph \cdot h}{W + Pv}$$

Where:

d : Distance from toe to acting point of resultant (m)

a : Horizontal distance from the toe of retaining wall to the center of gravity of W (m)

b : Horizontal distance from the toe of to the acting point of Pv (m)

h : Vertical distance from the heel to the acting point of PH (m)

Distance of eccentricity "e" from the center of the bottom slab to the acting point of resultant "R" can be expressed by:

$$e = \frac{B}{2} - d$$

Stability for overturning is conditioned by that the acting point of the resultant "R" must be within the central one-third portion of the width B of bottom slab. That is, the distance of eccentricity in the condition of safety must satisfy the following equation:

$$e \leq \frac{B}{6}$$

3.3.3 Stability on Bearing Capacity of Bearing Ground

Bearing capacity can be calculated by the following equation. See Figure 3-12.

$$q1 = (1/B)\ (Pv + W)\ (1 + 6e/B)$$

$$q2 = (1/B)\ (Pv + W)\ (1 - 6e/B)$$

q1 and q2 shall be

$$q1,\ q2 \leq qa = q\mu/Fs$$

Where:

qa : Allowable unit bearing capacity of ground

$q\mu$: Ultimate unit bearing capacity of ground

Fs : Factor of safety for bearing capacity of ground

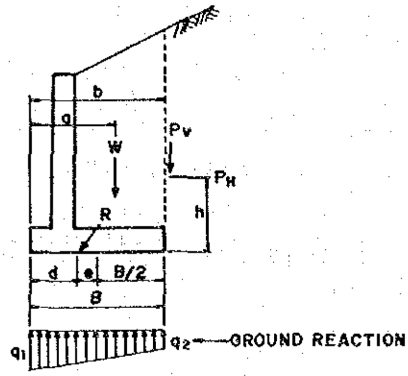


FIGURE 3-12 ANALYSIS OF BEARING CAPACITY

3.3.4 Stability During Earthquake

The following factors of safety should be used for the stability during earthquake:

(1) Factor of safety against sliding: $Fs \geq 1.2$

$F_s \geq 1.5$ when considering the passive earth pressure at the front.

(2) Stability on overturning

The acting point of resultant "R" must be within the central two- third portion of width B of the bottom slab. That is, the distance of eccentricity "e" must of satisfy the following equation for stability:

$$e \leq \frac{B}{3}$$

(3) Factor of safety for bearing capacity:

$$Fs \geq 2$$

Bearing capacity should be determined by the following equations.

When $e \leq B/6$ $\quad q_1 = (1/B)(P_{vE} + W)(1 + 6\,e/B)$

When $e \geq B/6$ $\quad q_1 = 2(P_{vE} + W)/3d$

Where:

P_{vE} = Vertical component of resultant of earth pressure during earthquake (t/m).

3.3.5 Stability of the Whole System Including Embankment and Foundation

For the retaining wall constructed on soft ground, stability of whole system should be checked including embankment and foundation. Stability analysis using Circular Arc Method can be applied, as shown in Figure 3-13.

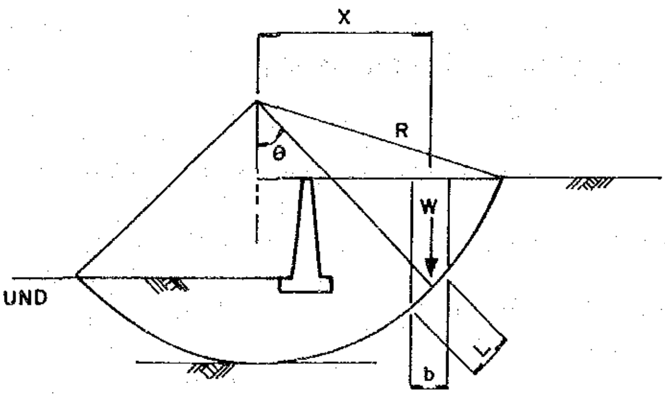


FIGURE 3-13 CIRCULAR RAPTURE ON SOFT GROUND

$$Fs = \frac{\Sigma \{c \cdot L + \tan \phi \cdot \cos \theta \, (w - \mu \cdot b)\}}{\Sigma W.x}$$

Where:

Fs : Factor of Safety

C : Cohesion along circular arc

ϕ' : Internal Friction Angle along circular arc

W : Weigth of slice

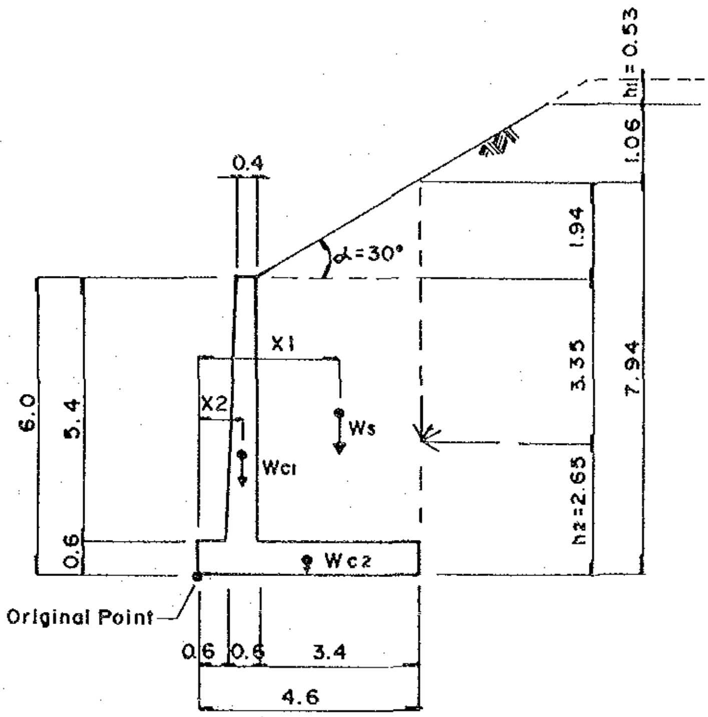
μ : Pore water pressure

(2) Retaining Wall on Slope

In many case, slope itself has problems of stability. Therefore, for the retaining wall on slope, the stability of slope including retaining wall and embankment, as a whole, should be examined, adopting the design concepts aforementioned.

3.4 EXAMPLE CALCULATION OF INVERTED T-TYPE

3.4.1 Cross Section



- Conversion Height of Surcharge (h_1)

$$h1 = \frac{1.0\ t/m2}{1.9\ t/m3} = 0.53\ m$$

- Height for Earth Pressure (h_2)

$$h2 = \frac{7.94}{3} = 2.65\ m$$

- Center of Gravity of Backfill (X_1)

$$x1 = 3.4 - \frac{3.4}{3} \times \frac{7.34 + 2 \times 5.4}{5.4 + 7.34} + 1.2 = 2.99\ m$$

- Center of Gravity of Wall (X_2)

$$X2 = 1.2 - \frac{0.4^2 + 0.4 \times 0.6 + 0.6^2}{3 \times (0.4 + 0.6)} = 0.95\ m$$

- Unit Weight of Backfill $\Gamma s = 1.9\ t/m^3$

3.4.2 Load

- Earth Pressure

$H1/H2 = 1.59/7.94 = 0.20$

Soil Grade (2) (Refer to Figure 3-6)

Slope Gradient N = 1.5:1

$KH = 0.38$

$Kv = 0.13$

$PH = \frac{1}{2} \times 0.38 \times (7.94)^2 = 11.98$ t/m

$Pv = \frac{1}{2} \times 0.13 \times (7.94)^2 = 4.10$ t/m

- Weight

$Ws = (5.4 + 7.34) \times 3.4 \times \frac{1}{2} \times 1.9 = 41.20$ t/m

$Wc = (0.4 + 0.6) \times 5.4 \times \frac{1}{2} \times 2.5 + 0.6 \times 4.6 \times 2.5$

$= 6.75 + 6.90 = 13.65$ t/m

3.4.3 Calculation of Stability

- Overturning

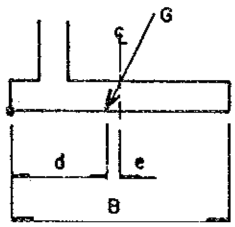
Moment

$$MPH = 11.98 \times 2.65 = 31.75 \text{ t . m}$$

$$MW + MPV = 41.2 \times 2.99 + 6.9 \times 2.3 + 6.75 \times 0.95 + 4.10 \times 4.6$$

$$= 123.19 + 15.87 + 6.41 + 18.86$$

$$= 164.33 \text{ t . m}$$



G: Acting Point of Component Force

$$d = \frac{Mw + MPV - MPH}{W + Pv} = \frac{164.33 - 31.75}{41.2 + 13.65 + 4.10}$$

$$= \frac{132.58}{58.95} = 2.25 \text{ m}$$

$$e = \frac{4.6 \text{ m}}{2} - 2.25 \text{ m} = 0.05 \text{ m} < B/6 = 0.77 \text{ m}$$

- Sliding

$$f = \frac{(W + Pv) \times \tan \emptyset}{PH} = \frac{58.95 \tan 30°}{11.98} = 2.8 > Fs = 1.5$$

- Bearing Capacity

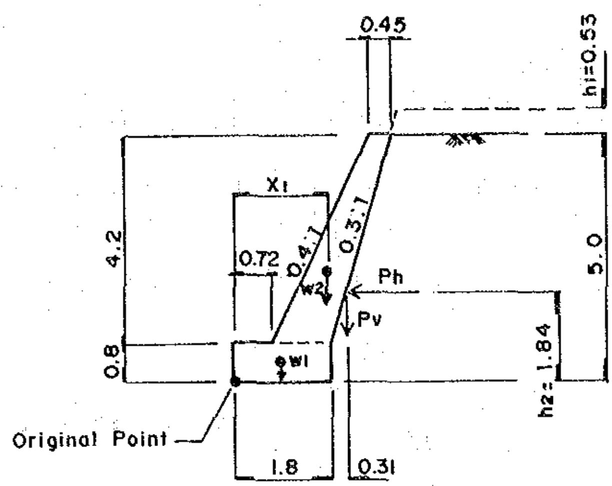
$$q1 = (67.45/4.6)\ (1 + 6 \times 0.16/4.6)$$

$$q2 = (67.45/4.6)\ (1 - 6 \times 0.16/4.6)$$

$$qd = 30 \text{ t/m (Refer to Table 3-5)}$$

3.5 EXAMPLE CALCULATION OF SUPPORTED TYPE

3.5.1 Cross Section



- Conversion Height of Surcharge (h_1)

$$h1 = \frac{1.0\ t/m2}{1.9\ t/m3} = 0.53\ m$$

- Height for Earth Pressure (h_2)

$$h2 = 5.53/3 = 1.84\ m$$

- Center of Gravity of Wall (X_1)

$$X1 = \frac{(0.45)2+0.45x1.08+(1.08)2+1.89x(0.9+1.08)}{3\ x\ (0.45 + 1.08)} + 0.72$$

$$= 1.94\ m$$

- Unit Weight of Backfill $\Gamma s = 1.9\ t/m^3$

3.5.2 Load

- Earth Pressure

$Hi/H = 0$

Soil Grade 2 (Refer to Figure 3-6)

$KV = 0.02$

$KH = 0.20$

$PH = \frac{1}{2} \times 0.20 \times (5.53)^2 = 3.06 \text{ t/m}$

$PV = \frac{1}{2} \times 0.02 \times (5.53)^2 = 0.31 \text{ t/m}$

- Weight

$W1 = 1.8 \times 0.8 \times 2.35 = 3.38 \text{ t/m}$

$W2 = (0.45 + 1.08) \times 4.2 \times \frac{1}{2} \times 2.35 = 7.55 \text{ t/m}$

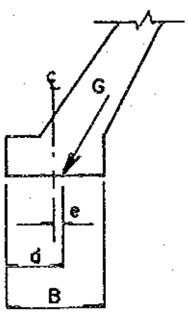
3.5.3 Calculation of Stability

- Overturning

Moment

$MPH = 3.06 \times 1.84 = 5.63 \text{ t/m}$

$Mw + MPv = 3.38 \times 0.9 + 7.55 \times 1.94 + 0.31 \times 2.11 = 18.34 \text{ t.m}$



G: Acting Point of Component Force

$$d = \frac{Mw + Mpv - MPH}{W + Pv} = \frac{18.34 - 5.63}{11.24} = 1.13 \text{ m}$$

$$e = 1.80/2 - 1.13 = -0.23 \text{ m} < B/6 = 0.30 \text{ m}$$

- Sliding

$$f = \frac{(W + Pv) \times \tan \emptyset}{PH}$$

$$= \frac{11.24 \times \tan 30°}{3.06} = 2.1 > fs = 1.5$$

- Bearing Capacity

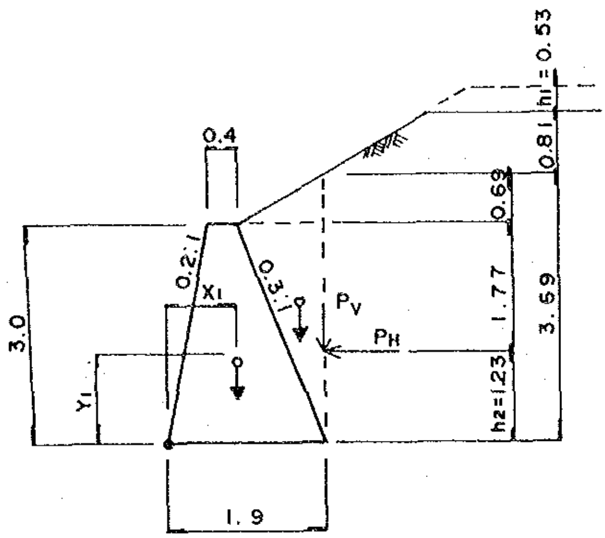
q1 = (11.24/1.8) (1 + 6 x -0.23/1.8) = 1.5 t/m2

q2 = (11.24/1.8) (1 - 6 x -0.23/1.8) = 11.0 t/m2

qd = 30 t/m2 (Refer to Table 3-5).

3.6 EXAMPLE CALCULATION OF GRAVITY TYPE

3.6.1 Cross Section and Configuration



- Conversion Height of Surcharge (h_1)

$$h1 = \frac{1.0 \text{ t/m2}}{1.9 \text{ t/m3}} = 0.53 \text{ m}$$

- Height for Earth Pressure

$$h2 = 3.67/3 = 1.23 \text{ m}$$

- Center of Gravity of Concrete Wall

$$y1 = \frac{3.0}{3} \times \frac{1.9 + 2 \times 0.4}{1.9 + 0.4} = 1.17 \text{ m}$$

$$x1 = \frac{\frac{1}{2}x0.6x3.0x0.4+0.4x3.0x0.8+\frac{1}{2}x0.9x3.0x1.3}{\frac{1}{2}x0.6x3.0+0.4x3.0+\frac{1}{2}x0.9x3.0} = 0.89 \text{ m}$$

- Unit Weight of Backfill Γs = 1.9 t/m^3

3.6.2 Load

- Earth Pressure

$$H1/H = \frac{1.34}{3.69} = 0.36$$

Soil Grade (2) (Refer to Figure 3-6)

Slope Gradient n = 1.5:1

KH = 0.55

Kv = 0.41

$$Ph = \tfrac{1}{2} \times 0.55 \times (3.69)^2 = 3.74 \text{ t/m2}$$
$$Pv = \tfrac{1}{2} \times 0.41 \times (3.69)^2 = 2.79 \text{ t/m2}$$

- Weight

$$W_s = 3.69 \times 0.9 \times \tfrac{1}{2} \times 1.9 = 3.15 \text{ t/m}$$

$$W_c = (0.4 + 1.9) \times 3.0 \times \tfrac{1}{2} \times 2.35 = 8.11 \text{ t/m}$$

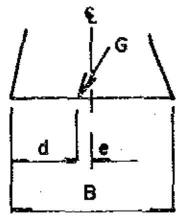
3.6.3 Calculation of Stability

- Overturning

$$MPH = 3.74 \times 1.23 = 4.60 \text{ t.m}$$

$$Mw + Mpv = 8.11 \times 0.89 + 3.15 \times 1.60 + 2.79 \times 1.90 = 17.56 \text{ t.m}$$

G: Acting Point of Component Force



$$d = \frac{Mw + Mpv - MpH}{W + Pv} = \frac{17.56 - 4.60}{3.15+8.11+2.79} = 0.92 \text{ m}$$

$$e = 1.90/2 - 0.92 \text{ m} = 0.03 \text{ m} < B/6 = 0.37 \text{ m}$$

- Sliding

$$f = \frac{(W + Pv) \times \tan \phi}{PH}$$

$$= \frac{(3.15+8.11+2.79) \times \tan 30°}{3.74} = 2.2 > fs = 1.5$$

- Bearing Capacity

$$q1 = (14.05/1.9)(1 + 6 \times 0.03/1.9) = 8.1 \text{ t/m2}$$

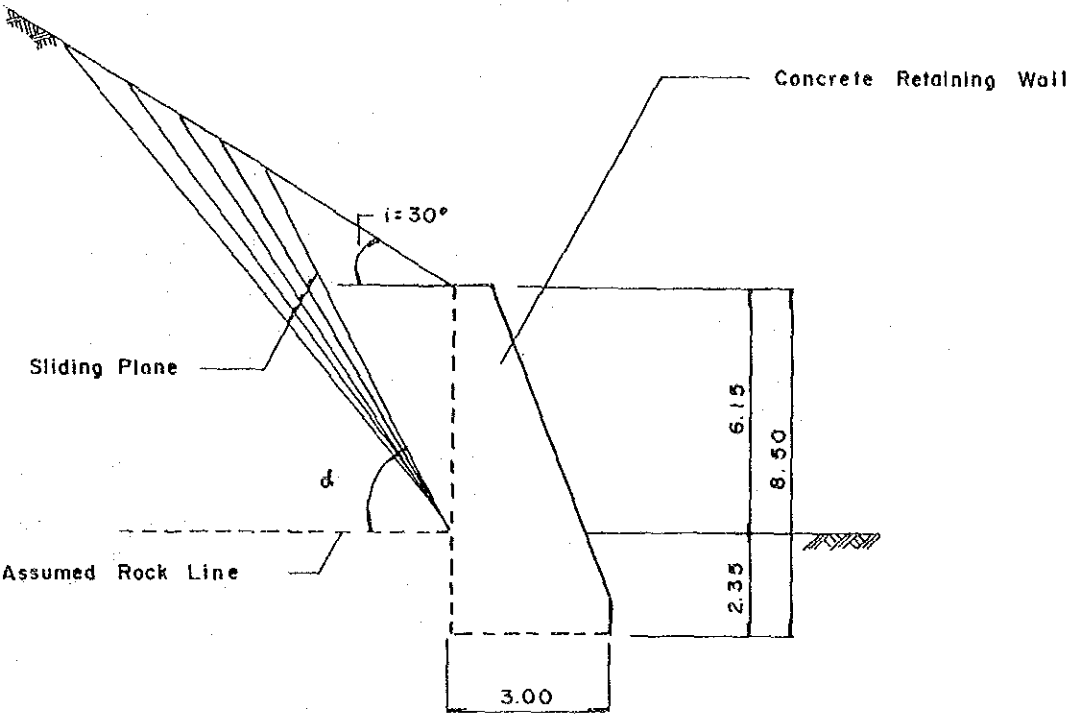
$$q2 = (14.05/1.9)(1 - 6 \times 0.03/1.9) = 6.7 \text{ t/m2}$$

$$14.05 = W + Pv$$

$$qd = 30 \text{ t/m2} \quad \text{(Refer to Table 3-5)}$$

3.7 EXAMPLE CALCULATION OF GRAVITY TYPE BY TRIAL METHOD WITH WEDGE SHAPE

3.7.1 Dimension of Slope



α = 44°, 48°, 52°, 56°, 60°

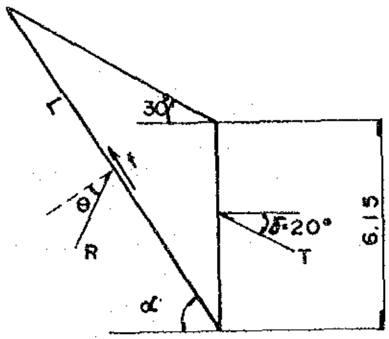
3.7.2 Characteristics of Back Soil

Unit weight of soil	Γ	=	1.9 t/m3
Internal angle of friction	$\emptyset$	=	30°
Cohesion	C	=	1.0 t/m2
Proposed safety factor	F	=	1.3
Friction between wall and soil	f	=	1/2 x C = 0.5 t/m2
Angle of reaction force	δ	=	2/3 Ø = 20°

