APPENDIX 1

DESIGN METHODS

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1. Slope Damage

1.1 Slope Erosion

1.1.1 Refilling

(1) Refilling of Cut Slope

- Application

The refilling of a cut slope is carried out to eliminate the problem of extensive gully erosion caused by runoff water flowing down the slope or by spring water seeping through to the slope's surface.

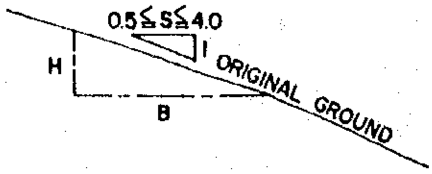
- Materials

Soil, soil cement, sandbags.

- Design & Construction Summary

1. Rock and soil in gullies are first removed and then bench cutting is carried out. In the case of a gully having a steep slope, construction work has to be executed as shown in Table 1.1.1. However, if the stability of a gully will disappear as a result of bench cutting, then such work should not be carried out.

Table 1.1.1 Bench Cutting



(in meter)

S	B	H
4.0~2.0	2.0	0.5~1.0
2.0~0.5	1.0	0.5~2.0

2. The thickness of a single refilling layer should be 20cm.

3. When refilling, the slope gradient, berm pitch, berm width, etc. should be designed and constructed to be the same as the existing slope.

4. The standard gradient, berm width, and slope heights between berms of the refilled portion of cut slope

are determined by the slope's geology as shown in Table 1.1.2.

Table 1.1.2 Structure of Refilled Cut Slope Based on Geological Conditions

Classification \ Geology	Soil	Soft Rock	Hard Rock
Slope gradient	1.0:1 to 1.5:1	0.5:1 to 1.0:1	0.25:1 to 0.5:1
Berm width	2.0 m	2.0 m	1.5 m
Slope height between berm	less than 7.0 m	less than 7.0 m	less than 10.0 m

5. From past experience, the height of a cut slope having berms should be less than 15m. When this condition can not be satisfied, the fill slope materials, berm width, slope gradient between berms, structural work, etc. to be applied should be reconsidered from the design stage.

6. When filling a gully, it can become quite cramped for workers. Therefore, compaction equipment should consist of tampers and rammers as shown in Fig. 1.1.1.

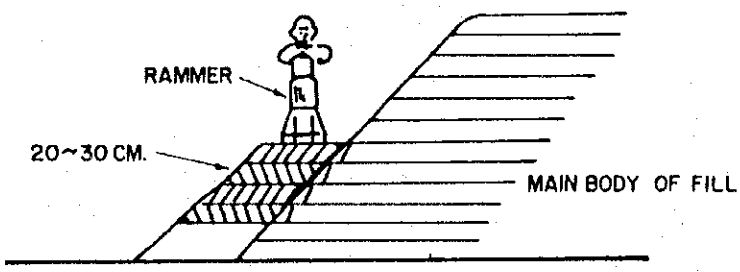


Fig.1.1.1 Slope Compaction with Tamper or Rammer

7. When refilling is difficult to carry out due to the slope gradient, which is predetermined by the composition of the soil, then sandbags will be placed on the slope's surface after refilling to prevent the refilling material from moving (see Fig.1.1.2).

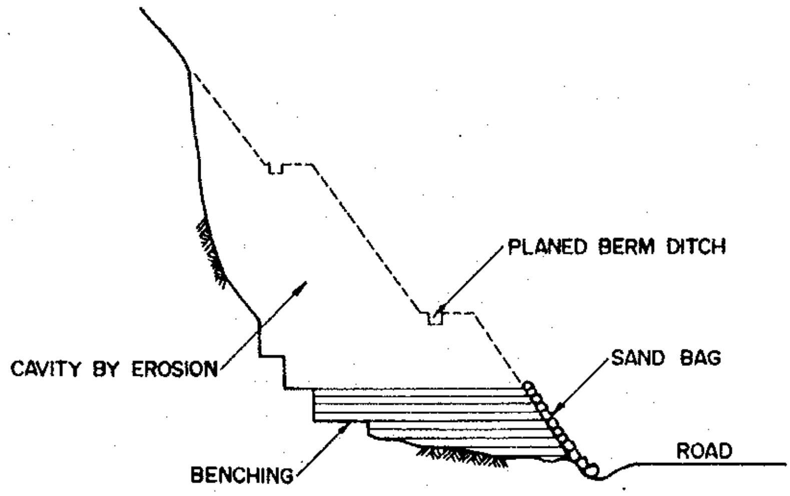


Fig.1.1.2 Protection of Refilling Material with Sandbags

8. To protect a refilled cut slope from erosion and weathering, the conditions of the slope should be confirmed at the design stage and a countermeasure such as surface drainage, subsurface drainage, vegetation, or structural work chosen.

(2) Refilling of Fill Slope

- Application

The refilling of a fill slope is carried out to restore the portion of the slope that has collapsed due to scouring.

- Materials

Soil, soil cement, and sandbags.

- Design & Construction Summary

1. To easily carry out refilling, bench cutting will be applied. When the collapsed portion of a slope has a steep gradient, bench cutting will be executed as indicated in Table 1.1.1.

2. The thickness of a single refilling layer will be 20cm.

3. When refilling, the berm pitch, berm width, and slope gradient between berms should be designed and constructed to be the same as that of the existing slope.

4. The standard gradient, berm width, and the height between berms of the refilled portion of a fill slope are determined by the slope's geology as shown in Table 1.1.3.

Table 1.1.3 Structure of Refilled Fill Slope Based on Geological Conditions

Classification \ Geology	Soil	Soft Rock	Hard Rock
Slope gradient	2.0:1	-	-
Berm width	2.0 m	-	-
Slope height between berm	less than 5.0 m	-	-

5. Based on past experience, fill slopes with berms should be less than 20m in height. If this condition can not be satisfied, the fill slope materials, berm width, slope gradients between berms, and structural work, etc. to be applied should be reconsidered from the design stage.