3.7.3 Computation of Earth Pressure

A : Area of Wedge

L : Length of Sliding Plane



W = A x Γ

K = C x L

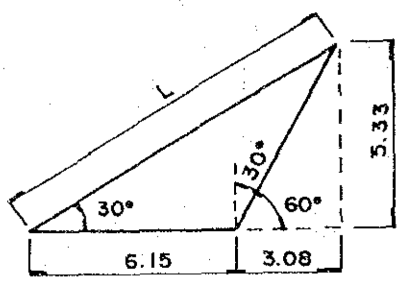
S = c/2 x H

R = Unknown

T = Unknown

W' = F x W

i) α = 60°



W = A x Γ = 16.39 m2 x 1.9 t/m3 = 31.14 t/m

K = C x L = 1.0 t/m2 x 10.68 m = 10.68 t/m

S = c/2 x H = 0.5 t/m2 x 6.15 m = 3.08 t/m

R = Unknown

T = Unknown

W' = F x W = 1.3 x 31.14 t/m = 40.48 t/m

A = 16.39 m2

$$\cos 30° = \frac{6.15 + 3.08}{L}$$

$$L = \frac{6.15 + 3.08}{\cos 30°}$$

L = 10.68 m

ß = 30 + 30 = 60°

W' = K sin 60° + S + R sin ß + T sin 20° ---------------- (1)

K Cos 60° + T cos 20° = R cos ß -------------------- (2)

From Equation (1)

$$40.48 = 10.68 \sin 60° + 3.08 + R \sin 60° + T \sin 20°$$

$$28.15 = R \sin 60° + T \sin 20°$$

$$R \sin 60° = 28.15 - T \sin 20°$$

$$R = \frac{(28.15 - T \sin 20°)}{\sin 60°} \text{ ---------------- } (3)$$

In Equation (2)

$$10.68 \cos 60° + T \cos 20° = \frac{(28.15 - T \sin 20°)}{\sin 60°} \cos 60°$$

$$5.340 + 0.940T = 16.250 - 0.197T$$

$$1.137T = 10.910$$

$$T = 9.60$$

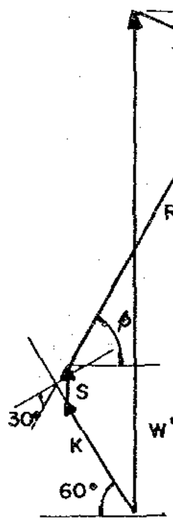
Substituting T in Equation (3)

$$R = \frac{28.150 - 9.60 \sin 20°}{\sin 60°}$$

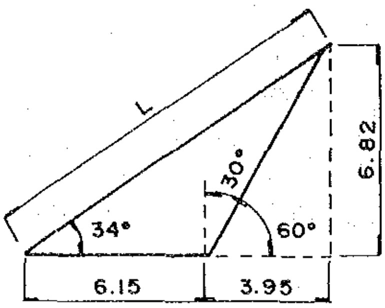
$$= \frac{24.87}{\sin 60°}$$

$$R = 28.720$$

VECTOR ANALYSIS



ii) $\alpha = 56°$



$W = 20.97 \times 1.9 = 39.84$ t/m

$K = 1 \times 12.18 = 12.18$ t/m

$S = 0.5 \times 6.15 = 3.08$ t/m

R = Unknown

T = Unknown

$W' = 1.3 \times 39.84 = 51.80$ t/m

$A = 20.97$ m2

$$L = \frac{6.150 + 3.950}{\text{Cos } 34°}$$

$L = 12.18$ m

$\beta = 30 + 34 = 64°$

$$W' = K \sin 56° + S + R \sin \beta + T \sin 20° \quad \text{---------- (1)}$$

$$K \cos 56° + T \cos 20° = R \cos \beta \quad \text{---------- (2)}$$

From Equation (1)

$$51.80 = 12.18 \sin 56° + 3.08 + R \sin 64° + T \sin 20°$$

$$38.62 = R \sin 64° + T \sin 20°$$

$$R = \frac{(38.62 - T \sin 20°)}{\sin 64°} \quad \text{---------- (3)}$$

In Equation (2)

$$12.18 \cos 56° + T \cos 20° = \frac{(38.62 - T \sin 20°)}{\sin 64°} \cos 64°$$

$$6.810 + 0.940T = 18.84 - 0.167T$$

$$1.11\, T = 12.03$$

$$T = 10.83$$

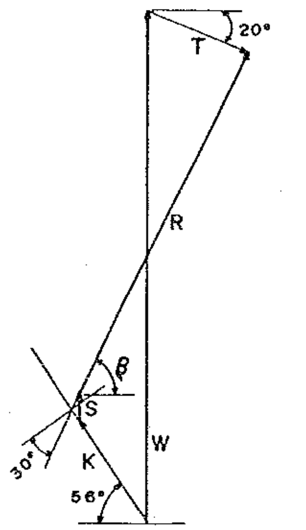
Substituting T in Equation (3)

$$R = \frac{(38.62 - 10.83 \sin 20°)}{\sin 64°}$$

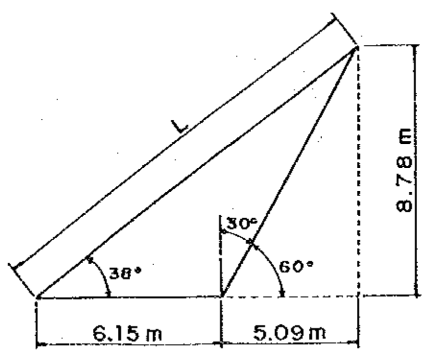
$$= \frac{34.92}{\sin 64°}$$

$$R = 38.85$$

VECTOR ANALYSIS



iii) $\alpha = 52°$



W = 26.9 x 1.9 = 51.11 t/m

K = 1 x 14.26 = 14.26 t/m

S = 0.5 x 6.15 = 3.08 t/m

R = Unknown

T = Unknown

W′ = 1.3 x 51.11 = 66.44 t/m

A = 26.90 m2

$$L = \frac{6.150 + 5.090}{\cos 38°}$$

L = 14.26 m

$\beta = 30° + 38° = 68°$

$W' = K \sin 52° + S + R \sin \beta + T \sin 20°$ ----------------- (1)

$K \cos 52° + T \cos 20° = R \cos \beta$ -------------------------- (2)

From Equation (1)

$$66.44 = 14.260 \sin 52° + 3.080 + R \sin 68° + T \sin 20$$

$$52.12 = R \sin 68° + T \sin 20°$$

$$R = \frac{52.12 - T \sin 20°}{\sin 68°} \text{ -------------------- } (3)$$

In Equation (2)

$$14.26 \cos 52° + T \cos 20° = \left(\frac{52.12 - T \sin 20°}{\sin 68°}\right) \cos 68°$$

$$8.78 + 0.940T = 21.06 - 0.138T$$

$$1.08T = 12.28$$

$$T = 11.39$$

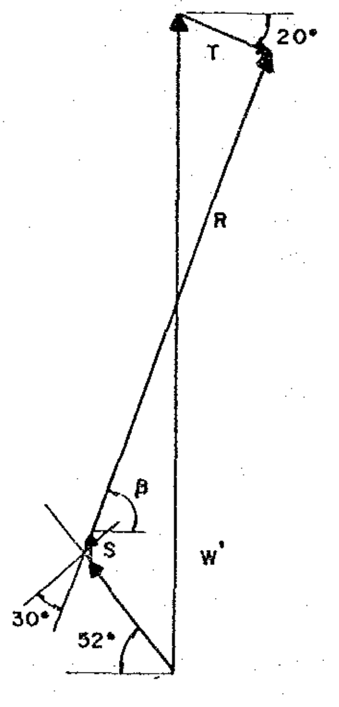
Substituting T in Equation (3)

$$R = \frac{52.12 - 11.39 \sin 20°}{\sin 68°}$$

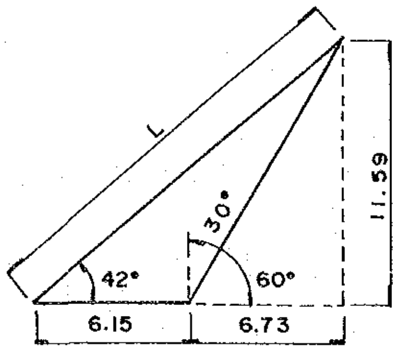
$$= \frac{48.230}{\sin 68°}$$

$$R = 52.02$$

VECTOR ANALYSIS



iv) $\alpha = 48°$



$W = 35.65 \times 1.9 = 67.73$ t/m

$K = 1.0 \times 17.33 = 17.33$ t/m

$S = 0.500 \times 6.160 = 3.08$ t/m

R = Unknown

T = Unknown

$W' = 1.3 \times 67.73 = 88.05$ t/m

$A = 35.65$ m2

$$L = \frac{6.150 + 6.730}{\text{Cos } 47°}$$

$L = 17.33$ m

$\beta = 30° + 42° = 72°$

$$W' = K \sin 48° + S + R \sin \beta + T \sin 20° \text{ ------------------ } (1)$$

$$K \cos 48° + T \cos 20° = R \cos \beta \text{ ------------------------------ } (2)$$

From Equation (1)

$$88.05 = 17.33 \sin 48° + 3.08 + R \sin 72° + T \sin 20°$$

$$72.10 = R \sin 72° \times T \sin 20°$$

$$R = \frac{(72.10 - T \sin 20°)}{\text{Sin } 72°} \text{ ------------------------------ } (3)$$

In Equation (2)

$$17.33 \cos 48° + T \cos 20° = \frac{(72.10 - T \sin 20°)}{\text{Sin } 72°} \text{Cos } 72°$$

$$11.60 + 0.940T = 23.430 - 0.110T$$

$$1.05T = 11.82$$

$$T = 11.26$$

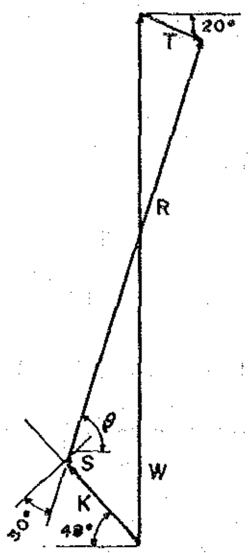
Substituting T in Equation (3)

$$R = \frac{72.10 - 11.26 \sin 20°}{\sin 72°}$$

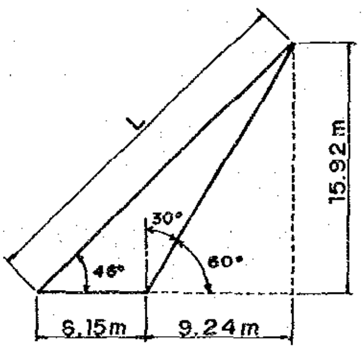
$$= \frac{68.25}{\sin 72°}$$

$$R = 71.76$$

VECTOR ANALYSIS



v) $\alpha = 44°$



W = 48.97 x 1.9 = 93.04 t/m

K = 1 x 22.15 = 22.15 t/m

S = 0.5 x 6.16 = 3.08 t/m

R = Unknown

T = Unknown

W' = 1.3 x 93.04 = 120.96 t/m

A = 48.970 m2

$$L = \frac{6.150 + 9.240}{\cos 46°}$$

L = 22.15

ß = 30 + 46 = 76°

W' = K sin 44° + S + R sin ß + T sin 20° ---------------- (1)

K cos 44° + T cos 20° = R cos ß --------------------------- (2)

From Equation (1)

$$120.95 = 22.15 \sin 44° + 3.08 + R \sin 76° + T \sin 20°$$

$$102.49 = R \sin 76° + T \sin 20°$$

$$R = \frac{102.94 - T \sin 20°}{\sin 76°} \text{ ------------------------------ } (3)$$

In Equation (2)

$$22.15 \cos 44° + T \cos 20° = \frac{(102.49 - T \sin 20°)}{\sin 76°} \cos 76°$$

$$15.93 + 0.940T = 25.55 - 0.085T$$

$$1.03T = 9.62$$

$$T = 9.39$$

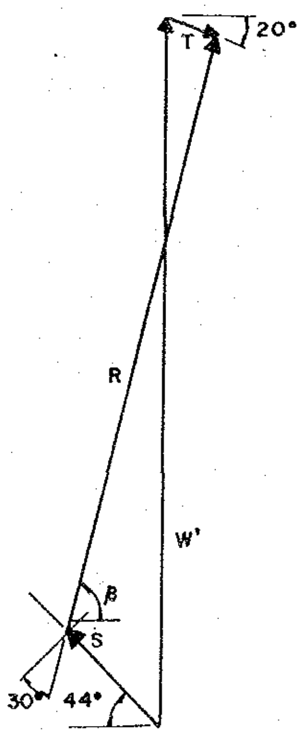
Substituting in Equation (3)

$$R = \frac{102.49 - 9.39 \sin 20°}{\sin 76°}$$

$$= \frac{99.28}{\sin.76°}$$

$$R = 102.32$$

VECTOR ANALYSIS

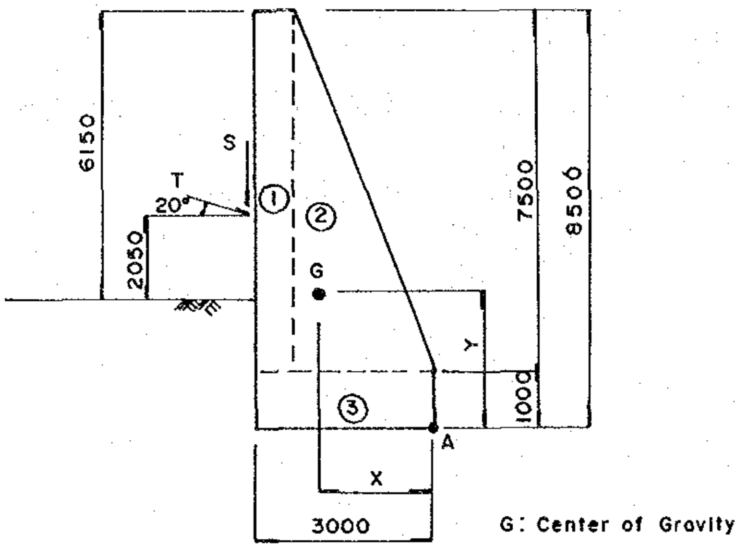


3.7.4 Computation of Resulting Forces at Point A

Critical Earth Pressure

$T = 11.39$ t/m $(\alpha = 52°)$

$S = 3.08$ t/m



Area:

$A1 = 0.5 \times 7.5 = 3.750$ m2

$A2 = 1/2 \times 2.5 \times 7.5 = 9.375$ m2

$A3 = 1.0 \times 3.0 = 3.000$ m2

$\Sigma A = 16.125$ m2

Mark	A (m^2)	X_o (m)	Y_o (m)	AX_o (m^3)	AY_o (m^3)
(1)	3.750	2.750	4.750	10.313	17.813
(2)	9.375	1.667	3.500	15.656	32.813
(3)	3.000	1.500	0.500	4.500	1.500
	Σ = 16.125			Σ = 30.469	Σ = 52.126

$$X = \frac{Axo}{\Sigma A} = \frac{30.469}{16.125} = 1.889 \text{ m}$$

$$Y = \frac{\Sigma Ayo}{\Sigma A} = \frac{52.126}{16.125} = 3.233 \text{ m}$$

Consider 1 m. strip:

Vertical Forces:

$V = \Gamma c \times A \times 1$

$V1 = 2.40$ t/m3 x 3.75 m2 x 1 m = 9.00 t

$V2 = 2.40 \times 9.375 \times 1 = 22.50$ t

$V3 = 2.40 \times 3.000 \times 1 = 7.20$ t

Concrete Weight, $VT = V1 + V2 + V3$

$VT = 38.70$ t

Earth Pressure, $S = 3.08$ t

Earth Pressure, $TY = T \sin 20°$

$= 11.39 \sin 20°$

$TY = 3.90$ t

Horizontal Forces:

Earth Pressure, $TX = T \cos 20°$

$= 11.39 \cos 20°$

$TX = 10.70$ t

Description	V	X	V · X	H	Y	H · Y
Earth Pressure, T	3.90	3.000	11.70	10.70	4.40	47.08
Earth Pressure, S	3.08	3.000	9.24	0	0	0
Concrete Weight	38.70	1.889	73.10	0	0	0
	Σ = 45.68		Σ = 94.04	Σ = 10.70		Σ = 47.08

$Mo = VX - HY$

$= 94.04 - 47.08$

$Mo = 46.96$ t.m

Total Resulting Forces at Point A

$V = 45.68$ t

$H = 10.70$ t

$M = 47.08$ t.m

3.7.5 Stability Check of Retaining Wall

1) Bearing capacity of soil; q max 30 t/m²

2) Overturning; resultant force with middle third

3) Sliding; SF > 1.5

$$e = \frac{M}{V} = \frac{49.96}{45.68} = 1.03 \text{ m.}$$

$$eo = \frac{B}{2} - e = 0.47 \text{ m} < \frac{B}{6} = 0.50 \text{ m}$$

$$\frac{q_{max}}{min} = (V/A) \pm (Mc/I)Y = (V/A) \pm (6\ Mc/B^2)$$

$$= \frac{45.68}{3.00} \pm \frac{6 \times 45.68 \times 0.47}{9}$$

$$= 15.23 \pm 14.31$$

$$q_{max} = 29.5 \text{ t/m2} < 30 \text{ t/m2}$$

$$q_{min} = 0.9 \text{ t/m2}$$

$$Sf = \frac{HR}{H} = \frac{V \times \mu}{H} \quad ; \mu = 0.6$$

$$= \frac{45.68 \times 0.6}{10.70}$$

$$Sf = 2.56 > 1.5$$

4. GROUTED RIPRAP RETAINING WALL

4.1 CALCULATION OF STABILITY

Stability analysis of the grouted riprap retaining wall is carried out in comparison with the gradient of the retaining wall and force direction line.

The equations for slope of riprap retaining wall are given as follows (See Figure 4-1):

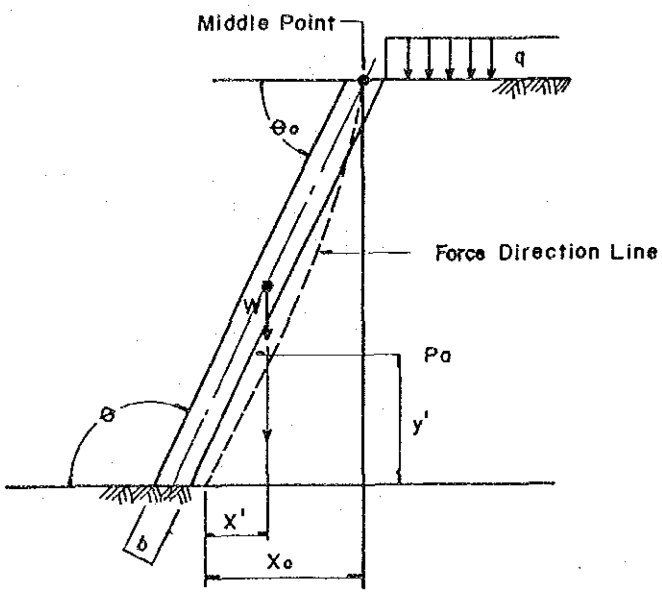


FIGURE 4-1 SLOPE STABILITY OF RIPRAP RETAINING WALL

$$P = (1/2\ \Gamma s y^2 + qy)\ Ka \qquad (1)$$

$$Ka = \frac{\sin^2(\theta + \phi)}{\sin\theta\ (\sqrt{\sin\theta} + \sqrt{\sin\phi\ \sin(\phi + \theta - 90°)})^2}$$

$$y' = \frac{qy/2 + ry^2/6}{qy + ry^2/2} \qquad (2)$$

$$W = gs\ by\ \sqrt{(1 + \cot^2\theta o)} \qquad (3)$$

$$xo = x' + (y \cot\theta o)/2 \qquad (4)$$

Where:

Pa : Total earth pressure acting on the wall (ton)

Ka : Earth pressure coefficient (See Table 4-1)

Γs : Unit weight of soil (t/m3)

ϕ : Internal friction angle of soil (degree) q : Surcharge load on the top of ground (t/m2)

θ : Slope angle of the wall (degree)

θo : 180 − θ (degree)

W : Weight of the wall (ton)

Γc : Unit weight of the wall (t/m3)

b : Thickness of the wall (m)

y' : Height of Pa from the base line (m)

xo : Distance of the resultant force (Pa and W) direction reaching point on the wall base line from the vertical line passing through the center of the wall top (m).

To maintain the equilibrium of the wall stability,

$$x' w = y' Pa$$

from the equations (1), (2) and (3)

$$x' = \frac{Ka\ (qy/2 + ry^2/6)}{\Gamma sb\ (\sqrt{1 + \cot^2 \theta o})}$$

Therefore, from the equation (4)

$$xo = \frac{Ka\ (qy/2+ry^2/6)}{\Gamma_B\ b\ (\sqrt{1+\cot^2\theta o})} + \frac{y \cot \theta o}{2} \quad \text{----------------} \quad (5)$$

is obtained. Equation (5) gives the force direction line for a given riprap retaining wall.

TABLE 4-1 VALUES FOR K

θ	90°	100° (0.2:1)	106°42' (0.3:1)	110°	111°48' (0.4:1)	116°34' (0.5:1)	120°	130°	140°
10°	0.70409	0.58685	0.52652	0.4995	0.4857	0.4490	0.4370	0.351	0.275
20°	0.49029	0.38530	0.32784	0.3019	0.2882	0.2534	0.2304	0.163	0.097
30°	0.33330	0.24508	0.19623	0.1743	0.1628	0.1339	0.1144	0.063	0.061
35°	0.27099	0.19128	0.14876	0.1279	0.1177	0.0924	0.0756	0.033	0.005
40°	0.21744	0.14619	0.10742	0.0904	0.08165	0.0602	0.0463	0.014	-
45°	0.15157	0.10866	0.07509	0.0697	0.05337	0.0359	0.0250	0.004	-

4.2 EXAMPLE CALCULATION OF GROUTED RIPRAP

Design data for calculation of riprap retaining wall

$H = 6.0$ m, $b = 0.5$ m, $gc = 2.30$ t/m3, $\theta = 116°34'$ (0.5:1)

$\Gamma s = 1.8$ t/m3, $\emptyset = 30°$ and $q = 0$ (no surchange road)

Calculation

From equation (5),

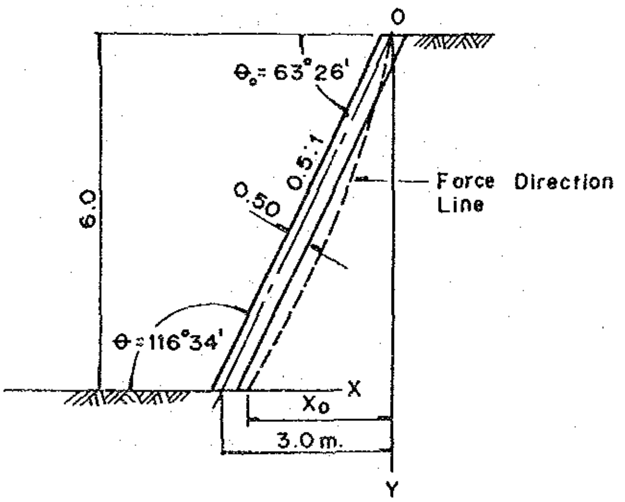
$$Xo = Ka\,(\Gamma s \cdot y^2/6)/\Gamma c \cdot b\,(\sqrt{1 + \cot^2 \Theta o}) + (\cot \Theta o/2)y$$

$Ka = 0.1339$ for $\emptyset = 30°$ and $\theta = 116°\ 34'$ ($\theta o = 63°24'$)

$$Xo = 0.1339 \times (1800/6)y^2/(2300 \times 0.5 \times \sqrt{1 + \cot^2 63°\ 26'}) + (0.5/2)y$$

$$= 0.0312\ y^2 + 0.25\ y$$

We calculated X_o values and the force direction line are shown in Figure 4-2. Point X_o is located behind the retaining wall. Therefore, this retaining wall with H = 6.0 m and slope gradient of 0.5:1 has safety for slope stability.



y (m)	x_o (m)	0.5 y (m)
0	0	0
1	0.281	< 0.50
2	0.627	< 1.00
3	1.036	< 1.50
4	1.499	< 2.00
5	2.030	< 2.50
6	2.632	< 3.00

FIGURE 4.2 RIPRAP RETAINING WALL WITH 0.5:1 SLOPE

5. MAT GABION WALL

5.1 CALCULATION OF STABILITY

Calculation of stability should be carried out by similar way of popular gravity type structures (e.g. gravity type retaining wall, abutment, etc), but in this case it is necessary to check the strength of wire net and skeleton steel bar which are supporting whole structure of gabion and sometimes deformed by shearing force.

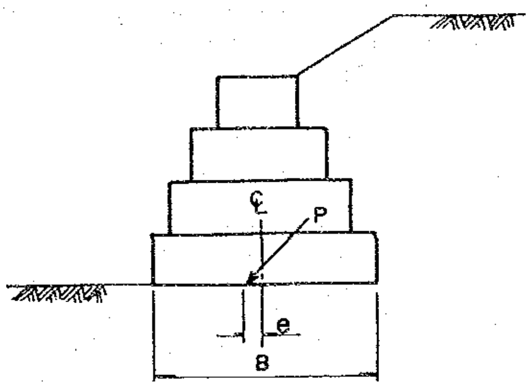
5.1.1 Stability on Overturning

The Requirement of stability is as follows.

Ordinary Condition $e \leq \frac{B}{6}$

Seismic Condition $e \leq \frac{B}{3}$

$$e = \frac{B}{2} \quad \frac{\Sigma NX - \Sigma HY}{\Sigma N}$$



Where:

B : Width of Bottom

e : Eccentricity from acting point of concentrated force

5.1.2 Stability on Bearing Capacity

The requirement of stability is as follows:

Ordinary Condition $q1$ or $q2 \leq Qa$

Seismic Condition $q1$ or $q2 \leq 1.5\ Qa$

Where: $q1, q2 = (\Sigma N/B)\ (1 + 6e/B)$

B : Width of bottom

e : Eccentricity from acting point of concentrated force

q1, q2 : Reacting forth of ground at both ends of wall

Qa : Allowable supporting forth of the ground

5.1.3 Stability on Sliding

The Requirement of stability is as follows:

Ordinary Condition $Fs \geq 1.5$

Seismic Conditin $Fs \geq 1.2$

$$Fs = (\Sigma N \cdot \mu + C \cdot B)/\Sigma H$$

Where:

Fs :	Factor of safety	ΣN :	Sum of vertical force
μ :	Coefficient of friction between bottom surface and ground surface	ΣH :	Sum of horizontal force
C :	Cohesion of soil under ground surface		
B :	Width of bottom		

5.1.4 Stability of Wire Net and Supporting Steel Bars

1) Allowable Tensile Strength of Wire Net

. tensile strength of wire coated with zinc 30~55 kgf/mm2

. on this account, allowable tensile stress is:

$$ba = 1/3 \times 30 = 10 \text{ kg/mm2} = 1000 \text{ kg/cm2}$$

. Because of thickness of wire for mat gabion is Ø 8.0 mm, allowable tensile forth is:

$$Ta = A \cdot ba = 0.8^2 \times (\pi/4)\ 1000 = 500 \text{ kg/each}$$

2) Allowable Bending Moment of Supporting Steel Bars

. The quality of supporting steel bars are usually same bars for reinforced concrete (i.e. SR 30 in JIS Standard)

Allowable bending and tensile stress of bars

$$ba = 1600 \text{ kg/m2}$$

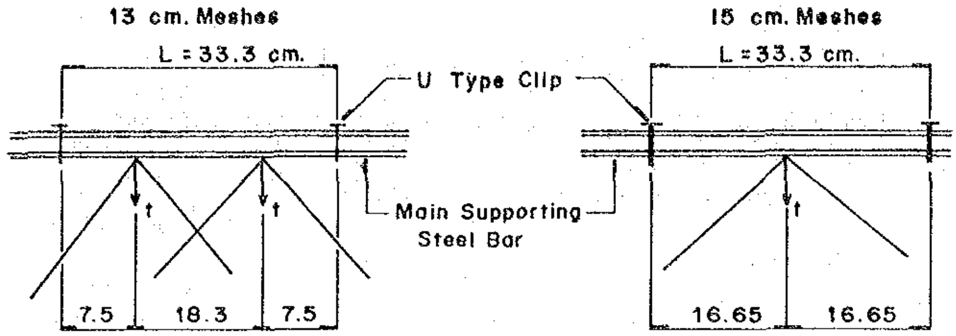
$$Ma = Z \cdot ba = 0.098\ d^3 \cdot ba$$

when Ø 13 is used, $Ma = 0.098 \times 1.33 \times 1600 = 344$ kg/cm

when Ø 16 is used, $Ma = 0.098 \times 1.63 \times 1600 = 642$ kg/cm

Two kinds of wire net are used, one is 13 cm meshes and another is 15 cm meshe.

3) Relation between Wire Net and Supporting Bars



for 13 cm meshe   M max = $\underline{t}$. a (a+b) 1/L = 5.81 t

for 15 cm meshe   M max = 1/8 . t . L = 4.16 t

Tensile forth by earth pressure acts mainly to supporting bars through wire net.

for 13 cm meshe   t = Ma/Mmax = Ma/5.81

Ø 13 t = 344/5.81 = 59 kg -- 59 x 2 x (100/33.3) = 354 kg/m

Ø 16 t = 642/5.81 = 110 kg --- 110 x 2 x 100/33.3) = 660 kg/m

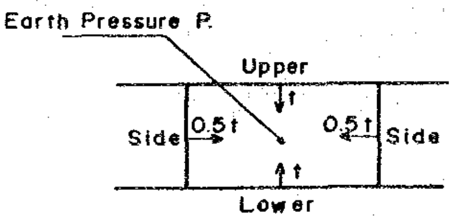
for 15 cm meshe   t = Ma/Mmax = Ma/4.16

Ø 13 t = 344/4.16 = 83 kg ---- 83 x 1 x (100/33.3) = 249 kg/m

Ø 16 t = 642/4.16 = 154 kg --- 154 x 1 x (100/33.3) = 462 kg/m

4) Maximum Earth Pressure

Earth pressure will be dispersed to supporting bars through wire net, and it will be distributed to three directions, i.e, upper, lower and side bars.



SUPPORTING BAR

Meshe \ Bar	ϕ13	ϕ16
13 cm	1.062 t/m	1.980 t/m
15 cm	0.747 t/m	1.386 t/m

5) Earth Pressure Acting to Each Gabion

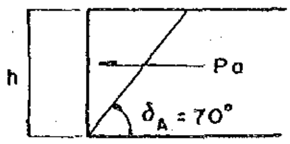
Unit weight of filling stones $\Gamma d = 2.0$ t/m3

Internal friction angle of filling stones $\phi = 40°$

Inclination angle of filling stones when it break down will be assumed $\delta A = 70°$

From above factors, earth pressure Pai is as follows:

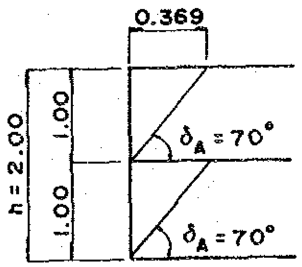
$$Pai = 1/2 \cdot \Gamma d \cdot h^2 \cdot \cot 70°$$



when h = 1.00 m Pa1 = ½ x 2.0 x 1.002 x cot 70° = 0.364 t/m

h = 1.50 m Pa1 = ½ x 2.0 x 1.502 x cot 70° = 0.819 t/m

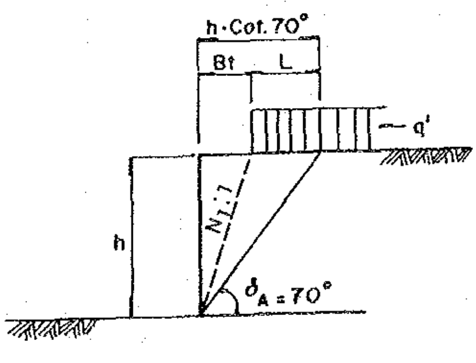
In the case of h = 2.00 m high, that is double layer of h = 1.00 m gabion, upper bars of lower gabion act as anchor bars.



Pa1 = (½ x 2.0 x 1.00^2 x cot 70°) + (2.0 x 1.0 x 0.364)

= 1.092 t/m

6) Earth Pressure by Uniform Surcharge on Backfill of Wall



Earth pressure by uniform surcharge acts as Figure Left so earth pressure Pa2 is as follows

$$Pa2 = q' (h \cdot \cot 70° - Bt) \text{ or } q' \cdot L$$

7) Allowable Uniform Surcharge

when h = 1.00 m

Diameter of Bars	Meshes of Net	Allowable Earth Pressure (P)	Earth Pressure to Wall (P_{a1})	P_{a2} = q'L	Remarks
φ13	13 cm	1.062 t	0.364 t	0.698 t	N_1 = 0.25 (B = 0.25) q'≤ 6.12 t/m^2 applicable
	15 cm	0.747 t	0.364 t	0.383 t	*N_1 = 0.25 (B = 0.25) q'≤3.37 t/m^2 applicable
φ16	13 cm	1.980 t	0.364 t	1.616 t	N_1 = 0.25 (B = 0.25) q'≤14.17 t/m^2 applicable
	15 cm	1.386 t	0.364 t	1.022 t	N_1 = 0.25 (B = 0.25) q'≤8.96 t/m^2 applicable

when front side inclination is N1 = 0.25 L = 0.114 m
N1 $\geq$ 0.364 L = 0

* q' : uniform surcharge
L : width of uniform surcharge acting as earth pressure

when h = 1.50 m

Diameter of Bars	Meshes of Net	Allowable Earth Pressure (P)	Earth Pressure to Wall (P_{a1})	P_{a2} = q'L	Remarks
φ13	13 cm	1.062 t	0.819 t	0.243 t	N_1 = 0.33 (Bt = 0.5) q'≤ 5.28 t/m^2 applicable
	15 cm	0.747 t	0.819 t	-	Not Applicable
φ16	13 cm	1.980 t	0.819 t	1.161 t	N_1 = 0.33 (Bt = 0.50) q'≤25.23 t/m^2 applicable
	15 cm	1.386 t	0.819 t	0.567 t	N_1 = 0.33 (Bt = 0.50) q'=12.32 t/m^2 applicable

when front side inclination is N1 = 0.33 L = 0.064 m
N1 = 0.364 L = 0

when h = 2.0 m

Diameter of Bars	Meshes of Net	Allowable Earth Pressure (P)	Earth Pressure to Wall (P_{a1})	P_{a2} = q' L	Remarks
φ13	13 cm	1.062 t	1.092 t	-	Not applicable
	15 cm	0.747 t	1.092 t	-	Not applicable
φ16	13 cm	1.980 t	1.092 t	0.888 t	N_1 = 0.125 (Bt = 0.25) q'≤7.79 t/m^2 applicable
	13 cm	1.386 t	1.092 t	0.294	N_1 = 0.125 (Bt = 0.25) q'≤2.57 t/m^2 applicable

when front side inclination is N1 ≥ 0.182 L = 0

8) How to investigate safety or stability

At first, calculate the reaction force at the bottom surface of mat gabion which is setting on the lowest layer of wall and investigate whether its value is under allowable uniform surcharge or not. But in case of L = 0 (h = 1.0 m or 1.50 m and N_1 0.364, h = 2.0 m N_1 0.182) it is not necessary to investigate except in case of special reasons thereon.

5.1.5 Deformation by Shearing Force

Stability on deformation by shearing force can be checked by following method. However, it is unnecessary to check in case that the friction coefficient of the ground surface is bigger than the friction coefficient of gablon. (i.e. tan 0.8).

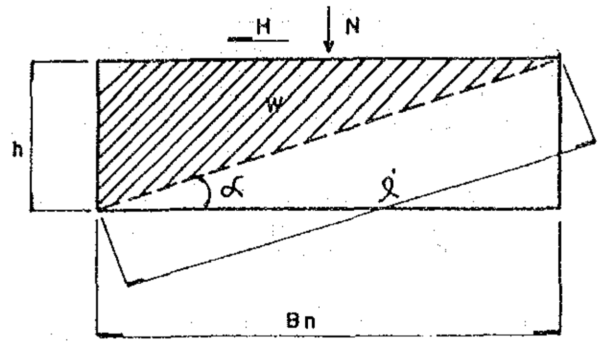
$$Fs' = R/Ro \geq 1.2$$

Where:

$$R = \{ (N + W) \cos \alpha + H \cdot \sin\alpha\} \tan \emptyset$$

$$Ro = (N + W) \sin \alpha + H \cos \alpha$$

$\emptyset$ = Internal friction angle (degree)



5.2 EXAMPLE CALCULATION OF MAT GABION

5.2.1 Design Scheme and Configuration

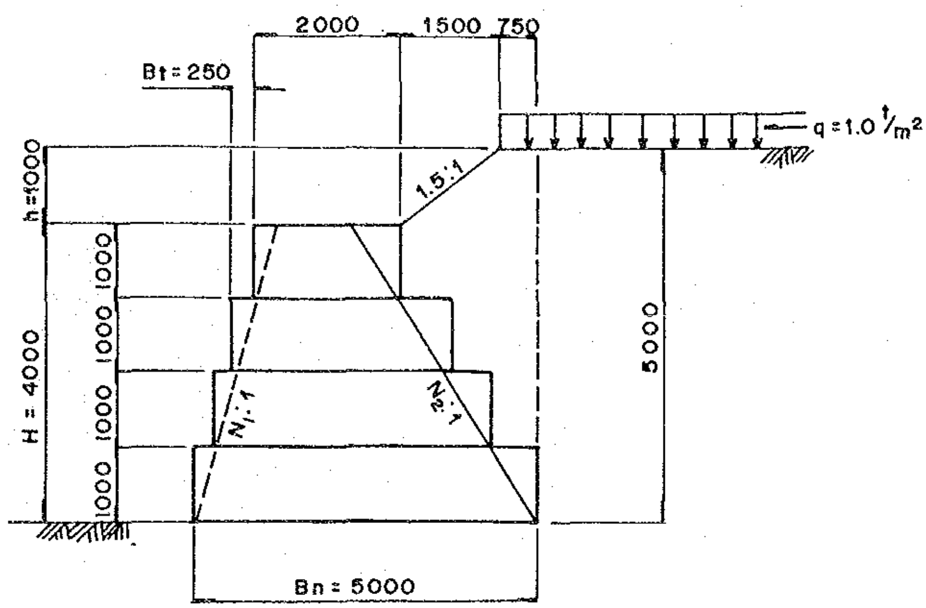


FIGURE 5-1 CROSS SECTION

- Backfill soil properties — Unit weight: Γ = 1.9 t/m3; Internal friction angle: Ø = 25°; Cohesion of soil: c = 0

- Earth pressure calculation — Coulomb's Theory

  Friction angle between soil and wall: δ = Ø = 25°

- Pore pressure calculation — Nothing

- Surcharge on backfill — q = 1.0 t/m2

- Examining of behavior during earthquakes — Nothing

- Soil properties of foundation ground — Unit weight of soil: δ_G = 1.9 t/m3; Unit weight of soil below underground water level: δ_G = 0.9 t/m3

- Foundation ground — Internal friction angle: ØG = 25°; Cohesion of Soil: CG = 0; Friction coefficient of bottom surface: μ = tan Ø = 0.466

Calculation can be carried out in same manner of that for gravity retaining wall as illustrated in Figure 5-2.