6. In the case of refilling a fill slope, large-scale compaction equipment, such as that shown in Fig. 1.1.3, is possible to apply. However, in narrow places, it is recommended that smaller equipment be used.

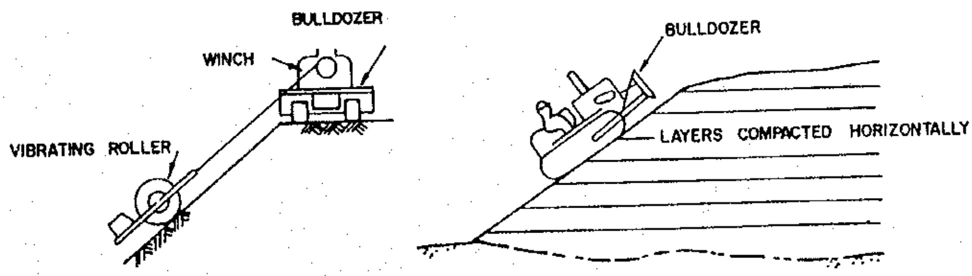


Fig.1.1.3 Slope Compaction with Vibrating Roller & Bulldozer

7. When refilling is being executed on sloping ground, which in many cases contains groundwater, a drainage layer will be included in the construction of each berm as shown in Fig.1.1.4.

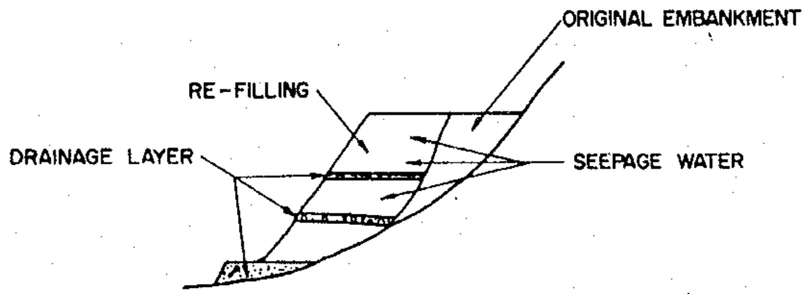


Fig.1.1.4 Berm Drainage Layer for Refilling a Fill Slope

8. To protect a refilled fill slope from erosion and weathering, the conditions of the slope should be confirmed at the design stage and a countermeasure such as surface drainage, subsurface drainage, vegetation, or structural work chosen.

1.1.2 Recutting

- Application

Recutting is carried out to stabilize an unstable cut slope. Instability can be mainly attributed to the following factors:

1. advanced slope erosion,

2. landslides, and

3. the existence of unstable rock on steep slope.

- Design & Construction Summary

1. When recutting is being executed on a stable cut slope, refer to Table 1.1.2 of Section 1.1.1 to decide berm pitch, berm width, and the slope gradient between berms.

2. In the case of erosion, recutting should only be done if the erosion is so advanced that the application of a drainage system, vegetation, and structural work is impossible. However, when there are restraints on right-of-way, cribwork should be applied to ensure a stable slope (see Fig.1.1.5). In this case, the cost effectiveness of construction work should be considered from the design stage. Furthermore, drainage and vegetation work is carried out to prevent erosion.

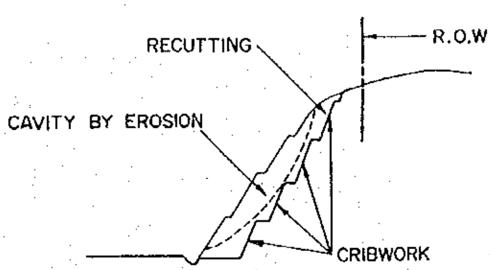


Fig.1.1.5 Cribwork

3. In the case of a landslide, it is very important to use the debris of the landslide to recut a new cut slope with a gradient gentler than that prescribed for the original cut slope. However, even if the new cut slope's gradient is gentler overall, it is still possible that a landslide will reoccur, as a result of the recutting work removing too much material from the foot of the original landslide (see Fig.1.1.6).

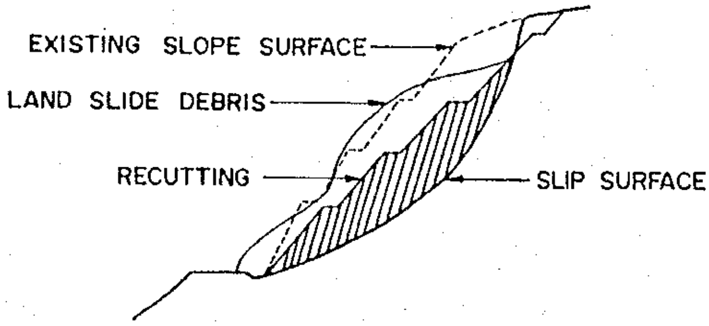


Fig.1.1.6 Trimming of Landslide Debris (1)

For this reason, it is very important to remove debris from the head of a landslide when recutting a cut slope with a gentler gradient.

4. In the case of recutting where there is unstable rock on steep ground, it is very important to decide on a stable slope gradient by considering the hardness of the slope's ground (see Fig.1.1.7). In addition, there should be a survey on the causes for rocks falling, as well as a comparison on the cost effectiveness and technical feasibility of recutting with other construction methods in order to choose the most suitable method.

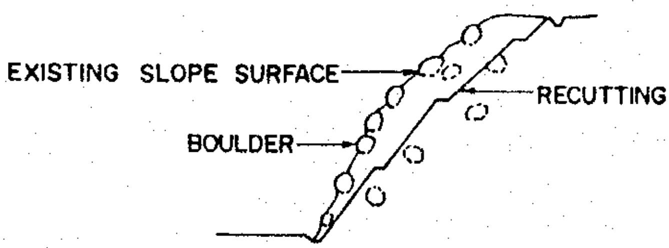


Fig.1.1.7 Recutting with Unstable Rocks Slope

1.1.3 Cribwork

Generally, there are three types of cribwork: 1.precast concrete cribwork, 2.cast-in-place cribwork, and 3.sprayed concrete cribwork. Below, the application, materials, and design work of these three types of cribwork are taken up.