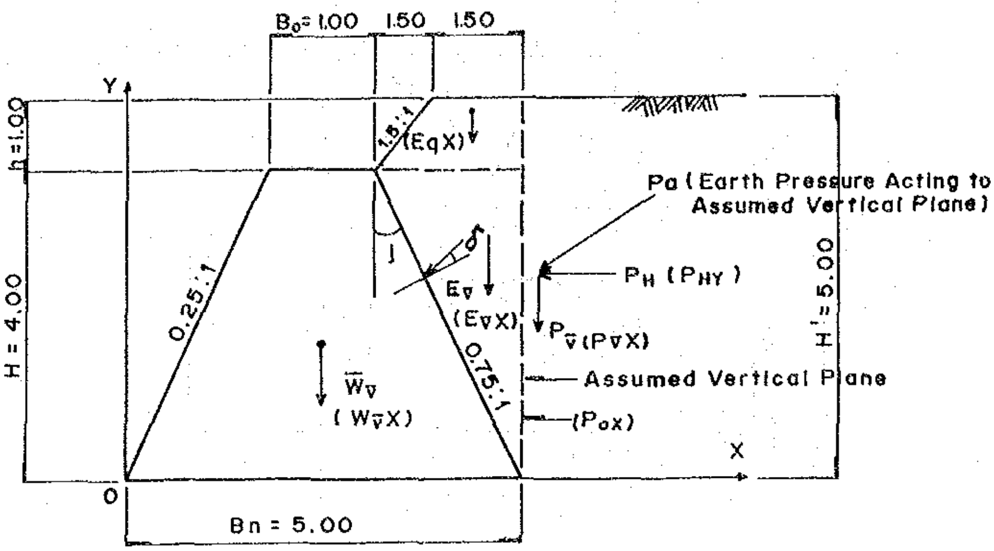


FIGURE 5-2 ILLUSTRATION OF GABION RETAINING WALL

5.2.2 The Summation of Vertical Forces

ΣN = Wv + Ev + Eq + Pv

When h = 1.00 m H = 4.00 m N1 = 0.25 N2 = 0.75

Bf = 0.25 Bo = 1.00 m Bn = 5.00 m Γ(Soil) = 1.9 t/m2

Wv = 24.00 t.m Ev = 11.40 t.m Eq = 4.28 + 1.50

Wvx = 50.67 t.m Evx = 45.60 t.m = 5.78 t/m

Eqx = 16.39 t.m

From Coulomb's equation earth pressure acting to assumed vertical plane is given.

$$Pa = \tfrac{1}{2} \cdot KA \quad H'^2 + q \cdot h \cdot KA$$

and

$$KA = \cos^2 (\varnothing - j)/\cos^2 j \cdot \cos (j + \delta) \cdot (1 + F)^2$$

$$F = \sqrt{\sin (\varnothing + \delta)} \cdot \sin (\varnothing - \beta)/\cos (j + \delta) \cdot \cos (j - \beta)$$

Where:

$\delta = \beta = \tan (1/1.5) = 33.66°$

$j = 36.87°$ $\qquad \varnothing - \beta = -8.66°$

$\varnothing - j = - 11.87°$ $\qquad j - \beta = 3.21°$

$\varnothing + \delta = 58.66°$

$$KA = 0.47/0.64 \times 0.33 \times (1 + 1.48)^2 = 0.363$$

$$Pa = \tfrac{1}{2} \times 0.363 \times 1.9 \times 5^2 + 1.0 \times 1.0 \times 0.363 = 8.98 \text{ t/m}$$

Earth Pressure distribution

$Pv = \sin 25° \times 8.98 = 3.76$ t/m

$PH = \cos 25° \times 8.98 = 8.14$ t/m

$\Sigma N = 24.00 + 11.44 + 5.78 + 3.76 = 44.98$ t

- Equilibrium of Moment by Vertical Forces

$$\Sigma NX = WV.X + Evx + Eqx + Pvx$$

$$= 50.67 + 45.60 + 16.39 + (3.76 \times 5.00)$$

$$= 131.46 \text{ t.m}$$

5.2.3 The Summation of Horizontal Forces

$\Sigma H = PH + Po = 8.14 + 0 = 8.14$ t

- Equilibrium of Moment by Horizontal Forces

$\Sigma HY = \Sigma H \times H = 8.14 \times 1.84 = 14.98$ t . m

$$H = 5.00 + \frac{1.0 \text{ t/m2}}{1.9 \text{ t/m3}} \times 1/3 = 1.84 \text{ m}$$

5.2.4 Calculation of Stability

- Stability on Overturning

Eccentricity from the center of bottom at acting point of summation forces; e

$$e = (Bn/2) - (\Sigma NX - \Sigma HY)/\Sigma N$$
$$= 5.00/2 - (131.46 - 14.98)/44.98 = -0.09 \text{ m} < B/6 = 0.83 \text{ m}$$

Stability against overturning is sufficient.

- Stability on Bearing

Allowable foundation pressure : qa

Maximum foundation bearing capacity : qu
is given by pradle's equation

$$qd = C \cdot Nc + \Gamma 1 \cdot Df \cdot Nq + 1/2\ \Gamma 2 \cdot \beta \cdot N$$

Where:

c : Cohesion of soil below foundation

Ø : Inner friction angle of soil below foundation

Γ' : Unit weight of soil above foundation

Γ2 : Unit weight of soil below foundation

Df : Penetration depth of foundation

B : Width of foundation

Nc, Nq, Nr : Pradle's bearign force coefficient

and now, from bearing force coefficient table.

When Ø = 25°   Nc = 20.72,   Nq = 10.66,   Nr = 10.88

$$qd = C \cdot Nc + \Gamma 1 \cdot Df \cdot Nq + 1/2\ \Gamma 2 \cdot \beta \cdot Nr$$
$$= 1/2 \times 1.9 \times 5.0 \times 10.88$$
$$= 51.68 \text{ t/m2}$$

Weight of soil expeled by retaining wall is negligible in this case.

i,e.

$$qa = qd/Fs = 51.68/3 = 17.2\ t/m2$$

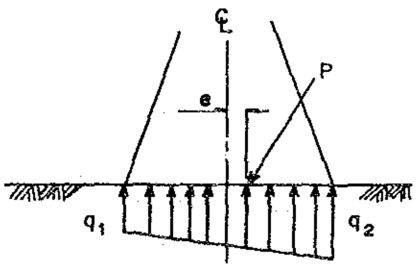
Where:

The factor of safety Fs = 3 (Ordinary Condition)

now foundation pressure (Reaction of Ground)

$$q1 \cdot q2 = \Sigma n/\beta n\ (1 \pm 6.e/\beta n) = (44.98/5)(1 \pm 6 \times -0.09/5)$$

$$= 8.02,\ 9.97 < 17.2\ t/m2$$



i.e

Stability against subsidence is sufficient.

- Stability on Sliding

The factor of safety Fs $\geq$ 1.5 (Ordinary Condition)

and

$$Fs = \Sigma N \cdot \mu + CG \cdot Bn/\Sigma H = (44.98 \times 0.466)/8.14 = 2.6 > 1.5$$

Where:

ΣN : Sum of vertical forces

ΣH : Sum of horizontal forces

CG : Cohesion of soil below foundation

Bn : Width of foundation

μ : Friction coefficient of foundation surface

- Stability of wire net and supporting steel bars

Earth pressure acting to each gabion

$Pa_1 = 1/2 \cdot \Gamma d \cdot h^2 \cdot \cot 70° = \frac{1}{2} \times 2.0 \times 1.00^2 \times \cot 70° = 0.364$ t/m

Where:

Γd : Unit weight of filling stones in gabion

h : Height of each gabion

Allowable load on gabion

$Pa2 = q' \cdot L = 6.12$ t/m2 x 0.114 m = 0.70 t/m2

Where:

q' : Uniform surcharge

L : Width of uniform surcharge acting as extra earth pressure

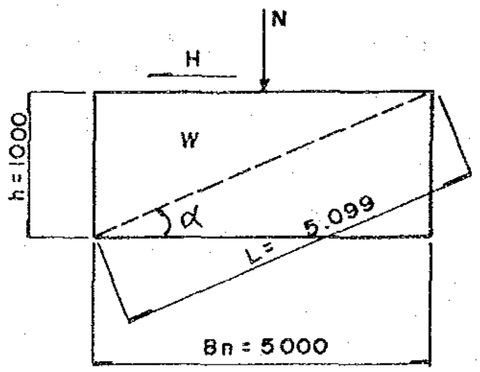
when Meshes of wire net are 15 cm and diameter of supporting steel bars are Ø 16.

Pa2 = 1.022 t/m2 > 0.70 t/m2

Meshes of wire net 15 cm

supporting steel bars Ø are applicable

Investigation of deformation by shearing force.



W = 1.00 x 5.00 x 2.0 = 10 t/m

N = ΣN-W = 44.98 - 10 = 34.98 t/m

H = 8.14 t

$\alpha = 11.31°$ $(\tan \alpha = 0.2)$

$$R = \{ (N + W) \cos \alpha + H \cdot \sin \alpha \} \tan \varnothing$$

$$= \{ (44.98 \times 0.981 + (8.14 \times 0.196) \} \times 0.466$$

$$= 20.31 \text{ t}$$

$$Ro = (N + W) \sin \alpha + H \cdot \cos \alpha$$

$$= 44.98 \times 0.196 + 8.14 \times 0.981$$

$$= 16.80 \text{ t}$$

$$R/Ro = 20.31/16.80 = 1.21 > 1.2$$

6. ANCHORING

6.1 GENERAL

The strength of anchor is determined by the resistance of the anchor portion against the pull-out force and the elongation of pre-stressed steel. Failure of the anchor in case of rock with relatively large strength will be at the bedrock, while failure occurs at the ground in case of low strength soil such as sediment.

Structural detail of anchoring members is shown in Figure 6-1.

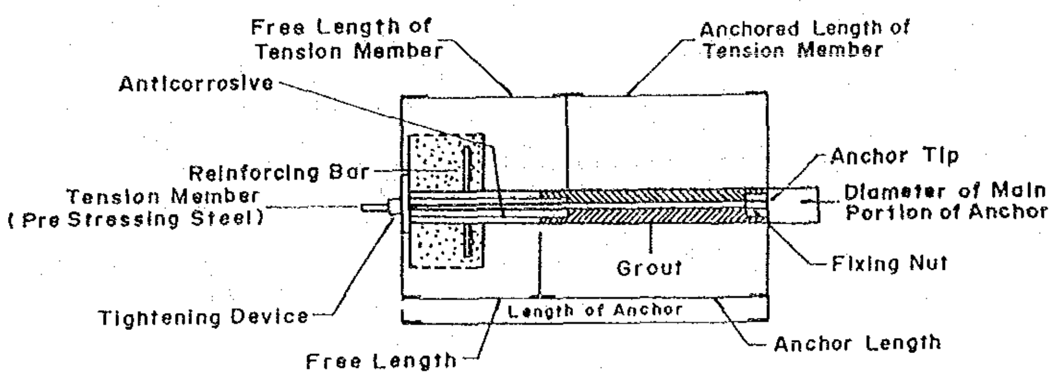


FIGURE 6-1 STRUCTURAL DETAIL OF ANCHORING MEMBER

Because of high tension acting against tension member of anchor, pre-stressed steels such as pre-stressed steel rod, pre-stressed steel strand and pre-stressed steel wire are generally used as materials for pre-stressing in order to reduce relaxation of pre-stressed steel.

6.2 ULTIMATE TENSILE RESISTANCE FORCE AND LENGTH OF ANCHOR

Ultimate tensile resistance force of the bonding portion between the anchor and the ground can be calculated from the following equation:

$$T = \pi D (L - Lf) \cdot \tau$$

Where:

T : Ultimate tensile resistance force of anchor (kg)

D : Diameter of main portion of anchor (cm)

L : Total length of anchor (cm)

Lf : Length of unanchored portion (cm)

τ : Pull-out shearing resistance between ground and main portion of anchor (kg/cm2)

If factor of safety F_s is determined, the length of anchor can be determined from the following equation:

$$L - Lf = FsT/\tau Dt$$

6.3 SHEARING RESISTANCE (τ)

Value of τ should be determined by the pull-out test. In the absence of test, τ may be approximately estimated based on values in Table 6.1.

TABLE 6.1 SHEARING RESISTANCE (τ) OF ANCHOR

Type of Ground			τ (kg/cm^2)
Rock	Hard Rock		15 ~ 25
	Soft Rock		10 ~ 15
	Weathered Rock		6 ~ 10
Gravel	N-Value	10	1.0 ~ 2.0
		20	1.7 ~ 2.5
		30	2.5 ~ 3.5
		40	3.5 ~ 4.5
		50	4.5 ~ 7.0
Sand	N-Value	10	1.0 ~ 1.4
		20	1.8 ~ 2.2
		30	2.3 ~ 2.7
		40	2.8 ~ 3.5
		50	3.0 ~ 4.0
Clayey Soil			1.0 C

Note: C = cohesion of clayey soil

6.4 FACTOR OF SAFETY

The following factor of safety may be recommended.

TABLE 6-2 FACTOR OF SAFETY (Fs) OF ANCHORING

Type of Structure and Load		F_s
Temporary Structure		1.5
Permanent Structure	Short Term Load	2.0
	Permanent Load	3.0

6.4.1 Bond Stress

Bond stress between pre-stressed steel and grouting material shall be checked in accordance with the method in analyzing bond stress between reinforcing bar and concrete.

6.4.2 Pre-stressed Steel

Tensile stress of pre-stressed steel shall be also be calculated, satisfying the following two conditions:

(a) $\sigma ap \leq 0.6\ \sigma up$

(b) $\sigma ap \leq 0.75\ \sigma yp$

Where:

σap : Allowable tensile stress of Pre-stressing steel (kg/cm2)

σup : Maximum tensile stress of steel (kg/cm2)

σyp : Yield stress of steel (kg/cm2)

7. ROCK SHED

7.1 GENERAL

A structure standing up to withstand the impact force must be designed estimating the absorption energy derived from elastic and plastic deformation of members and the ultimate supporting capacity of the structure. Since this method based on the theory of energy is not yet established for the design of huge structures such as rock shed, the simplified method may be used as described hereunder.

7.2 IMPACT LOAD

Impact load is calculated by the following rationale formula

$$P = 15.49 \, W^{2/3} \cdot H^{3/5} \cdot \alpha$$

Where:

P : Impact Force (t)

W : Weight of Falling Rock (t)

H : Height of Fall (m)

α : Factor of increase in accordance with thickness of absorption layer (h) h shall be 0.9 m so that a is 1.0.

7.2.1 Height of Fall

In case of free fall

$$H' = H$$

In case of rolling or sliding down

$$H' = (1 - \mu/\tan \Theta) \cdot H$$

Where:

H' : Height of fall for design

H : Vertical height of fall

Θ : Angle of slope

μ : Coefficient equivalent to friction of slope

7.2.2 Dispersion of Impact Load

Further in the design, the impact load may be assumed to act on the most influential point and be uniformly dispersed within the range of 45 degree by a shock absorbing material, as shown in Figure 7-1.

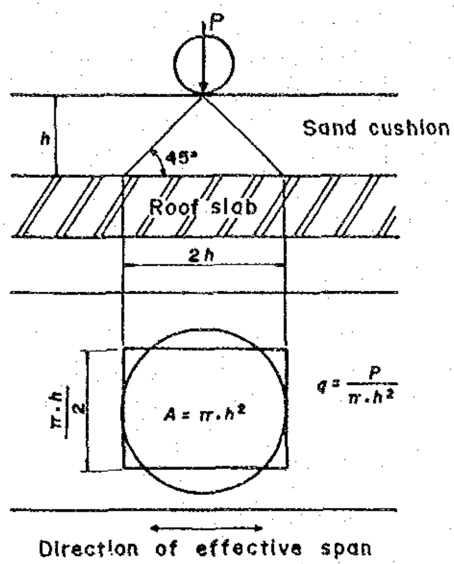


FIGURE 7-1 DISPERSION OF IMPACT LOAD

7.3 DEPOSITED MATERIALS

The weight of the collapsed sediment deposited on roof should be considered in the design. A 30-degree slope composed of collapsed sediment is assumed, as shown in Figure 7-2. However, absorbing effect by this material must also be considered. Therefore, this weight should not be considered with the impact force of falling rock at the same time.

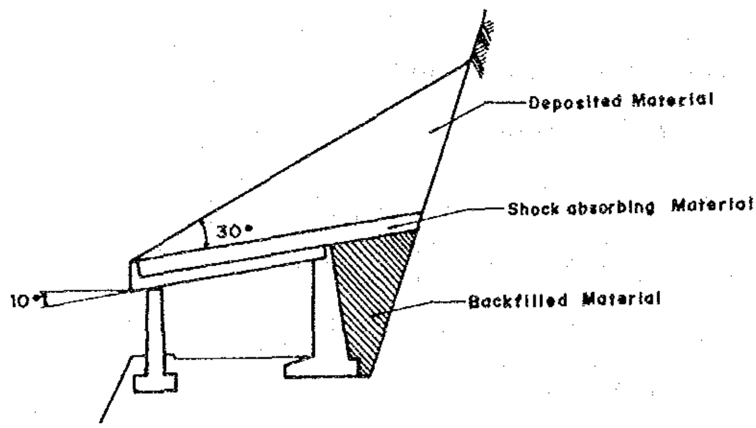


FIGURE 7-2 WEIGHT OF DEPOSITED MATERIALS

8. SABO DAM

8.1 GENERAL

Gravity type concrete sabo dam is designed using the same method applied to design of gravity type concrete retaining wall except external force acting on the sabo dam.

8.2 EXTERNAL FORCE

In design of sabo dam with less than 15 m high, analysis for normal case may not be required. However, an analysis during floods may be made considering only its own weight and the hydrostatic pressure as an external force.

On the other hand, a sabo dam whose height is more than 15 m, uplift, silt pressure (pressure due to deposited silt), seismic force should be taken into consideration aside from its own weight and hydrostatic pressure. For debris flow dam which may be directly hitted debris flow, impact force due to debris flow should be considered.

Hydrostatic pressure may be estimated using the following rationale formula.

$$P = 1/2 \cdot Wo \cdot Ho$$

Where:

P : Hydrostatic pressure (t/m^2)

Wo : Unit Weight of Water (t/m^3)

Ho : Water Depth (m)

8.3 VOLUME OF DEPOSITED DEBRIS

The volume of debris flowing with the current and sediments transported with bed load traction lane shall be estimated for design purposes as follows:

8.3.1 Estimation by Actual Survey

The volume may be estimated by actual survey in the field investigating the depth of stream and scouring due to flood.

8.3.2 Estimation by Past Experience

For water-shed wider than 3 km^2 the following equation is proposed based on past experiences:

$$Vs = H \times L \times B$$

Vs : A volume of debris and sediment (m3)

H : Average depth of stream bed scoured (m) H = 2 m is generally used as scoured depth

L : Length of torrent scoured (m)
$L = 3000 \times \sqrt{A10}$

A10 : Catchment Area whose gradient is more than 10 (km2)

B : Average width of torrent scoured (m)
$B = 3 \sqrt{Op}$.

Op : (1/3.6) . f . r . A10

f : Coefficient of run-off

f = 1.0 is generally used

r : Average Rainfall Intensity within the concentration time of flood (mm/hr)

8.3.3 Water-shed less than 3 km^2

In the equation mentioned above, total catchment area may be used instead of A_{10} to estimate the volume of debris and sediments.

8.3.4 Estimation by Kyoto University's Formula

This assumption is given by the Disaster Prevention Institute, Kyoto University, Japan as follows:

$$D = \alpha \, (A \cdot R24 - I200)^2$$

D : Volume of debris flow for one flood time (m3)

A : Catchment Area (km2)

R24: Maximum Daily Rainfall (mm)

I200: Gradient along the torrent between the deposited site and the site higher than 200 m as the deposited site. (See Figure 8-1).

α : Coefficient (α = 7 ~ 10)

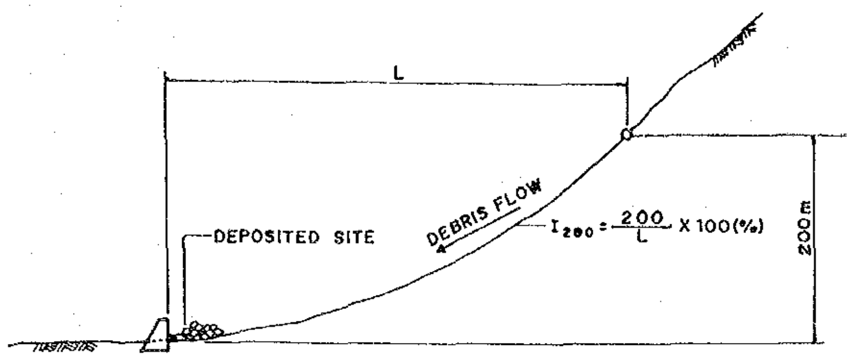


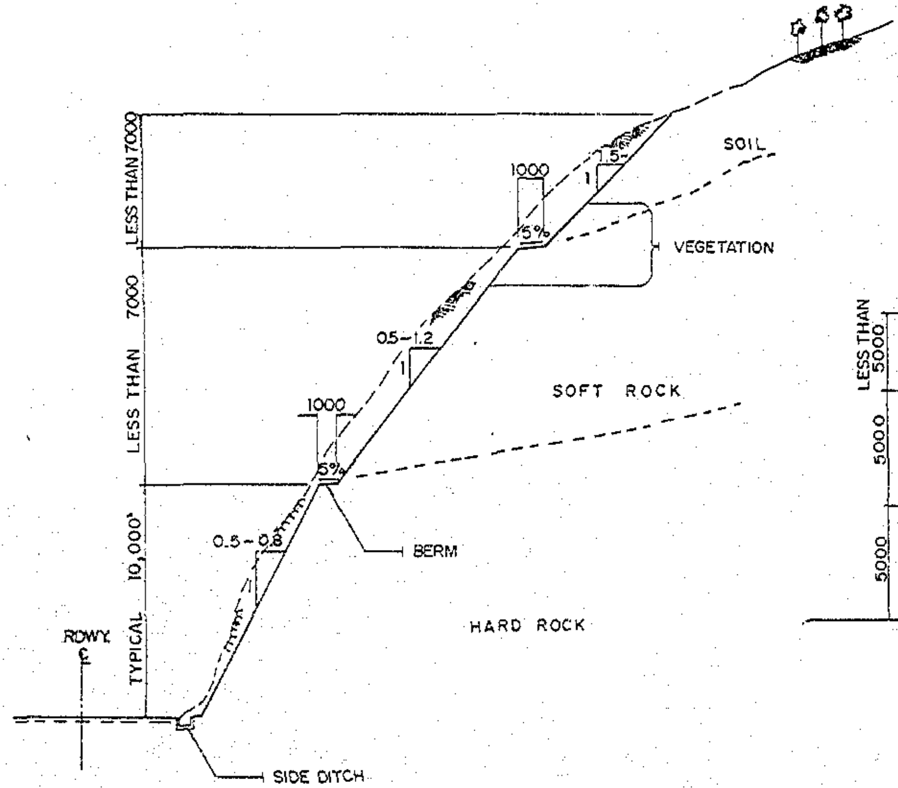
FIGURE 8-1 DEPOSITED SITE OF DEBRIS

APPENDIX II
STANDARD DRAWINGS

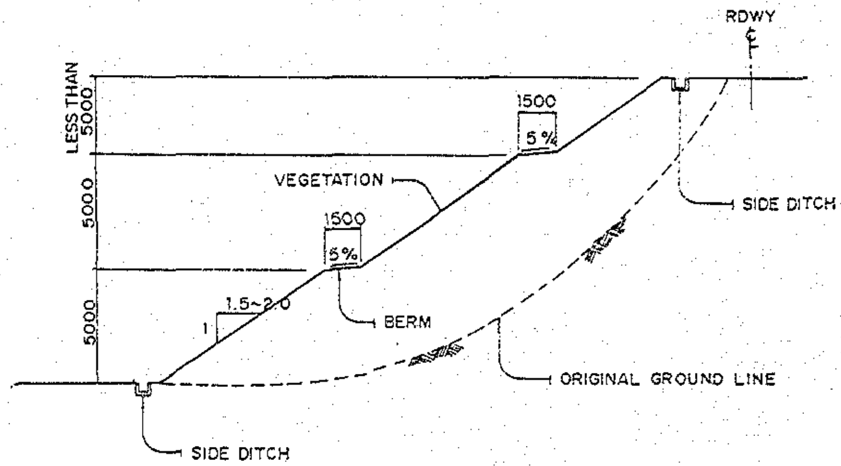
APPENDIX II
STANDARD DRAWINGS

TABLE OF CONTENTS

STANDARD DRAWINGS : CUT SLOPE, EMBANKMENT SLOPE	SCALE 1 : 20	DRAWING NO. 1



TYPICAL CROSS SECTION FOR CUT SLOPE
SCALE 1:20

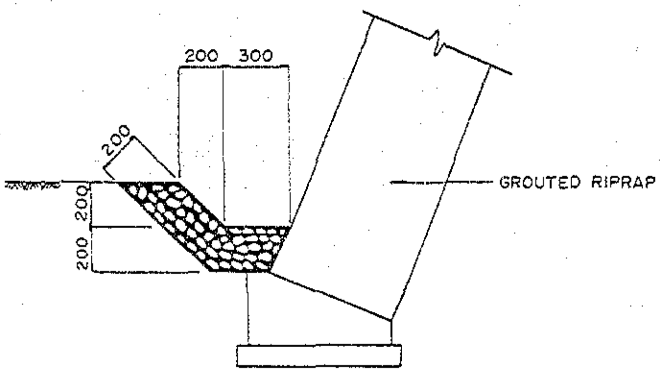


TYPICAL CROSS SECTION FOR EMBANKMENT SLOPE
SCALE 1:20

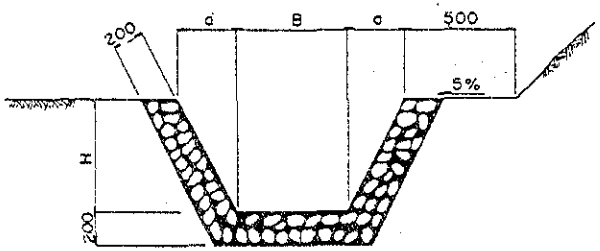
ITEM	KINDS OF ROCKS		
	SAND	SOFT ROCK	HARD ROCK
GRADIENT	MORE THAN 1.5	0.5 ~ 1.2 : 1	0.5 ~ 0.8 : 1
LOCATION OF BERM	LESS THAN 7.0 M	LESS THAN 7.0 M	TYPICAL 10.0 M

STANDARD DRAWINGS : SIDE DITCH , CATCH BASIN

SCALE AS SHOWN | DRAWING NO. 2



SIDE DITCH (TYPE-A)



SIDE DITCH (TYPE-B,C)

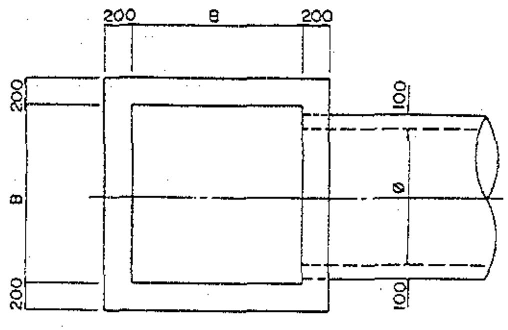
SIDE DITCH
SCALE 1:20

LIST OF UNIT VOLUME PER/M

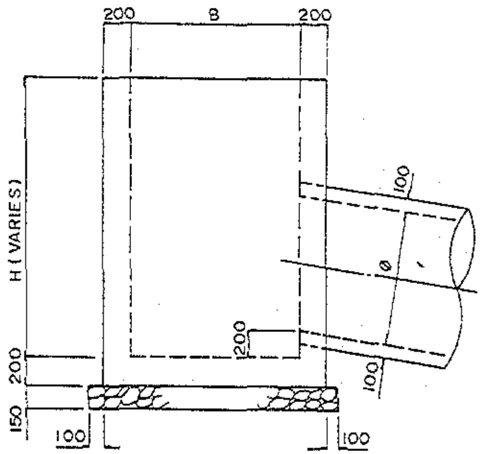
TYPE	A	B	C
VOLUME	0.14^{m^3}	0.27^{m^3}	0.40^{m^3}

LIST OF DIMENSION

TYPE	H	B	a
B	300	300	150
C	500	500	250



P L A N



ELEVATION

CATCH BASIN
SCALE 1:30

LIST OF DIMENSION

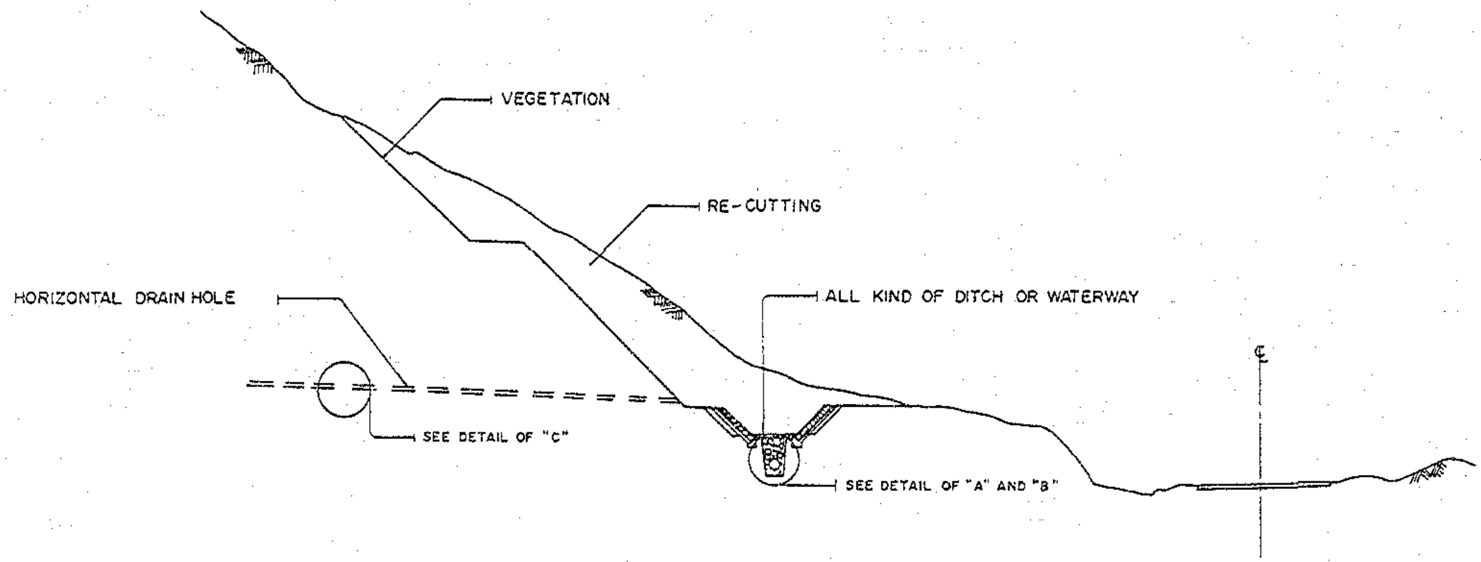
Ø	B
600	900
900	1200
1000	1300
1200	1500

LIST OF VOLUME

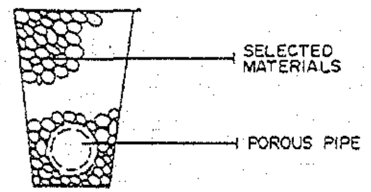
Ø	VOLUME
600	2.10^{m^3}
900	2.75^{m^3}
1000	2.98^{m^3}
1200	3.44^{m^3}

NOTE: In Case of $H=2.0^m$

STANDARD DRAWINGS :	SUBSURFACE DRAINER	SCALE	DRAWING NO. 3

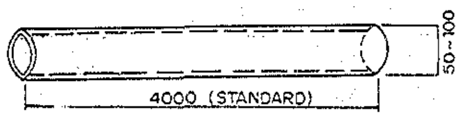


TYPICAL CROSS SECTION SUBSURFACE DRAINER



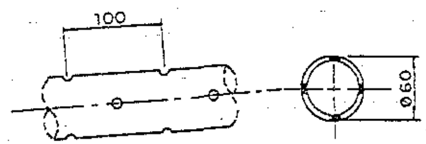
SUBSURFACE DRAINER

DETAIL OF "A"



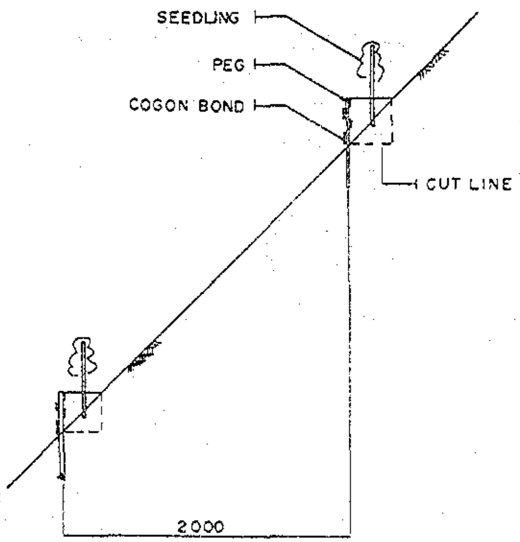
POROUS PIPE

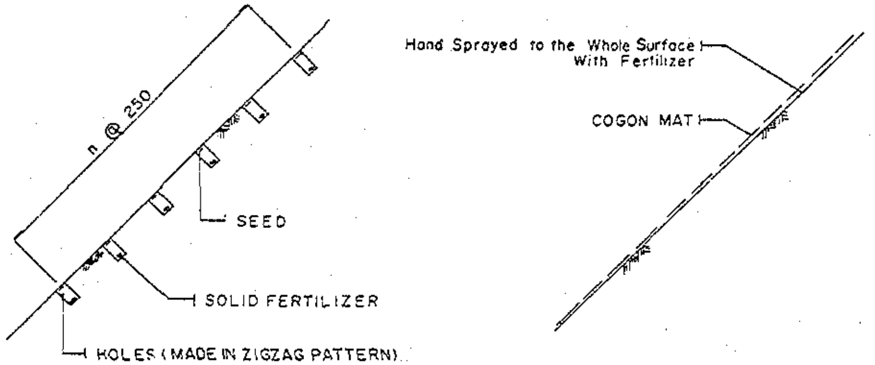
DETAIL OF "B"



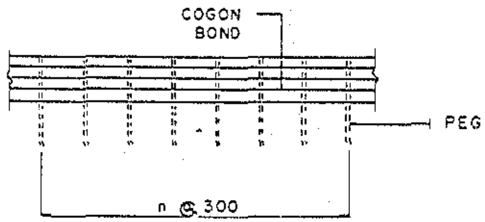
PERFORATED PIPE
(FOR HORIZONTAL DRAIN HOLE)
DETAIL OF "C"

STANDARD DRAWINGS : VEGETATION	SCALE AS SHOWN	DRAWING NO. 4

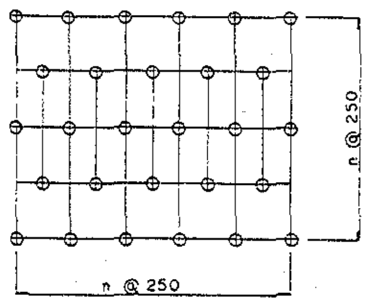




HAND SEEDING & HANDSEEDING WITH MAT

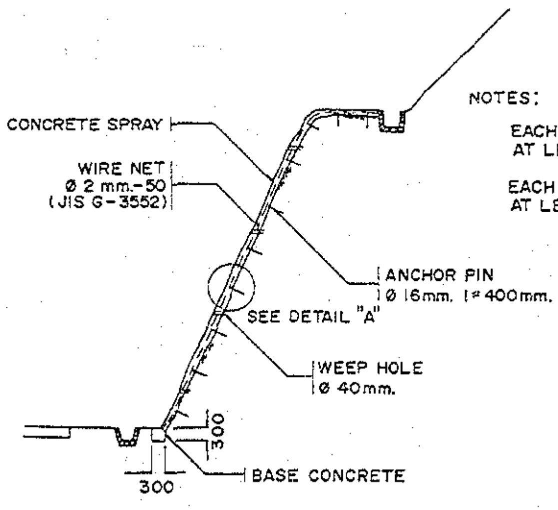


WATTLING
SCALE 1:30



PICK HOLE SEEDING
SCALE 1:20

STANDARD DRAWINGS :	CONCRETE SPRAYING , MORTAR SPRAYING	SCALE AS SHOWN	DRAWING NO. 5



CONCRETE SPRAYING

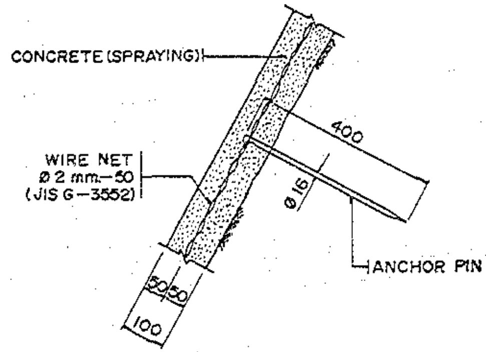
NOTES:

EACH ANCHOR PIN SHALL BE PLACED AT LEAST FOR EVERY 1 SQ.M.

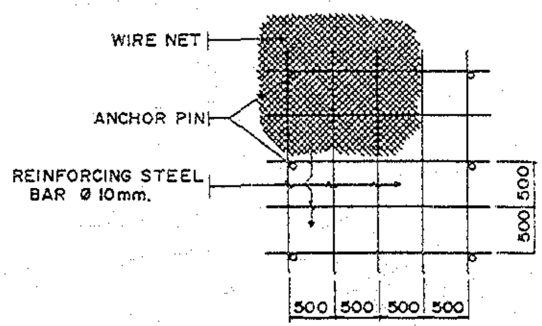
EACH WEEP HOLE SHALL BE PLACED AT LEAST FOR EVERY 2 SQ. M.

LIST OF UNIT MATERIALS PER/10 m^2

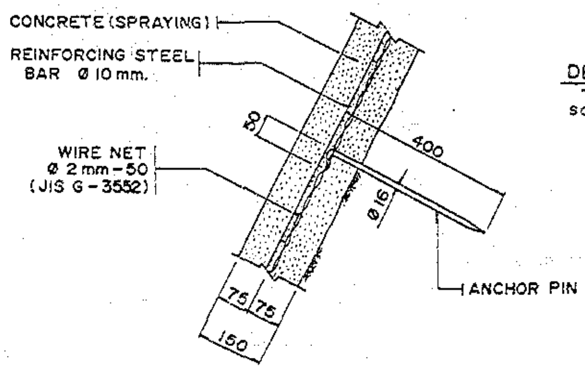
Thickness (Cm.)	Concrete or Mortar (m^3)	Anchor Pin (Each)	Wire Net (m^2)	Reinf. Steel (Kg.)	Weep Hole (Each)
15	15	10	10	246	10
10	10	10	10	—	10



DETAILS OF "A"
THICKNESS 10 cm.
SCALE 1:10



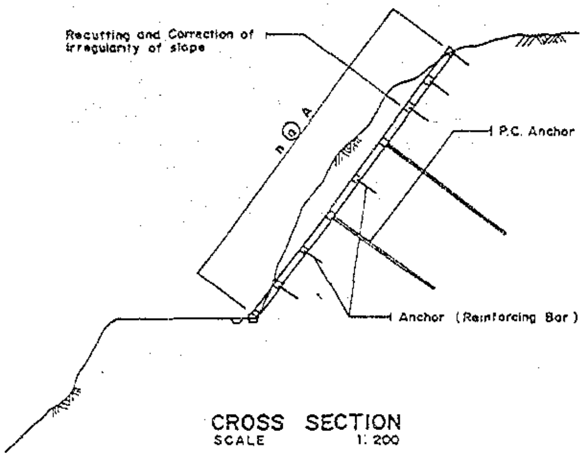
BAR ARRANGEMENT
THICKNESS 15 cm.
SCALE 1:50



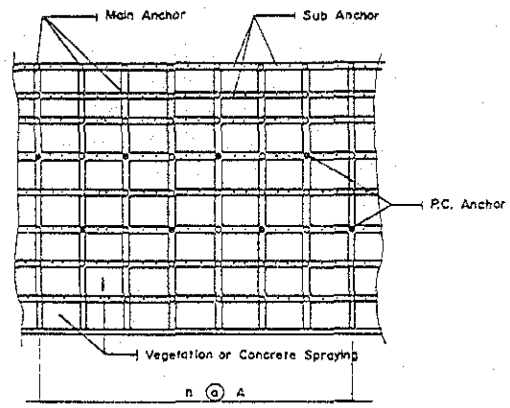
DETAILS OF "A"
THICKNESS 15 cm.
SCALE 1:10

STANDARD DRAWINGS : **CAST – IN – PLACE CONCRETE CRIB**

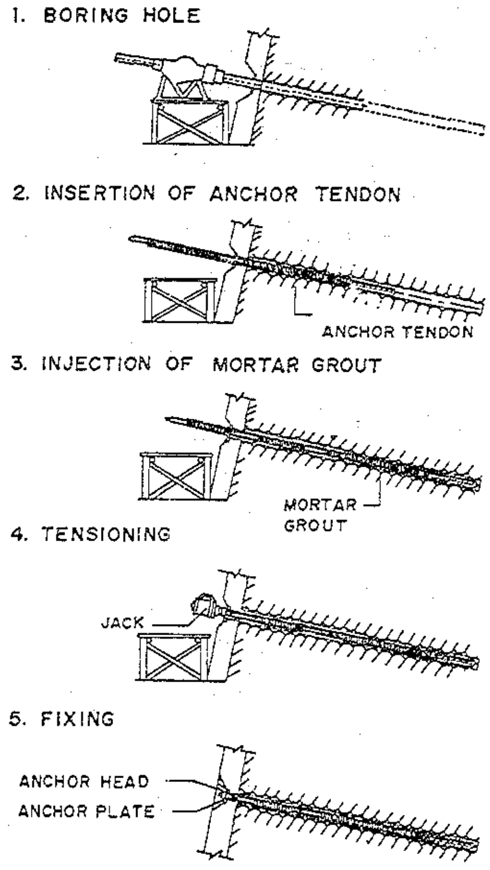
SCALE	DRAWING NO.
AS SHOWN	6



CROSS SECTION
SCALE 1:200

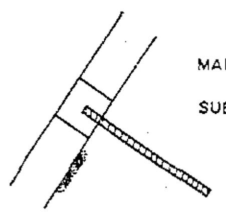


DEVELOPMENT
SCALE 1:200



PROCEDURE OF P.C ANCHOR INSTALLATION
SCALE 1:40

NOTE : 1. If the ground is stable, P.C. Anchor is not required.
2. If the ground is unstable, P.C. Anchor is required.
3. A = 1 500 mm / 2 000 mm / 2 500 mm / 3 000 mm
4. Can be applied to any kind of material for framing.