(1) Precast Concrete Cribwork

- Application

Precast concrete cribwork is applied to slopes with a gradient gentler than 1.0:1 and is effective in the following situations:

1. a cut slope having spring water,

2. in the case where there is advanced erosion and weathering due to runoff from rainwater,

3. in the case where a slope's soil is not conducive for vegetation but vegetation is required by the surrounding conditions, and

4. in the case where vegetation work has been carried out but there is still the fear of the slope collapsing.

- Materials

Precast cribwork blocks, concrete blocks for structures, and foam.

- Design

1. The span of a crib is to be 1m. The dimensions of crib members are 15cm x 15cm or 15cm x 20cm.

2. To prevent cribs from sliding, the spaces between the cribs are filled with mortar after anchor pins from 50-100cm are inserted into the ground and the steel wires extruding from the crib ends are tied together tautly (see Fig.1.1.8).

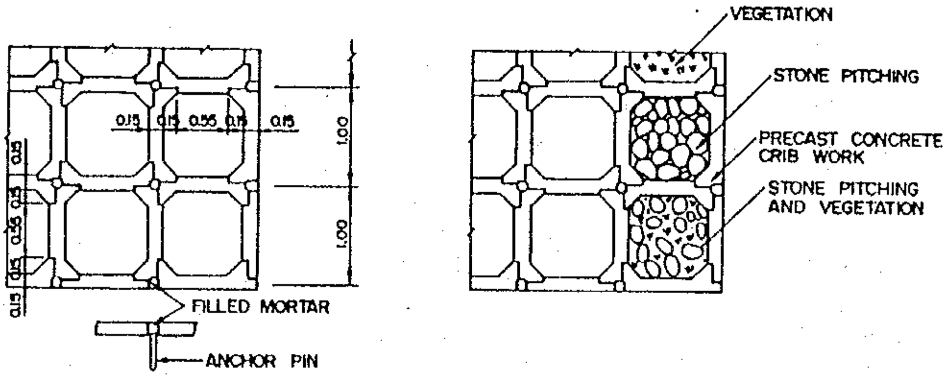


Fig.1.1.8 Sample of Precast Concrete Cribwork

3. Within cribs, new soil rich in nutrients is put in in place to ensure that the new vegetation planted there takes strong hold. Or, as shown in Fig.1.1.9, stone pitching, concrete block pitching, etc. is used if the following conditions exist:

    1. the gradient of the slope is steeper than 1.2:1, and

    2. there is a considerable amount of seepage.

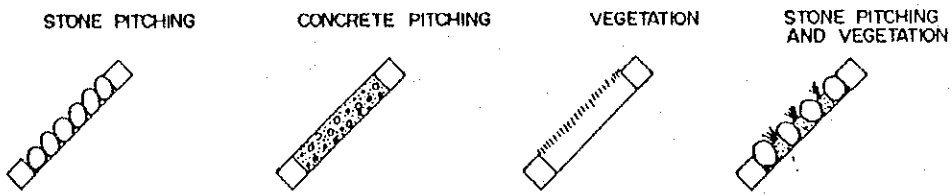


Fig.1.1.9 Sample of Work Inside Crib

4. In the case where there is spring water on a slope, either a branch-like underground drainage system or water-absorption mats are put into place to prevent the soil of the slope from being washed away before precast concrete cribwork is begun.

5. Furthermore, it is necessary to consider at the design stage the sufficient meshing of the cobblestones and the problem of weathered and small stones being washed away.

(2) Cast-in-place Concrete Cribwork

- Application

Cast-in-place concrete cribwork is applied to slopes with a gradient gentler than 1.0:1 and is effective in the following situations:

1. when a slope's future stability is questionable (e.g., weathered rock slope with spring water or large-size slope),

2. when there is a fear that precast concrete cribwork would collapse, and

3. when unstable rock can not be anchored to bedrock by shotcrete because of joints and cracks in the bedrock.

- Materials

Concrete for structures, foam, reinforcing bar.

- Design

1. Cribs will made up of cast-in-place reinforced concrete.

2. The dimensions of crib members will be from 0.3m x 0.3m to 0.6m x 0.6m, with span width being 5 to 10 times that size.

3. Slope gradient will depend on the hardness of the soil, and anchor bars will be used at the intersections of cribs to prevent cribs from sliding (see Fig.1.1.10).

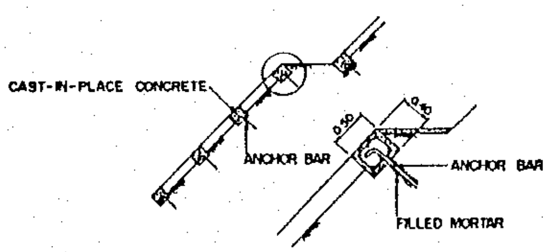


Fig.1.1.10 Cast-in-place Concrete Cribwork

4. Depending on a slope's condition, stone pitching, concrete pitching, stone riprap with mortar, vegetation work, etc. is used to protect the slope.

(3) Sprayed Concrete Cribwork

- Application

1. Sprayed concrete cribwork is applicable to the same situations that cast-in-place concrete cribwork is applicable, but it is easier to apply and can be used on rough slopes. In addition, this type of cribwork can be made to fit the shape of a slope.

2. Applicable to an undulated slope.

3. Sprayed concrete cribwork is highly useful in reinforcing prefabricated light-weight cribs.

- Materials

Concrete used for spraying, light-weight steel-net crib, concrete sprayer.