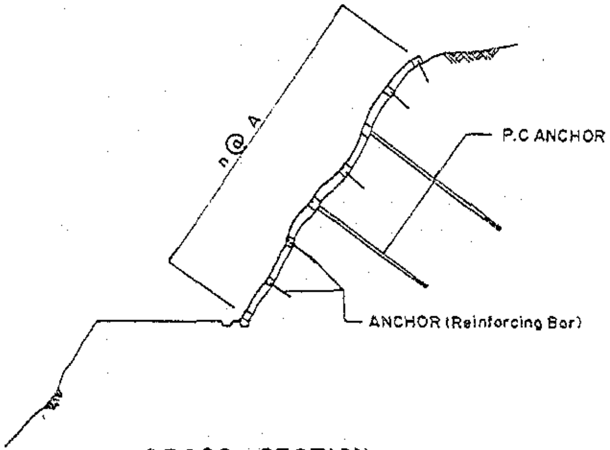


MAIN ANCHOR Ø16 ~ Ø22 L=750 ~ 1500
SUB ANCHOR Ø10 ~ Ø16 L=400 ~ 700

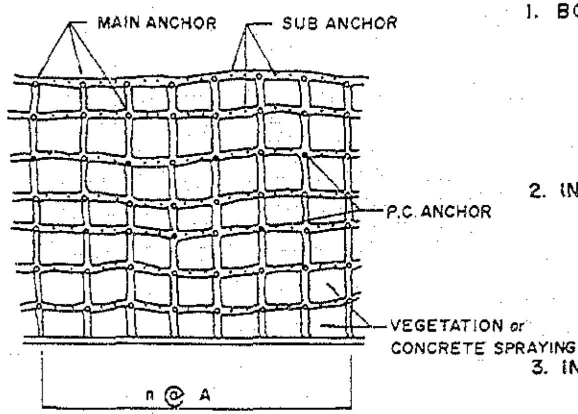
ANCHOR (REINFORCING BAR)
SCALE 1:30

STANDARD DRAWINGS : SPRAYED CONCRETE CRIB, P.C. ANCHOR

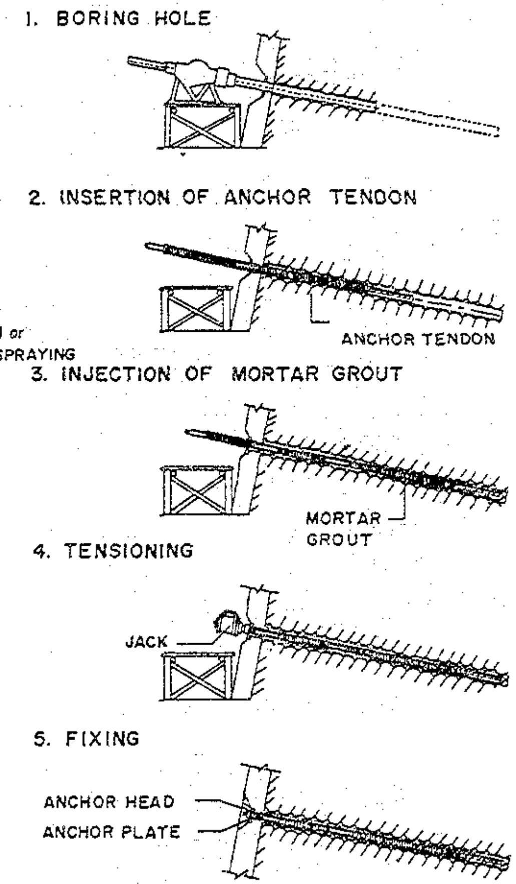
SCALE AS SHOWN | DRAWING NO. 7



CROSS SECTION
SCALE 1:200

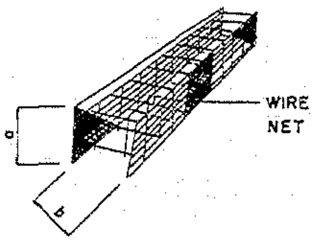


DEVELOPMENT
SCALE 1:200



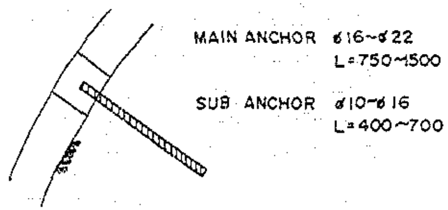
PROCEDURE OF P.C. ANCHOR INSTALLATION
SCALE 1:40

NOTE: 1. If the ground is stable, P.C. Anchor is not required.
2. If the ground is unstable, P.C. Anchor is required.
3. A = 1500 mm / 2000 mm / 2500 mm / 3000 mm



a x b : 150 x 150
200 x 200
300 x 300
400 x 400
500 x 500

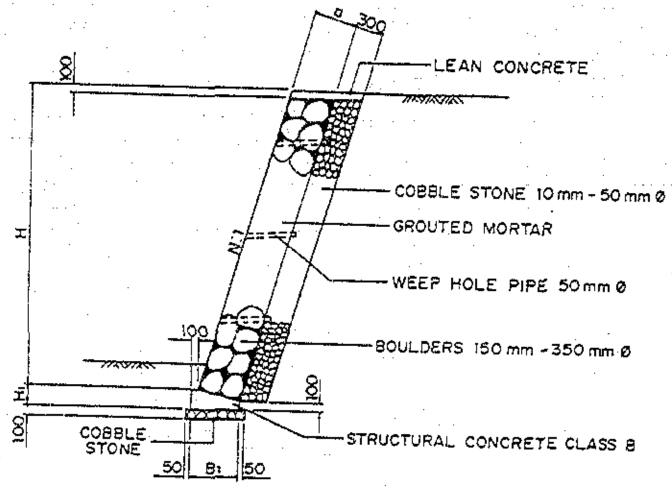
FRAME



ANCHOR (REINFORCING BAR)
SCALE 1:30

STANDARD DRAWINGS : GROUTED RIPRAP

SCALE 1:50 | DRAWING NO. 8



GROUTED RIPRAP

LIST OF DIMENSION

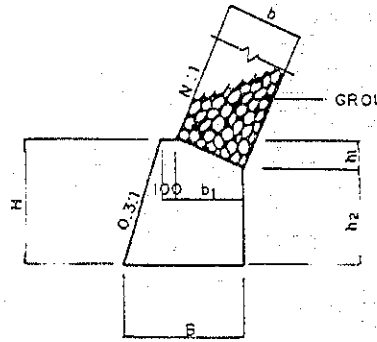
CLASS	H	N	a	B_1	H_1
A	H ≦ 3000	0.3 (0.3)	300	390	180
B	3000 H ≦ 4000	0.3 (0.4)	500	580 (560)	240 (320)
C	4000 H ≦ 5000	0.4 (0.5)	600	660 (640)	320 (450)
D	H > 5000	0.5 (0.6)	800	820 (790)	450 (510)

NOTE: 1. (n) = Embankment Slope Factor

LIST OF MATERIALS PER/10 M.

DIMENSION			GROUTED RIPRAP	BACKFILL COBBLE STONE	BASE CONCRETE	BASE COBBLE STONE
H (M)	a (cm)	N	(m^3)	(m^3)	(m^3)	(m^3)
3.0	30	0.3	9.4	9.4	0.59	0.49
4.0	50	0.3	20.9	12.5	1.06	0.68
4.0	50	0.4	21.5	12.9	1.29	0.66
5.0	60	0.4	32.3	16.2	1.50	0.76
5.0	60	0.5	33.5	16.8	1.94	0.74
5.0	80	0.5	44.7	16.8	2.43	0.92
5.0	80	0.6	46.6	17.5	2.61	0.89

STANDARD DRAWINGS : **FOUNDATION FOR GROUTED RIPRAP** | SCALE 1:50 | DRAWING NO. 9



FOUNDATION FOR GROUTED RIPRAP

LIST OF DIMENSION AND CONCRETE VOLUME

N = 0.3 , b = 300

H	h_1	h_2	B	b_1	CONCRETE (m^3)
2000	90	1910	990	290	1.4
3000	"	2910	1290	"	2.5
4000	"	3910	1590	"	4.0
5000	"	4910	1890	"	5.7

N = 0.4 , b = 600

H	h_1	h_2	B	b_1	CONCRETE (m^3)
2000	220	1780	1260	560	1.9
3000	"	2780	1560	"	3.3
4000	"	3780	1860	"	5.0
5000	"	4780	2160	"	7.0

N = 0.3 , b = 500

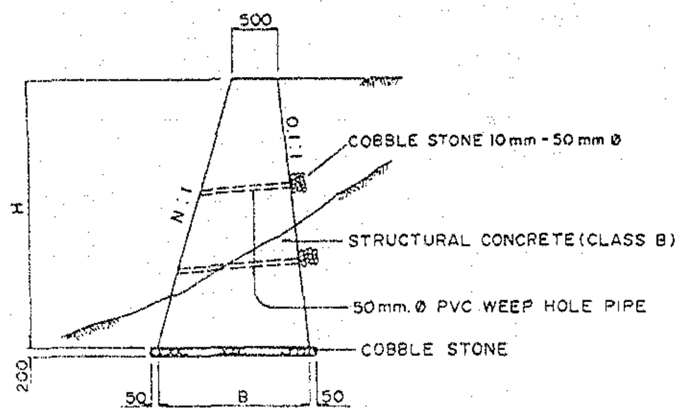
H	h	h	B	b	CONCRETE (m)
2000	140	1860	1180	480	1.7
3000	"	2860	1480	"	3.1
4000	"	3860	1780	"	4.7
5000	"	4860	2080	"	6.6

N = 0.5 , b = 800

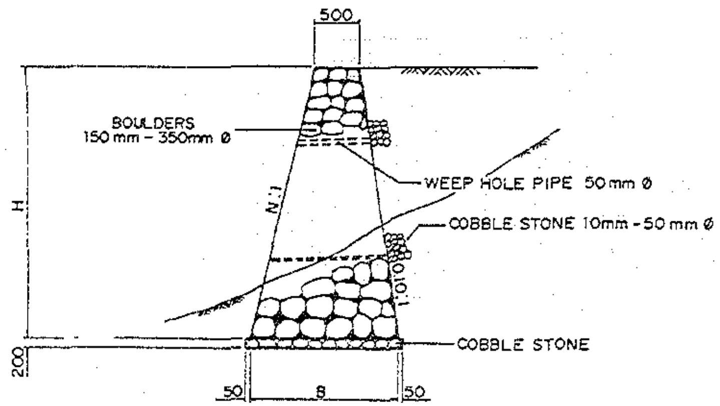
H	h	h	B	b	CONCRETE (m)
2000	360	1640	1420	720	2.1
3000	"	2640	1720	"	3.7
4000	"	3640	2020	"	5.6
5000	"	4640	2320	"	7.7

STANDARD DRAWINGS : GRAVITY WALL , GRAVITY TYPE STONE MASONRY WALL

SCALE 1 : 50 | DRAWING NO. 10



GRAVITY WALL



GRAVITY TYPE STONE MASONRY WALL

LIST OF DIMENSION AND MATERIALS PER/M

H	N_1	B	CONCRETE H (m)	CONCRETE V(m^3)	COBBLE STONE (m^3)
H ≦ 2000	0.25	500+H (N +0.1)	2.0	1.7	0.26
2000 < H ≦ 3000	0.30	"	3.0	3.3	0.36
3000 < H ≦ 4000	0.35	"	4.0	5.6	0.48
4000 < H ≦ 5000	0.40	"	5.0	8.8	0.62

NOTE :

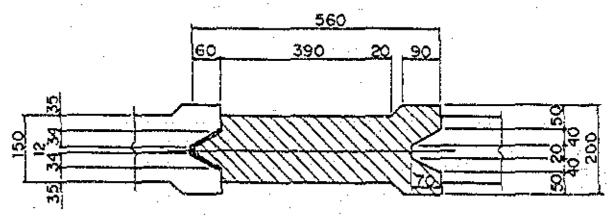
1. For Weep hole pipe, Use 1-50mm. Ø pipe for every 2.0 m^2
2. For base foundation not made of rock, Use concrete base.
3. Cobble Stone must be well compacted.

LIST OF DIMENSION AND MATERIALS PER/M

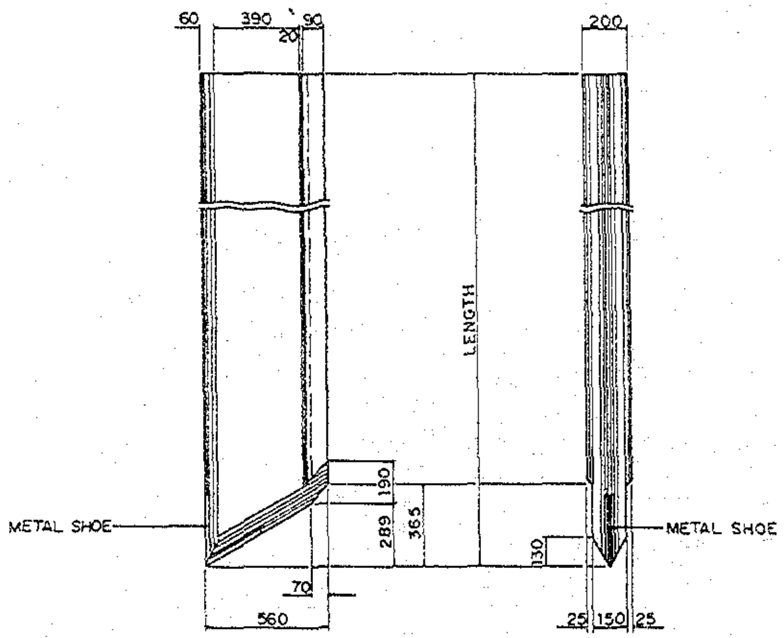
H	N	B	CONCRETE H(m)	CONCRETE V(m^3)	COBBLE STONE (m^3)
H ≦ 2000	0.25	500 + H x (N + 0.1)	2.0	1.7	0.26
2 000 < H ≦ 3 000	0.30	"	3.0	3.3	0.36
3 000 < H ≦ 4 000	0.35	"	4.0	5.6	0.48
4 000 < H ≦ 5 000	0.40	"	5.0	8.8	0.62

NOTE: 1. For Weep hole pipe, Use 1-50 mm. Ø pipe for every 2.0 m^2
2. For base foundation not made of rock, Use concrete base.
3. Cobble stone must be well compacted.

STANDARD DRAWINGS : R.C. SHEET PILE , GRAVITY TYPE SEA WALL	SCALE	DRAWING NO.
	AS SHOWN	11



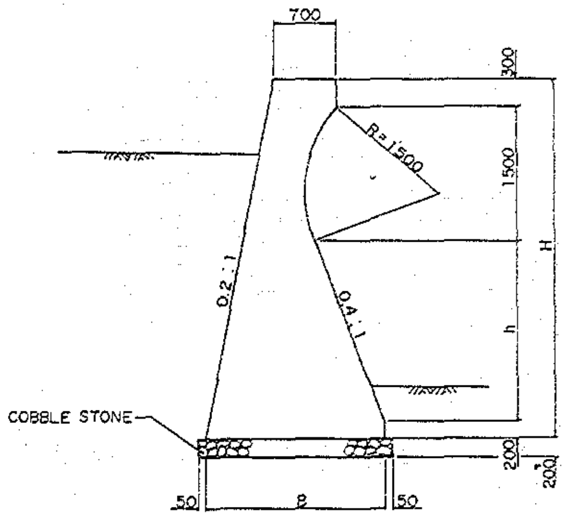
PLAN
SCALE: 1:10



FRONT ELEVATION

SIDE ELEVATION

R.C. SHEET PILES
SCALE: 1:20



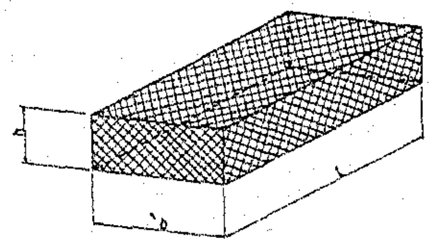
SEAWALL
SCALE: 1:50

LIST OF DIMENSION AND MATERIALS PER/M

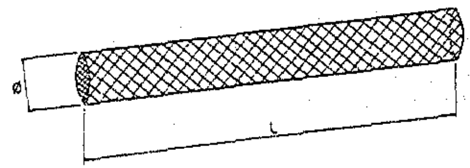
H (m.)	B (m.)	h (m.)	CONCRETE (m^3)	COBBLE STONE (m^3)
3.0	1.48	1.0	2.84	0.32
3.5	1.78	1.5	3.68	0.38
4.0	2.08	2.0	4.68	0.44
4.5	2.38	2.5	5.82	0.50
5.0	2.60	3.0	6.99	0.54

NOTE: R.C. piles or Steel H-pile or Ladder foundation to be used depending on geotechnical condition.

STANDARD DRAWINGS : MAT GABION , CYLINDER GABION	SCALE AS SHOWN	DRAWING NO. 12

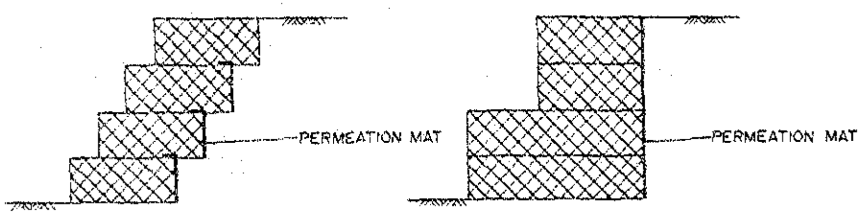


h : 0.40 m, 0.50 m, 0.60 m
b : 1.2 m
L : 2.0 m ~ 6.0 m (1.0 m Pitch)



Ø : 0.45m, 0.60m, 0.90m
L : 3 m ~ 8 m (1.0 m Pitch)

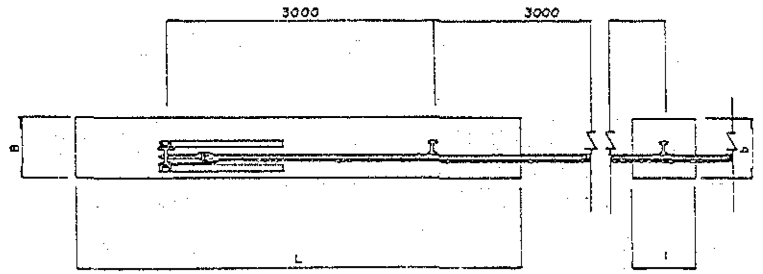
CYLINDER GABION



MAT GABION
SCALE: 1:50

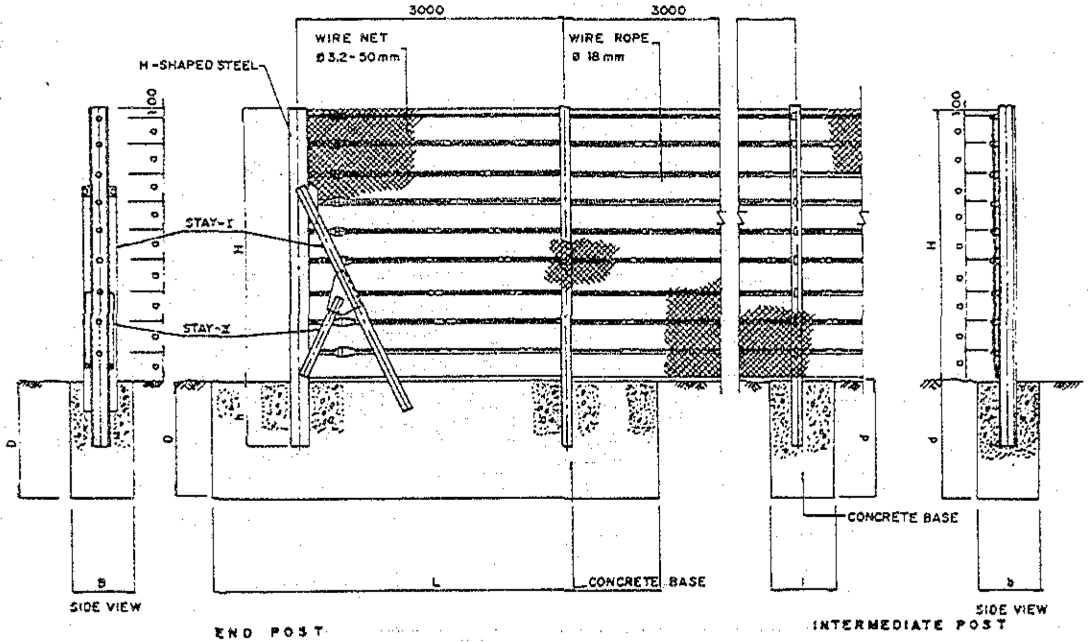
NOTE: Permeation mat was used to protect the backfill from sinking.

STANDARD DRAWINGS : CATCH FENCE	SCALE 1:500	DRAWING NO. 13



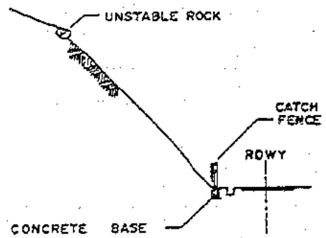
PLAN

TYPE	HEIGHT	WIRE ROPE NUMBER	WIRE ROPE SPACING	INTERMEDIATE SUPPORT POST SECTION	INTERMEDIATE SUPPORT POST LENGTH	INTERMEDIATE SUPPORT POST EMBEDMENT	END SUPPORT POST	ABSORPTION ENERGY BY FENCE
	H(m)	EACH	a(mm)	(mm)	(mm)	h(mm)		(t-m)
A	1.00	3	300 ~350	H-150x75 x5x7	1,500	500	H-125x125x6.5x9-1500 STAY-I H-100x100x6x8	4.5
B	1.25	4	300 ~350	H-150x75 x5x7	1,800	550	H-125x125x6.5x9-1800 STAY-I H-100x100x6x8	4.3
C	1.55	5	300 ~350	H-200x100 x5.5x8	2,200	650	H-150x150x7x10-2200 STAY-I H-125x125x6.5x9 PLATE PL-6x50	6.3
D	2.00	6	300 ~350	H-200x100 x5.5x8	2,750	750	H-175x175x7.5x11-2700 STAY-I H-150x150x7x10 PLATE PL-6x50	5.8
E	2.50	8	300 ~350	H-200x100 x5.5x8	3,300	800	H-200x200x8x12 STAY-I C-180x75x6.5x10 STAY-II C-100x50x5x7.5	5.5
F	3.00	9	300 ~350	H-200x100 x5.5x8	3,800	800	H-200x200x8x12 STAY I C-180x75x6.5x10 STAY II C-100x50x5x7.5	5.4



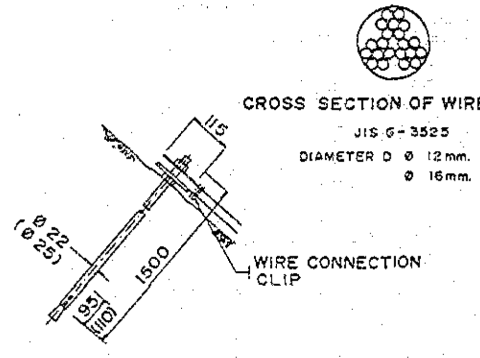
ELEVATION

TYPE	INTERMEDIATE POST b	INTERMEDIATE POST l	INTERMEDIATE POST d	END POST B	END POST L	END POST D
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
A	600	600	1000	600	4000	800
B	600	600	1000	600	4000	1000
C	700	700	1100	600	4000	1300
D	700	700	1100	600	4500	1300
E	700	700	1100	700	5000	1300
F	700	700	1100	700	5000	1400



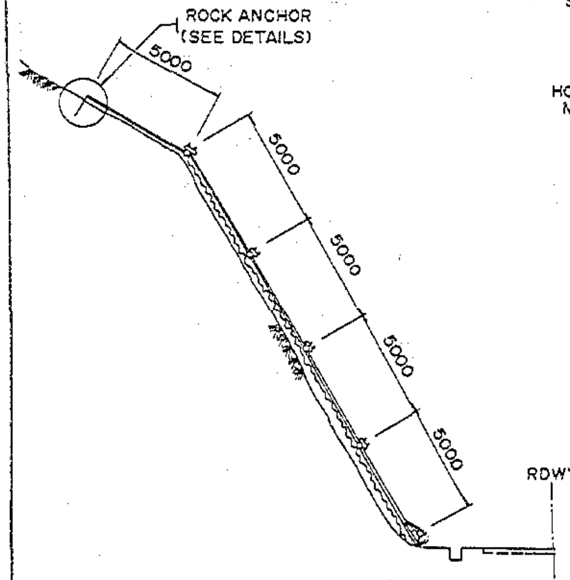
LOCATION OF CATCH FENCE

STANDARD DRAWINGS :	CATCH WIRE NET	SCALE AS SHOWN	DRAWING NO. 14

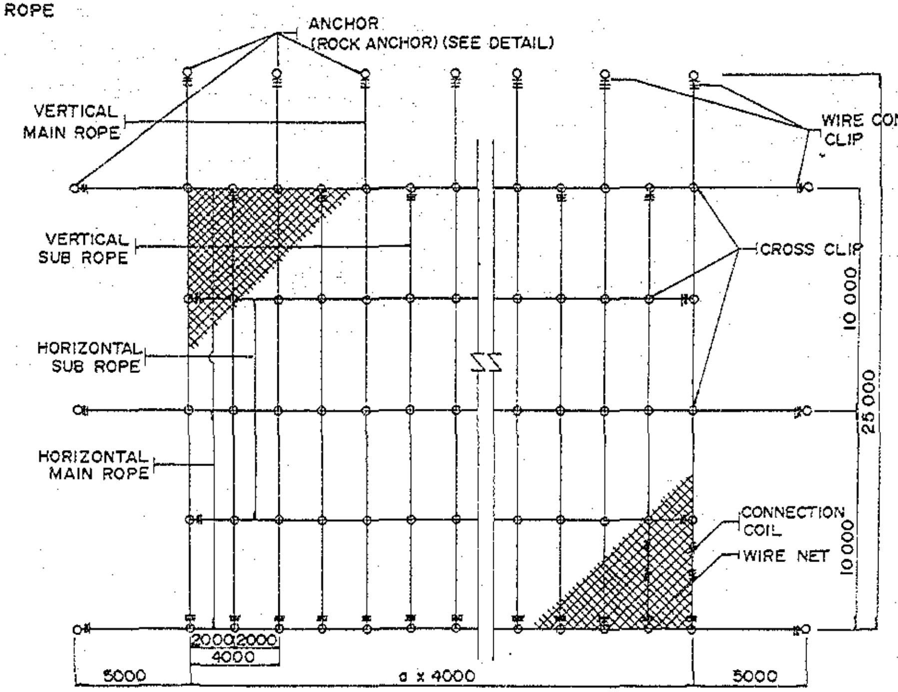


CROSS SECTION OF WIRE ROPE

DETAILS OF ROCK ANCHOR



CROSS SECTION



CROSS CLIP

WIRE CONNECTION CLIP

CONNECTION COIL

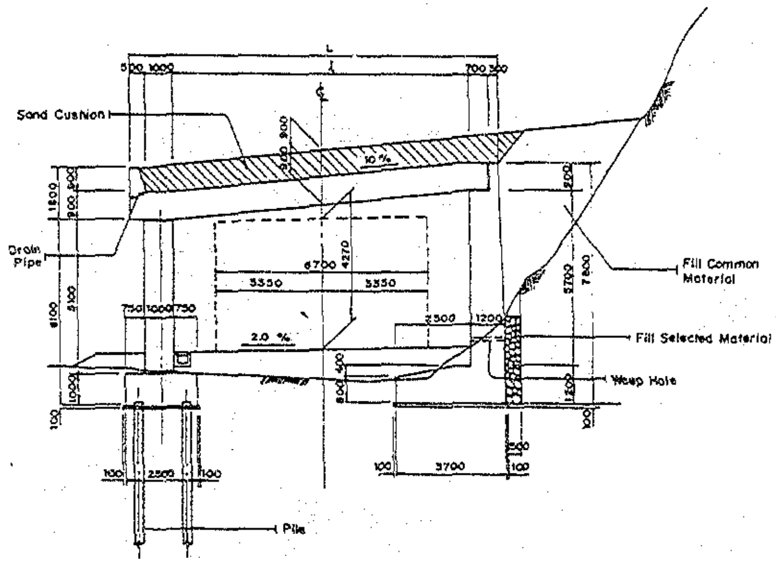
DIMENSION TABLE OF ROCK NET

ITEM	WIRE NET	WIRE ROPE*1		CONDITION OF SLOPE AND ROCK			ANCHOR
	GALVANIZED WIRE NET	MAIN ROPE	SUB ROPE	MAX. SLOPE LENGTH	MAX. SLOPE GRADIENT	ALLOWABLE*2 WT. OF ROCK	ROCK ANCHOR
1500	Ø 4.0mm. x 50 x 50	Ø 16	Ø 12	50 m.	0.5 : 1	1500 Kg.	Ø 25
1000	Ø 3.2mm. x 50 x 50	Ø 16	Ø 12	70 m.	0.5 : 1	1000 Kg.	Ø 25
500	Ø 2.6mm x 50 x 50	Ø 12	Ø 12	70 m.	0.5 : 1	500 Kg.	Ø 22

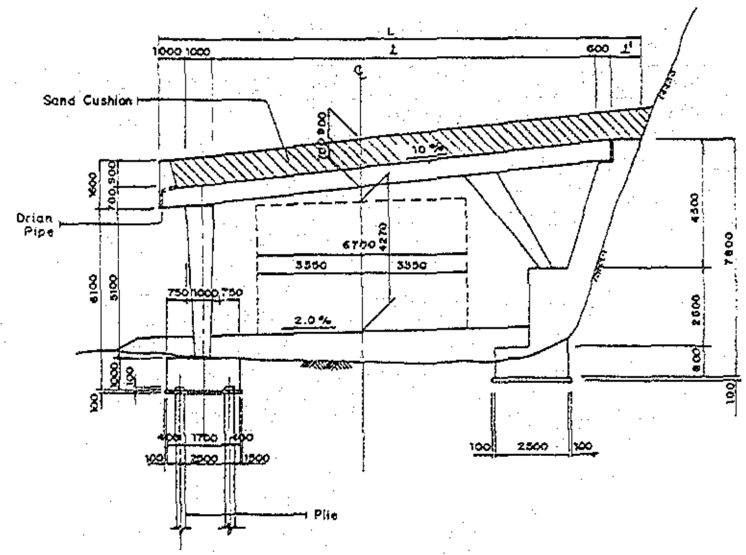
*1 JIS-G 3525 3 x 7 G/O TYPE
ULTIMATE TENSILE STRENGTH MORE THAN 7000 Kg. For Ø12mm.
MORE THAN 12000 Kg. For Ø16mm.

*2 UNIT; PER 40 SQUARE METER (4 m. x 10 m.)

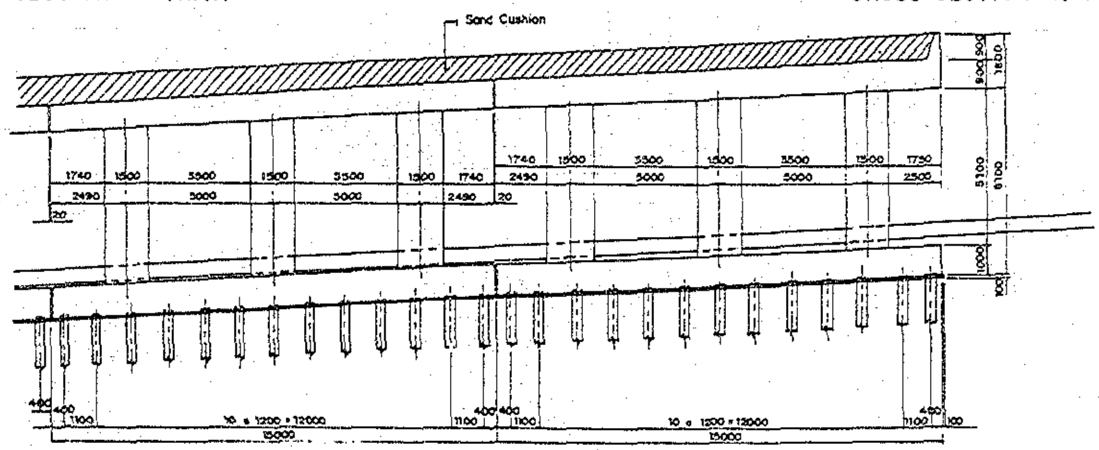
STANDARD DRAWINGS :	ROCK SHED	SCALE 1:150	DRAWING NO. 15



CROSS SECTION (R.C.)

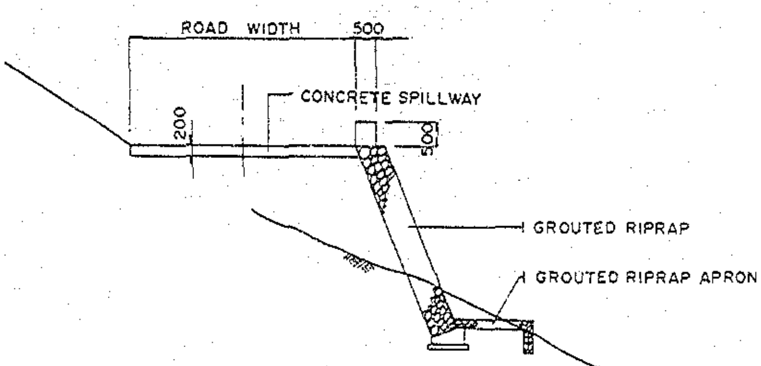


CROSS SECTION (P.C.)

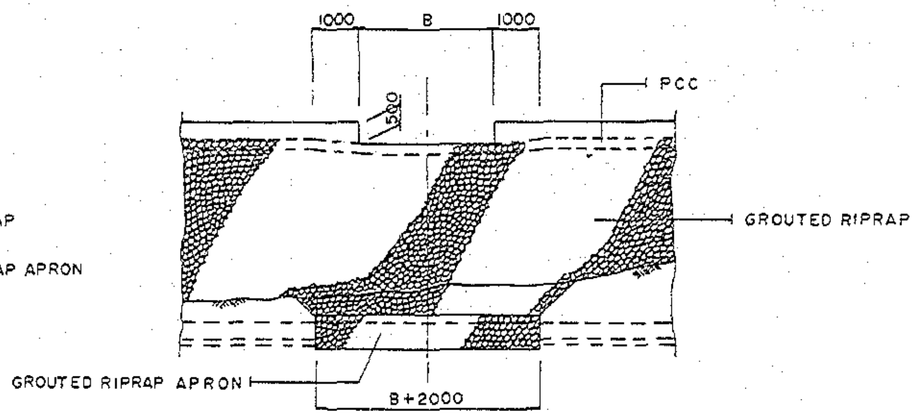


ELEVATION (R.C.)

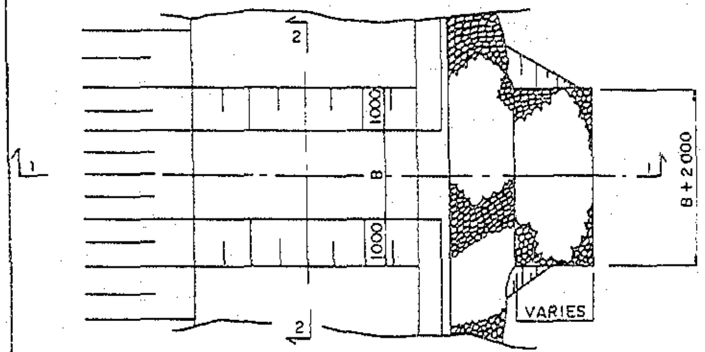
STANDARD DRAWINGS : CONCRETE SPILLWAY, GROUTED RIPRAP APRON	SCALE	DRAWING NO.
	1:100	16



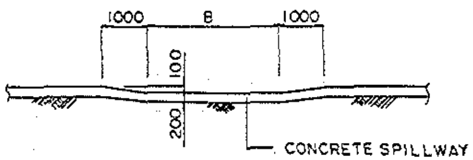
CROSS SECTION (1-1)



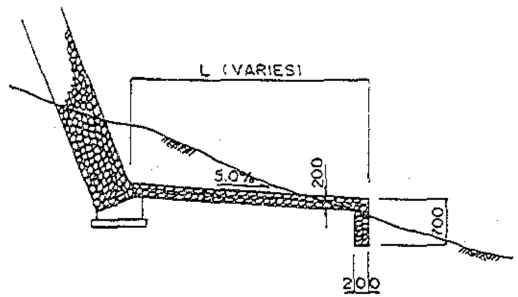
FRONT VIEW



PLAN

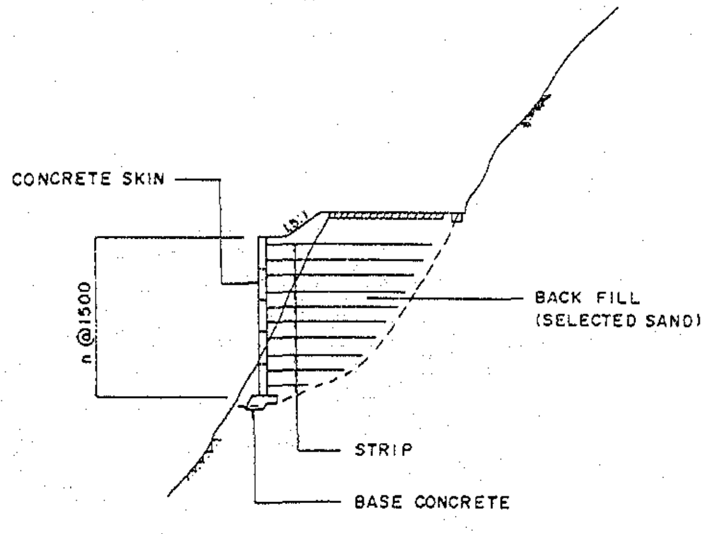


CROSS SECTION (2-2)

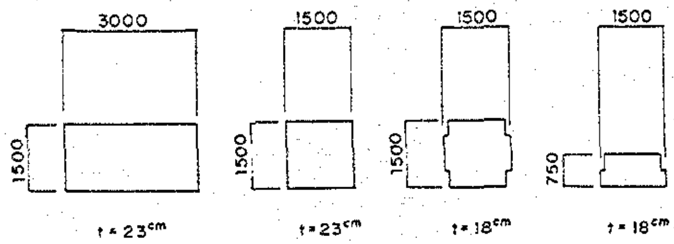


GROUTED RIPRAP APRON

STANDARD DRAWINGS : REINFORCED EARTH WALL	SCALE AS SHOWN	DRAWING NO. 17



CROSS SECTION
SCALE 1:200



TYPICAL OF CONCRETE SKIN
SCALE 1:100

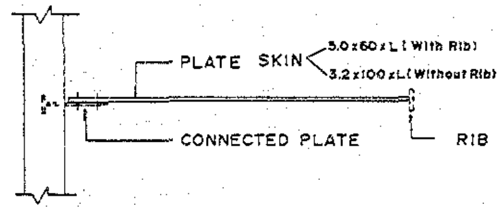
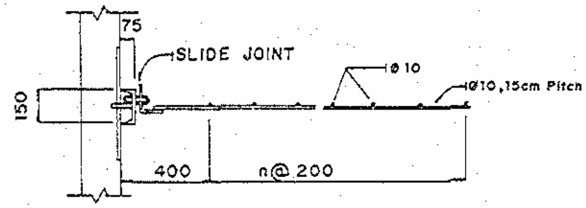
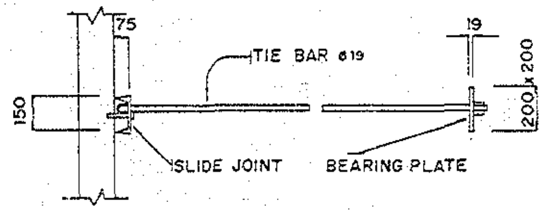


PLATE TYPE



REINFORCING BAR TYPE



BEARING PLATE TYPE

TYPICAL OF STRIP
SCALE 1:20

APPENDIX III
EXAMPLES OF DESIGN

APPENDIX III

EXAMPLES OF DESIGN

TABLE OF CONTENTS

APPENDIX III
EXAMPLES OF DESIGN

1. Cut Slope Failure

1) Spot L-84 (Leyte)

Location	:	7.6 km from Jct. Abuyog-Mahaplag Road
Road Name	:	Tadoc-Southern Leyte Road
Road Classification	:	Barangay Road
Geological Condition	:	Highly weathered hard clay
Water Condition	:	Water from hinterland

2) Description of Disaster

Top soil of the cut slope slid down 30 meters in length and 12 meters in height. The road was partially covered by the fallen soil.

3) Causes of Disaster

Steep gradient of cut slope and erosion of top soil are major causes of failure.