- Design

1. Cribs will be made on site of reinforced concrete via shotcrete.

2. The members of cribs will be from 0.15m x 0.15m to 0.5m x 0.5m, with span width being 5 to 10 times that size.

3. For crib intersections and members, anchor bars will be applied to prevent cribs from sliding (see Fig. 1.1.11).

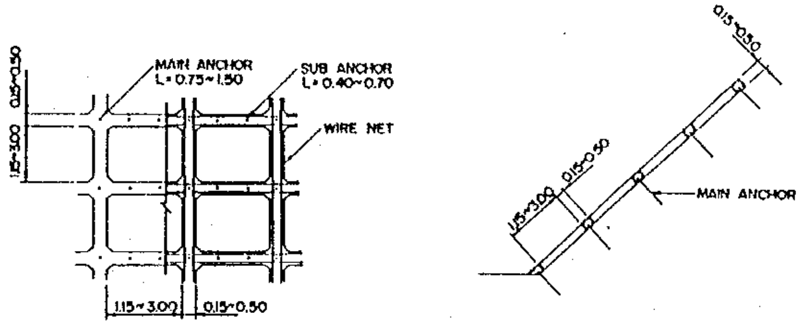


Fig.1.1.11 Sprayed Concrete Cribwork

4. Depending on a slope's condition, stone pitching, concrete pitching, shotcrete, stone riprap with mortar, and vegetation work, etc. will be applied to protect the slope.

5. As for the concrete mixture, it is the same as for shotcrete. That is, the weight ratio of cement, sand, and gravel is 1:3:2 and the water cement ratio is 45%.

1.1.4 Vegetation Work

- Purpose

The purpose of vegetation work is to protect slope surface from erosion to reduce the velocity of runoff water and to beautify an area.

- Application

1. Depending on the condition of a slope, the appropriate type of vegetation work is chosen. However, in the five situations below, vegetation work can not be applied.

1) Where sunlight nor rain can not reach an area because of structures such as bridges, viaducts, etc.

2) Where the acidity of soil is very strong.

3) Where the moisture of soil is extreamly limited.

4) Where the soil is extreamly hard.

5) Where a slope is extremely steep (0.6:1 or less).

2. Regarding the selection of an appropriate type of vegetation work, it is necessary to carry out a survey of the object slope taking into consideration the items below.

1) Slope gradient

When a slope's gradient is less than 1.2:1 in the case of soft rock or clay, or less than 1.5:1 in the case of sand or sandy soil, it is possible to prevent erosion due to runoff or the collapsing of top soil by vegetaion work only.

However, if the gradients for these soil types become steeper, it is difficult to ensure slope stability using vegetation only. Therefore, such measures as cribwork and wicker work must also be carried out.

2) Slope rock, soil, & soil hardness

Using the results of a slope survey, the types of vegetation work that are applicable for the slope can be determined by referring to Table 1.1.4.

Table 1.1.4 Applicable Vegetation Work by Soil Type

Type of work	CUT SLOPE						FILL SLOPE		
	Sand	Sandy soil, Sandy soil with gravel or rock		Clayey soil, Clayey soil wih gravel or rock, Clay		Hard cray, Weathered soft rock, Mudstone	Sandy soil, Sandy soil soft rock, Mudstone	Clayey soil, Clayey soil with gravel or rock	Sand
		Hardness of soil		Hardness of soil					
		≤ 27 mm	27 mm ≤	≤ 27 mm	27 mm ≤				
Block Sodding	A	A		A			A	A	A
Stripe Sodding							A	A	
Seed Packet Work			A		A	A			
Pick-hole Seedling Work			A		A	A			
Seed Spraying with a Pump	B	B		B			A	A	C
Seed-mix Spraying with a Gun	A	A	A	A	A	A			A

A : Applicable geology
B : Applicable for fertile soil with Soil hardness test result < to 23 mm
C : Applicable when laying top soil

Soil hardness is measured to determine whether or not the roots of plants can take hold in a particular type of soil. Usually, for the roots of plants to grow into the ground, it is necessary either that the pores of the soil be larger than the diameter of the roots (generally 0.1mm in diameter) or that the penetrating force exerted by the growth of the roots be greater than the fricative resistance of the soil particles.

To measure soil hardness so that the appropriateness of the different types of vegetation work can be confirmed, a soil hardness tester like the one shown in Fig. 1.1.12 is applied.

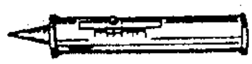


Fig. 1.1.12 Yamanaka-Type Soil Hardness Tester

Generally, the penetrating force exerted by the growth of the roots of a plant is $10kg/cm^2$, which corresponds to 23mm on the above-mentioned soil hardness tester.

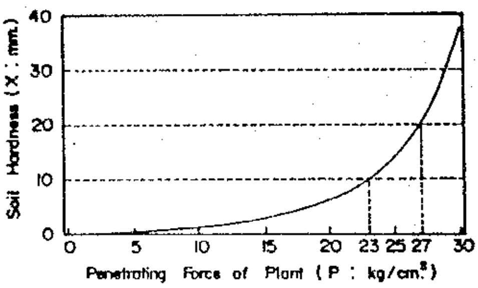
Since there are variations in the pores of soil, it has been decided that 27mm on the soil hardness tester, or a penetrating force of $20kg/cm^2$, is the value that will permit the growth of vegetation. As for the relationship between soil hardness and the penetrating force of a plant, it is shown by the equation and figure below(see Fig.1.1.13).

$$P = \frac{100X}{0.7952(40-X)^2}$$

Where,

P: penetrating force of plant (kg/cm^2)

X: soil hardness (mm)



Soil hardness test result (mm)	Applicable vegetation work
0 ~ 9	Block sodding, spot sodding, stripe sodding Erosion control with local material.
9 ~ 21	Erosion control with local material, Spot sodding.
21 ~ 27	Erosion control with local material, seed packet work.
27 ~ 30	Seed packet work, pick-hole seedling work.
30 ~ Soft rock	Seed spraying with pump, Seed-mixed spraying with a gun.
Soft rock ~ Hard rock	Seed spraying with pump, Seed-mixed spraying with a gun.

Fig.1.1.13 Relation between Soil Hardness & Penetrating Force of a Plant

3. Soil acidity

When soil is highly acidic (around a pH of 4 or less), there are usually adverse effects on vegetation growth. Therefore, the pH of soil should be measured and action taken when it is found to be acidic.

When material used for a fill slope is highly acidic in nature, the alkaline carbonic acidic calcium is mixed with the material to neutralize its acidity.

In the case of a cut slope, it is generally difficult to reduce the acidity of the soil. However, there is the method where new soil is laid, which is mixed with carbonic acidic calcium, and seed packet and pick-hole seedling work carried out.

When there is spring water o either a fill slope or cut slope, a thick spray of seed mix is effective. Here, the seeds are mixed with binder to prevent them from being easily washed away. Another effective measure is cribwork that applies soil and rock.

- Design

1. Selection of Vegetation & Allocation of Seed

For vegetation work, three or more types of vegetation are chosen as candidates after considering weather (temperature, rainfall), soil conditions soil type, gradient, humidity), the growth patterns and lifespan of vegetation, and the purpose of the vegetation work itself.

In the case of grass, it is relatively easy to grow, but it requires large amounts of fertilizer. In addition, its roots are rather shallow and is not so effective in preventing a slope from collapsing. As for trees, their growth is slow but they have deep roots and are able to stabilize a slope well ; also, they do not require fertilizer in most cases.

Based on the above, in the case of preventing slope erosion, it can be said that planting grass is effective as a short-term measure, while planting a combination of trees and grass is effective for the long term since it creates a slope requiring little maintenance.

As for the quantity of grass seed to use, it is determined by the equation below.

$$W = \frac{G}{S \times \frac{P}{100} \times \frac{B}{100}} \times K$$

Where,

W: quantity of seed (grams/m^2)
G: expected number of seed to grow (seed/m^2)
S: average number of seed per gram (seed/gram)
P: purity of seeds (weight ration of net weight of seed and gross weight)
B: the number of seeds per 100 seeds that actually grows
K: correction coefficient for work conditions

As for K, since there is little experience in vegetation work in Thailand, the smallest value used in Japan will be applied and adjusted afterwards based on actual work results.

As for the types of grass to plant, this depends on the location where planting is to take place. Based on tests carried out at these locations, appropriate types of grass are chosen as candidates for planting. To assist in this selection process, the table below can be referred to.

Common Name (Scientific Name)	Purity	Seed Per 100 That Grows
Ruzi Grass (Brachiaria Ruziziensis)	70	50
Common Bermuda Grass (Cynodon Dactylon)	50	30
Creeping Signal Grass (Brachiaria Humidicola)	-	-
Star Grass (Cynodon Plectostachyus SPP.)	-	-
Weeping Love Grass (Eragrostis Parviflora)	50	30
Goose Grass (Eleucine Indica)	40	25
Regume (Stylosanthes Hamata)	70	30

2. Characteristics & Materials by Vegetation Work Type

Below, the characteristics and materials for the different kinds of vegetation work are described.

1) Block Sodding

Outline: As shown in Fig. 1.1.14, blocks of sod are laid tightly next to one another on a slope's surface and anchored down with pins, starting from the top and going down to the toe of the slope. Then, fertilizer and soil is lightly scattered about. Sod blocks should be placed so that the edges of successive layers of block do not correspond with the edges of previous sod block.

Characteristics: Applicable to erosion-prone soil.

Materials: Blocks of sod, sodding anchor pins, fertilizer.

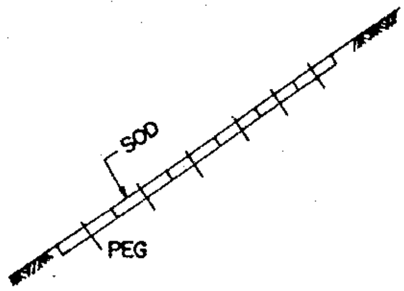


Fig.1.1.14 Block Sodding

2) Stripe Sodding

Outline: As shown in Fig.1.1.15, rectangular sod blocks are placed tightly next to each other to form a row starting from the toe of a slope. After a row of sod blocks has been laid, two-thirds of each block is covered with 30cm of top soil and a new row begun. This results in what is called stripe sodding.

Characteristics: Stripe sodding is used for fill slopes.

Materials: Blocks of sod, top soil.

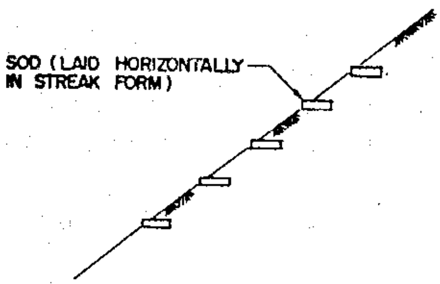


Fig.1.1.15 Stripe Sodding

3) Seed Packet Work

Outline: As shown in Fig.1.1.16, holes are dug at regular intervals to form rows and a packet containing fertilizer, soil, seed, and cut straw put into each hole and fastened with an anchor pin. The distance between each hole is approximately 50cm and about 6 packets per square meter are used.

Characteristics: Seed packet work is applied to cut slopes. Also, because packets are used, the seed, fertilizer, and soil are not easily washed away. Finally, it is possible to carry out seed packet work on relatively steep slopes.

Materials: Polyethylene bags, seed, fertilizer, soil, anchor pins.

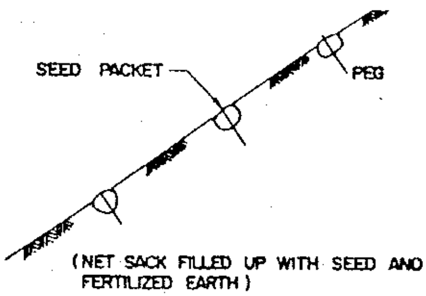


Fig.1.1.16 Seed Packet Work

4) Pick-Hole Seedling Work

Outline: As Fig.1.1.17 shows, after a hole is dug and some fertilizer is put in, a tree seedling is inserted together with a support rod.

Characteristics: Pick-hole seedling work is carried out on cut slopes, with the purpose of protecting the environment and improving the landscape. As for the parts of a slope not protected with seedlings, other types of vegetation work would be carried out.

Materials: Tree seedlings, fertilizer, support rods.