4) Proposed Restoration Measures

<u>Urgent Restoration Measures</u>

The proposed measures are as follows:

Proposed Measures		Purpose
U1-1	Removal of Deposit Materials	To remove traffic obstruction

<u>Permanent Restoration Measures</u>

Two options were proposed; 1) grouted riprap with vegetation and 2) grouted riprap and concrete crib. They were compared as shown in Figure 1-1. From the economic and environmental aspects, the former was selected. The selected measures are as follows:

Proposed Measures		Purpose
P1-1	Recutting	To make the slope stable
P2-2	Side Ditch	To collect surface water coming from mountain side and thus prevent water from running directly on the surface of the road.
P4-2	Hand Seeding with Mat	To prevent the slope from erosion
P6-2	Grouted Riprap	To protect the slope toe

Note: See Figure 1-2.

FIGURE 1-1 COMPARISON OF ALTERNATIVE RESTORATION MEASURES

TYPE OF DISASTER : CUT SLOPE FAILURE
PROVINCE AND SPOT NO: LEYTE, L-84

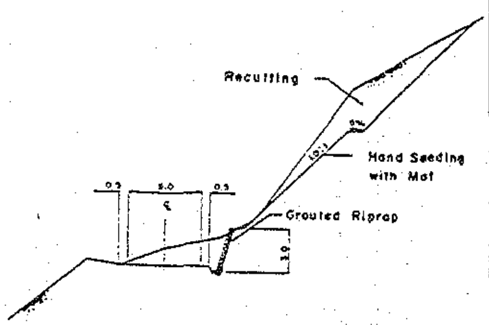
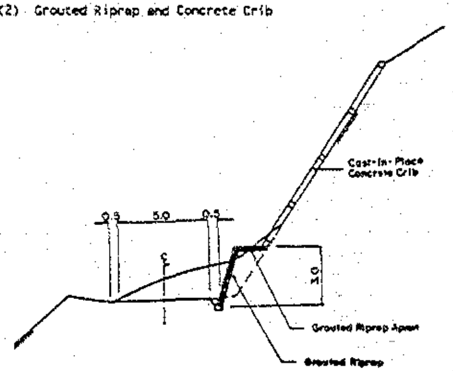
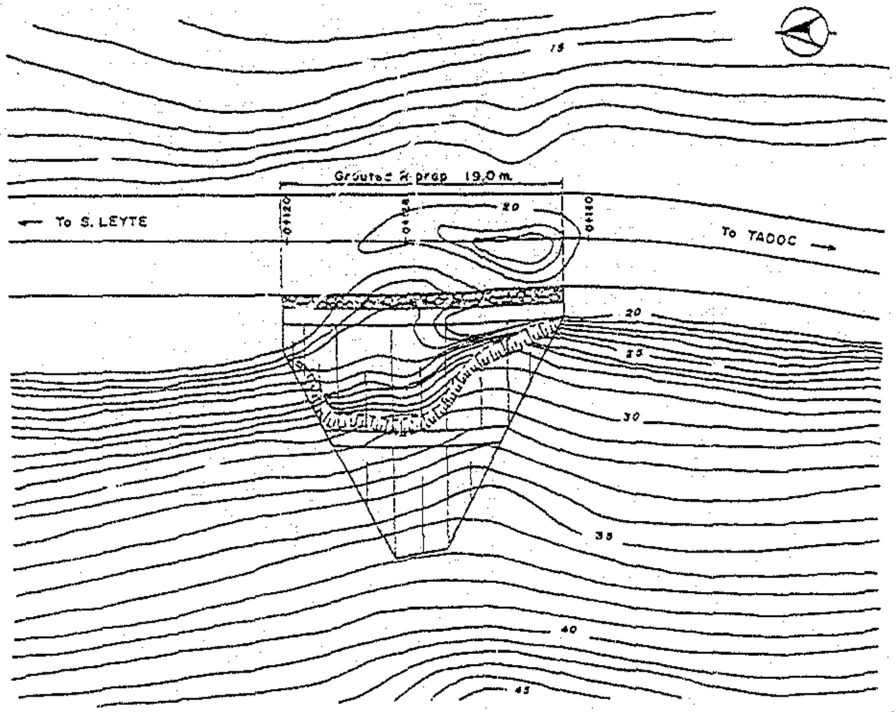
TYPE OF WORK AND ILLUSTRATION	ENGINEERING CHARACTERISTICS	CONSTRUCTION COST (₱1,000)	CONSTRUCTION CHARACTERISTICS	ENVIRONMENTAL ASPECTS	REMARKS
(1) Grouted Riprap and Vegetation 	· The slope is stabilized by cutting to stable gradient and protected by vegetation. · Maintenance is needed for vegetation growth.	Recutting 240 m^3 x 50 = 14 Grouted Riprap 34m^3 x 1,326 = 25 Hand Seeding w/ Mat 290 m^2 x 44 = 13 Side Ditch 19 m x 250 = 5 ₱57	· Special equipment and expertise are not necessary. · Construction period is about 1 month and 1 lane is passable during construction.	· The vegetation conserves natural environment.	· Recommendable from the economic and environmental aspects.
(2) Grouted Riprap and Concrete Crib 	· The slope is completely protected from erosion by concrete crib.	Concrete Crib 25m^3 x 2,238 = 78 Reinforcing Steel Bars 300 kg x 34 = 10 Grouted Riprap 19m^3 x 1,326 = 25 Grouted Riprap Apron 8 m^3 x 1,326 = 11 Side Ditch 19 x 250 = 5 ₱129	· Construction of cast-in-place on steep slope needs time and many labors but special equipment and expertise are not necessary. · Construction period is about 3 months and 1 lane is passable during construction	· Cast-in-place concrete harms natural view.	· Applicable.

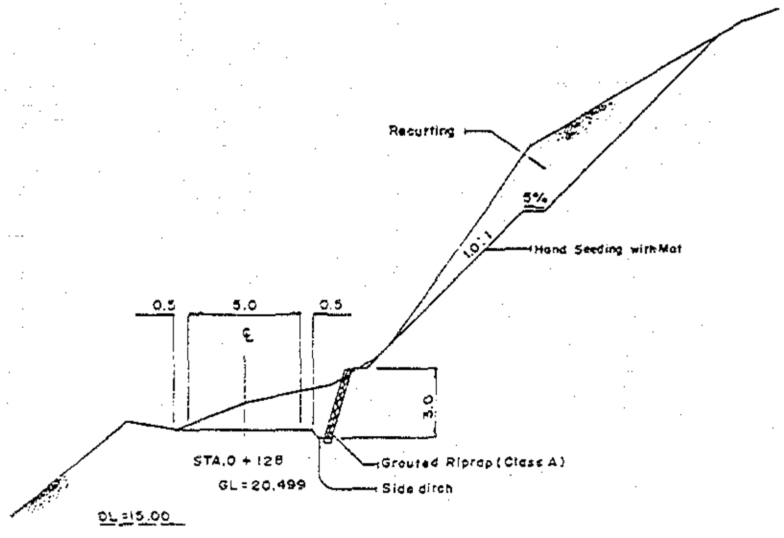
FIGURE 1-2 PROPOSED RESTORATION MEASURES (L-84)

PROVINCE : LEYTE SPOT No. : L-84 (2/2)	NAME OF ROAD : TADOC-SOUTHERN LEYTE ROAD ROAD CLASSIFICATION : BARANGAY ROAD	TYPE OF DISASTER : CUT SLOPE FAILURE

DRAWING NO.



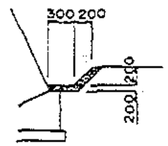
P L A N
SCALE 1:300



CROSS SECTION
SCALE 1:200

SUMMARY OF QUANTITY

TYPE OF WORK		UNIT	TOTAL
PERMANENT RESTORATION			
P1-1	RECUTTING	CU.M.	240
P2-2	SIDE DITCH	L.M	19
P4-2	HAND SEEDING WITH MAT	SQ.M.	290
P6-2	GROUTED RIPRAP	CU.M	19
URGENT RESTORATION			
U1-1	REMOVAL OF DEPOSIT MATERIALS	CU.M.	80



DETAIL OF SIDE DITCH
SCALE 1:50

URGENT RESTORATION
- Removal of deposit materials

2. Embankment Slope Failure

1) Spot Bt-54 (Benguet)

Location	:	23.4 km from Kibungan proper
Road Name	:	Kibungan-Kapangan Road
Road Classification	:	National Secondary Road
Geological Condition	:	Gravel bed
Water Condition	:	Surface water runs directly on the slope but not concentrates on a particular portion

2) Description of Disaster

A river runs along the road embankment. The embankment slope with a gradient of 45 degrees and a height of 7 meters fell down 16 meters in length and 2.5 m in thickness on top. Debris was deposited on the riverbed. A triangular side ditch on the mountain side of the road was silted.

3) Causes of Disaster

Debris deposits on the riverbed narrowed river width at this point, resulting in rise of water level and high velocity of flow which caused erosion of embankment.

4) Proposed Restoration Measures

Urgent Restoration Measures

The proposed measures are as follows:

Proposed Measures		Purpose
U1-4	Refilling/Embankment	To fill scoured portion of slope
U4-3	Wooden Fence	To retain refilled slope temporarily

Permanent Measures

Three options were proposed; 1) grouted riprap, 2) gabion wall and 3) supported type concrete wall. They were compared as shown in Figure 2-1. From the engineering and economical aspects, the second option was selected, which is as follows:

Proposed Measures		Purpose
P1-3	Refilling/Embankment	To fill scoured portion of slope
P6-9	Gabion Wall	To retain refilled slope and protect it from scour

Note: See Figure 2-2.

FIGURE 2-1 COMPARISON OF ALTERNATIVE RESTORATION MEASURES

TYPE OF DISASTER : EMBANKMENT SLOPE FAILURE
PROVINCE AND SPOT NO.: BENGUET, Bt-54

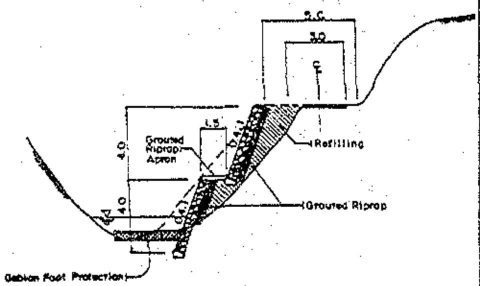
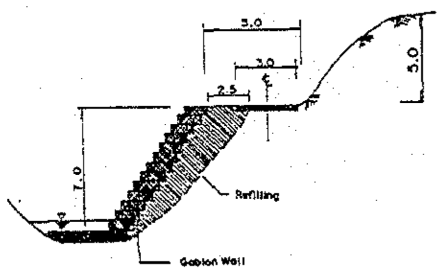
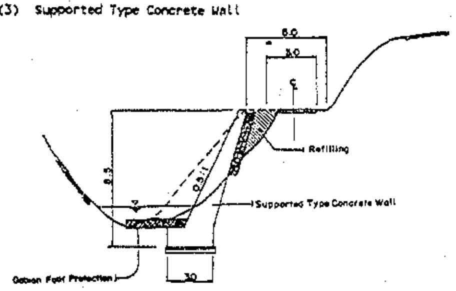
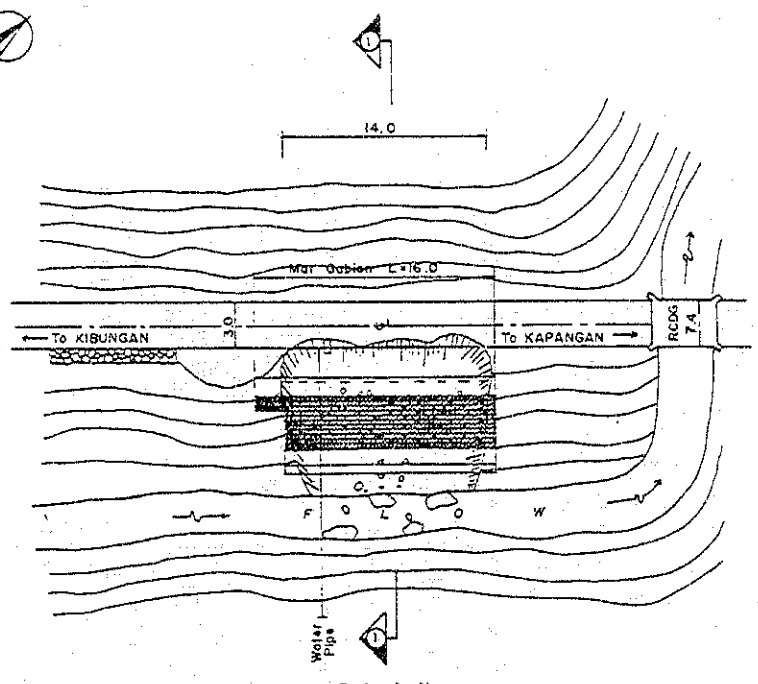
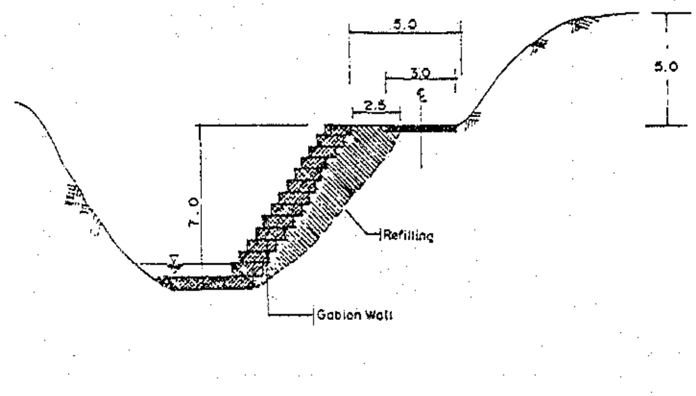
TYPE OF WORK AND ILLUSTRATION	ENGINEERING CHARACTERISTICS	CONSTRUCTION COST (₱1,000)	CONSTRUCTION CHARACTERISTICS	ENVIRONMENTAL ASPECTS	R E M A R K S
(1) Grouted Riprap 	· Grouted Riprap is weaker than Gabion Wall and Concrete Wall in strength of sustaining earth pressure. · The foundation of Grouted Riprap of upper row is in the embankment it is questionable.	Excavation 68 m^3 x 90 = 4 Grouted Riprap 72 m^3 x 1,326 = 95 Grouted Riprap Apron 5 m^3 x 1,326 = 7 Refilling 94 m^3 x 68 = 6 Gabion Foot Protection 29 m^3 x 1,425 = 41 ₱153	· Special equipment and expertise are not necessary. · Construction period is about 1 month.		· Not recommendable from the engineering aspect.
(2) Gabion Wall 	· Gabion is effective when there is seepage water from backfill. · River is maintained to its original width.	Refilling 310 m^3 x 68 = 21 Gabion Wall 154m^3 x 1,425 =219 Permeance Mat 100 m^2 x 34 = 3 ₱243	· Special equipment and expertise are not necessary. · Construction period is about 1.5 months.		· Recommendable from the engineering and economical aspects.
(3) Supported Type Concrete Wall 	· Durability and reliability is higher than the others.	Excavation 192 m^3 x 90 = 17 Concrete Wall 230m^3 x 2,942 =677 Refilling 43 m^3 x 68 = 3 Gabion Foot Protection 29m^3 x 1,425 = 41 ₱738	· Special equipment and expertise are not necessary. · Construction period is about 2 months.		· Applicable.

FIGURE 2-2 PROPOSED RESTORATION MEASURES (Bt-54)

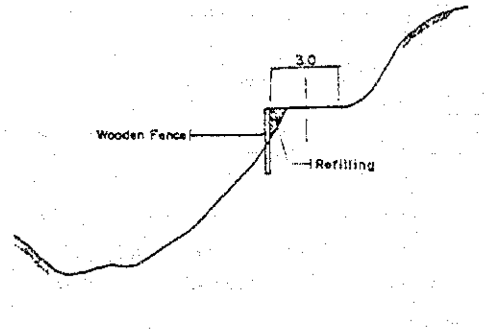
PROVINCE : BENGUET SPOT No. : Bt-54 (2/2)	NAME OF ROAD : KIBUNGAN – KAPANGAN ROAD ROAD CLASSIFICATION : NATIONAL SECONDARY ROAD	TYPE OF DISASTER : EMBANKMENT SLOPE FAILURE	DRAWING NO.



PLAN
SCALE 1:300



CROSS SECTION
SCALE 1:200



CROSS SECTION FOR URGENT RESTORATION
SCALE 1:200

SUMMARY OF QUANTITY

TYPE OF WORK		UNIT	TOTAL
PERMANENT RESTORATION			
P1-3	REFILLING / EMBANKMENT	CU. M.	310
P6-6	GABION WALL	CU. M.	154
URGENT RESTORATION			
U1-4	REFILLING / EMBANKMENT	CU. M.	7
U4-3	WOODEN FENCE	LM.	16

3. Rock Fall/Debris Fall

1) Spot L-65 (Leyte)

Location	:	3.3 km from Albuera Jct. - Baybay Road
Road Name	:	Albuera - Burauen Road
Road Classification	:	National Secondary Road
Geological Condition	:	Bedrock is tuff breccia but surface is highly weathered.
Water Condition	:	Surface water from hinterland

2) Description of Disaster

Rock fall occured last November 12, 1990 during typhoon "Ruping". Damaged section was about 34.0 m long and 25.0 m high. The cut slope is perpendicular or overhung.

3) Causes of Disaster

Weathered rocks on the surface of overhung slope were detached from bedrock due to seepage of water during heavy rain.

4) Proposed Restoration Measures

Urgent Restoration Measures

The proposed measures are as follows:

Proposed Measures		Purpose
U1-1	Removal of Deposite Materials	To remove traffic obstruction
U1-2	Removal of Unstable Materials	To prevent temporarily recurrence of rock fall

Permanent Restoration Measures

The two options were proposed: 1) recutting and grouted riprap and 2) realignment and catch wall. The two were compared as shown in Figure 3-1 and the first option was selected mainly for economical reason. The selected measures are as follows:

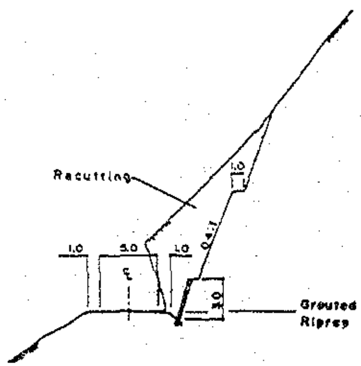
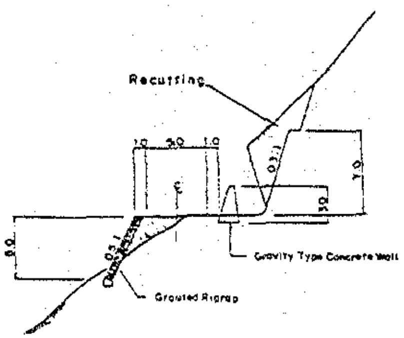
Proposed Measures		Purpose
P1-1	Recutting	To stabilize the slope
P2-2	Side Ditch	To prevent surface water from running on road surface
P6-2	Grouted Riprap	To protect the slope toe

Note: See Figure 3-2

FIGURE 3-1 COMPARISON OF ALTERNATIVE RESTORATION MEASURES

TYPE OF DISASTER: FALL

PROVINCE AND SPOT NO.: LEYTE, L-65

TYPE OF WORK AND ILLUSTRATION	ENGINEERING CHARACTERISTICS	CONSTRUCTION COST (₱1,000)	CONSTRUCTION CHARACTERISTICS	ENVIRONMENTAL ASPECTS	REMARKS
(1) Recutting and Grouted Riprap 	· The slope is stabilized to a degree of little possibility of recurrence of disaster.	Recutting 1,440m^3 x 359 =409 Side Ditch 34 m x 250 = 9 Grouted Riprap 34 m x 1,326 = 45 ₱463	· Special equipment and expertise are not necessary. · Construction period is about 3 months. · During the recutting work (dynamite blasting), road is temporarily closed to traffic.		· Recommendable to economical reason.
(2) Realignment and Catch Wall 	· There is a possibility of surface failure at the upper part of slope. However, fallen rocks will be checked by the catch wall. · Maintenance will be needed for removing the debris accumulating on the back of the catch wall.	Recutting 360 m^3 x 359 =129 Concrete Wall 112 m^3 x 2,240=251 Grouted Riprap 119 m^3 x 1,326=158 Refilling 183 m^3 x 68 = 12 Gravel Surfacing 24 m^3 x 316 = 8 ₱558	· Special equipment and expertise are not necessary. · Construction period is about 4 months. · During the recutting work (dynamite blasting), road is temporarily closed to traffic, but in shorter period than the above scheme.		· Applicable.