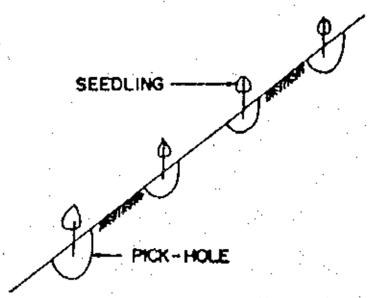


Fig.1.1.17 Pick-hole Seedling Work

5) Seed Spraying

Outline: As shown in Fig.1.1.18, seed, fiber, and binder is mixed with water and sprayed over a slope using a pump.

Characteristics: Seed spraying is applied to fill slopes and to cut slopes with soft soil. This method is efficient and suitable for low places or gentle slopes.

Materials: Seed, high quality fertilizer, fiber (pulp, woody cellulose, etc.), binder (polyvinyl alcohol, polyvinyl acetate,etc.)

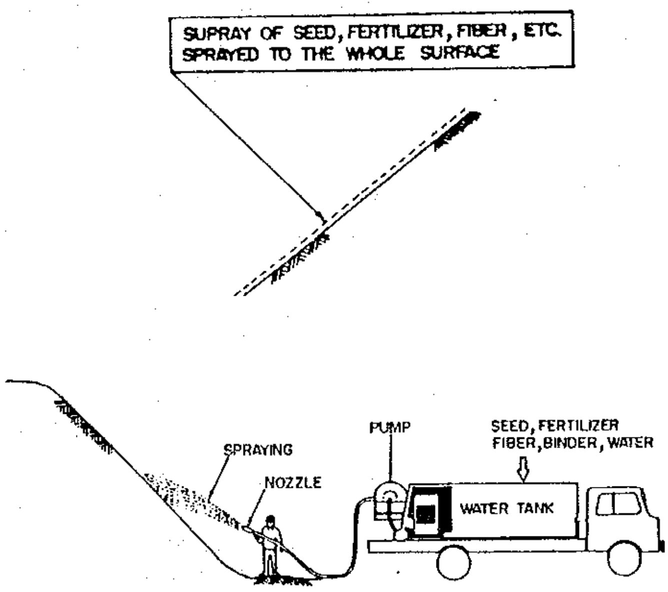


Fig.1.1.18 Seed Spraying

6) Hydroseeding

Outline: As shown in Fig.1.1.19, a mixture of seed, rtilizer, soil, and water is sprayed thickly over a slope with a mortar gun. Then, an anti-erosion agent is sprayed over the slope.

In the case of a slope's surface being soil, the above-mentioned mixture is sprayed until there is a 1 to 2cm layer. In the case of soft rock, the mixture is sprayed on top of a tightly spread net, which is to prevent the mixture from falling off, until there is a 2 to 3 cm layer.

Finally, in the case of the surface being hard rock, this method is in principle not applied. However, when for environmental reasons it is necessary that this process be carried out, the mixture is sprayed over a tightly spread net until there is a layer of 5cm or more. Note that in this case the soil of the original mixture is replaced with binder to ensure that the mixture does not fall off.

Characteristics: Hydroseeding is applied to cut slopes and is capable of spraying a seed, fertilizer, and soil mixture having a high soil content. This method can also be applied to high places and steep slopes, as well as to slopes with gravelly soil.

Materials: Seed, high quality fertilizer, soil (with less than 5% of its contents made up of gravel 6mm or less in diameter and mixed with compost or something similar in nature), anti-erosion agent (asphalt emulsion), binder (Portland cement, high polymer plastics), water, netting.

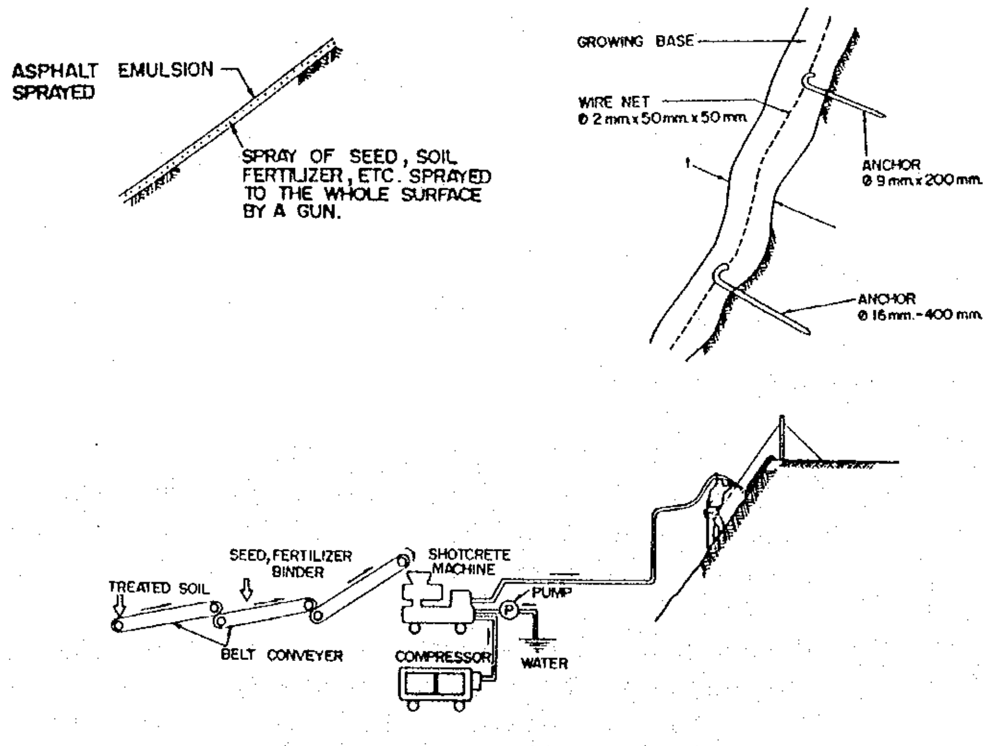


Fig.1.1.19 Hydroseeding

1.1.5 Shotcrete

- Application

1. For slopes with no spring water and easily weathered rock.

2. For slopes with weathered rock about to fall.

3. For slopes with rock having many cracks and joints.

4. For slopes having an overhang.

5. For cut slopes just constructed and that are stable, but where there is the fear that seepage will result in an unstable slope surface.

6. For slopes where mudstone and vegetation work are unsuitable.

- Materials

Spray-type concrete, diamond-shaped wire net (2mm x 50mm), anchor pins (16mm x 40cm), PVC pipe (50mm), concrete spray gun.

- Design

1. Generally, a concrete layer of 10cm is sprayed. In the case where the surface is rock and extremely bumpy, a layer of 15cm is appropriate.

2. To prevent cracking in sprayed concrete after it has dried and to prevent the concrete from peeling off, the concrete will contain diamond-shaped steel wire netting (2mm x 50mm). When necessity requires it, steel bars can be used instead.

3. Before spraying concrete, steel wire netting is attached to the object slope with anchor pins (16mm x 40cm) at intervals of 1 or more per square meter.

4. To construct weep holes, PVC pipe with a diameter of 50mm is used and a weep hole constructed every 2 to 4 square meters.

5. As for the amount of concrete used, there is to be a weight ratio of 1:3:2 for cement, sand, and gravel, with a water cement ratio of 45%.

6. In the case where the area of operation is large and and the slope relatively flat, there is the possibility of the concrete shrinking and cracks appearing. For this reason, as shown in Fig.1.1.20, a vertical expansion joint is placed every 20m to prevent the spread of cracking. In the case of where the rock surface is very bumpy, the number of expansion joints can be reduced owing to the absorption of temperature-induced stress.

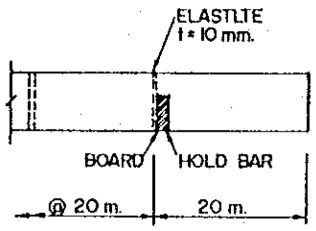


Fig.1.1.20 Vertical Expansion Joint

To prevent seepage after covering a slope's surface with shotcrete, either the partial shotcrete method or the complete shotcrete method (which includes the use of a crest ditch) can be applied (see Fig.1.1.21(1), Fig. 1.1.21(2)).

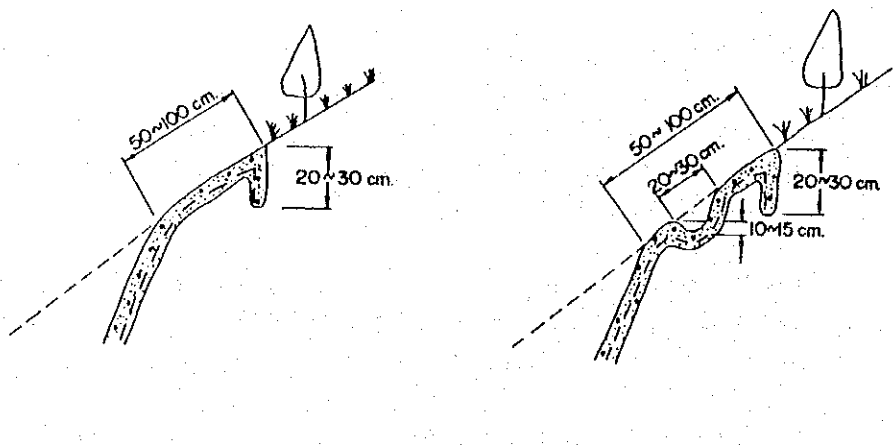


Fig.1.1.21(1) Partial Shotcreting Method

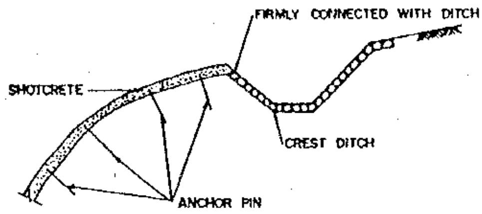


Fig.1.1.21(2) Complete Shotcreting Method

- Construction

1. As for the service life of shotcreting, since it is highly influenced by the weather at the time of construction, it is important that the timing of shotcreting be taken into consideration. For reference, shotcreting is not carried out when any one of the following conditions exists:

    1. a strong wind that will blow away shotcrete,

    2. a strong rain that will wash away shotcrete, and

    3. a strong wind during fine weather causing extreme dryness.

2. As for the construction itself, after pressurized water or air is used to remove unstable rock and dust, wire netting is attached to a slope's rocky surface and shotcreting carried out.

3. When executing shotcreting, only stop when a vertical expansion joint has been reached.

1.1.6 Slope Surface Drainage

- Crest Ditch

1. To prevent rainwater or spring water at the top of a cut slope from seeping into the slope, a crest ditch is constructed at the top of the cut slope. When constructing a crest ditch, it should be sufficiently large, since crest ditches are usually difficult to reach and therefore difficult to maintain (see Fig.1.1.22). In addition, the proper disposal of water in a crest ditch must be considered.

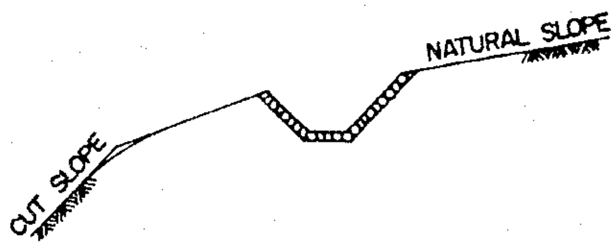


Fig.1.1.22 Crest Ditch

2. As for construction material, stone riprap is suggested.

3. As for the location of a crest ditch at the top of a cut slope, it should be from 1 to 3m away from where rounding was carried out.

- Berm Ditch

1. To prevent erosion caused by rainfall or spring water that flows down a cut or fill slope, berm ditches are constructed at each berm of the cut or fill slope.

2. The longitudinal gradient of a berm ditch is usually made to be the same as that of the adjacent road, but it is desirable if it could be set from 0.3% to 5%. As for the latitudinal gradient, as shown in Fig. 1.1.23, it is 1:15 going in the opposite direction of a slope.

3. For a berm ditch, U-shaped reinforced concrete is used. The function of a berm ditch is to connect

crest and vertical ditches to assist in the disposal of rainwater or spring water.

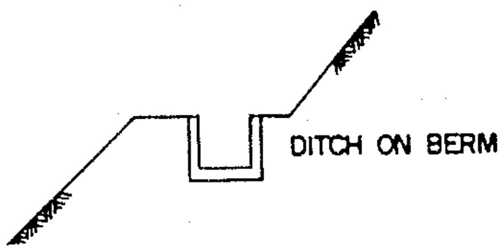


Fig.1.1.23 Berm Ditch

4. As shown in Fig.1.1.24, to facilitate the flow of rainwater on a slope with hard rock, the gradient is set at 5% in the direction of the slope.

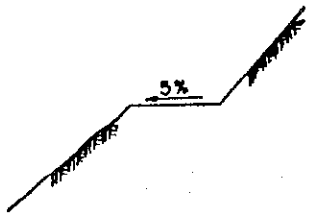


Fig.1.1.24 Berm Structure in Case of Slope with Hard Rock

- Toe Ditch

A toe ditch is constructed to prevent the scouring of a slope's toe by rainwater and the intrusion of rainwater onto a road's surface. The structure of a toe ditch for a cut slope is as shown in Fig.1.1.25. As for the toe ditch of a fill slope, its cross section is determined by the size of the object slope.

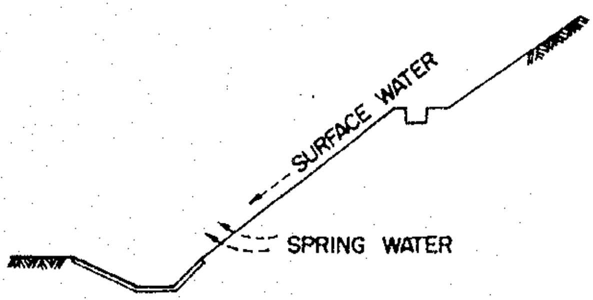


Fig.1.1.25 Toe Ditch

- Gutter

A gutter, as shown in Fig.1.1.26, is constructed at the shoulders of a road to dispose of water on the road's surface. In addition, gutters are supposed to prevent water from flowing down the surface of a fill slope. A gutter is only constructed if the fill slope is 6m or higher.

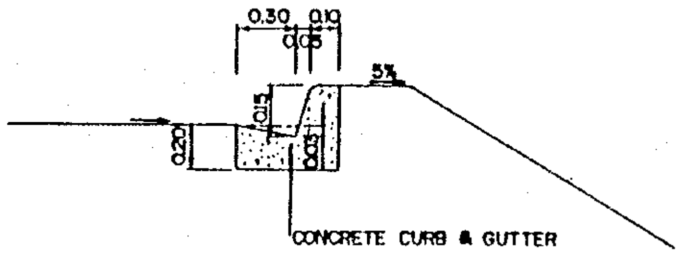


Fig.1.1.26 Gutter

- Vertical Ditch

Vertical ditches are constructed to channel water from the gutters of a road to the toe ditch of a fill slope and/or to channel water from the crest or berm ditches of a cut slope to the toe ditch, taking into consideration slope gradient. Vertical ditches are also constructed if the capacity of berm ditches is insufficient or if the topology warrants it. However, even if the capacity of crest ditches, berm ditches, and gutters is sufficient, if the length of berm ditches is more than 100m, then vertical ditches are constructed every 100m or less, as shown in Fig.1.1.27. As for materials, U-shaped reinforced concrete or stone riprap is used.

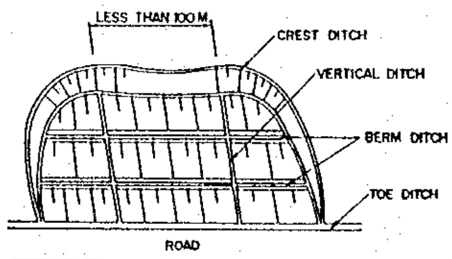


Fig.1.1.27 Vertical Ditch

- Surface Drainage

1. Surface drainage is constructed to prevent the re-occurrence of a landslide by preventing the seepage of rainwater into previous landslide debris. This is done by collecting the rainwater in the area of a landslide and disposing of it outside that area.

2. Surface drainage is made up of numerous horizontal and vertical sections that cut up and down the entire length of a landslide debris mass.

3. As for the materials used, stone riprap is applied. As shown in Fig.1.1.28, in the case of a horizontal ditch the stone riprap is extended against the flow of rainwater, while in the case of a vertical ditch stone riprap is extended on both sides of the ditch. This is done to protect the slope.

4. As for the number of horizontal and vertical sections to construct, in the case of horizontal sections this should be decided by the topography, although one every 10m is a desirable goal. In the case of vertical sections, this should be decided by calculating the amount of rainfall with a 20% safety margin, although it is suggested that vertical sections be within 100m of each other.

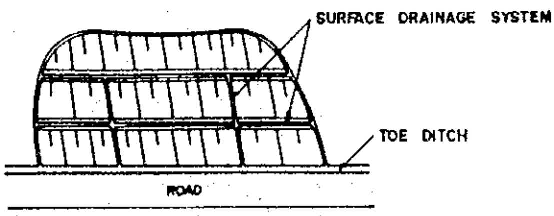


Fig.1.1.28 Surface Drainage System

1.2 Rockfalls

1.2.1 Removal of Unstable Rock

The purpose of this method is to remove rock that is in danger of falling. two typical methods are shown below.

1. Use of manpower

   In this case, loose rock that is inaccessible to large construction machinery is broken off with a breaker, which then falls down to the road. As Fig. 1.2.1 indicates, this work is done in high places and requires restrictions on traffic for safety reasons. This type of work also usually requires a lot of time.

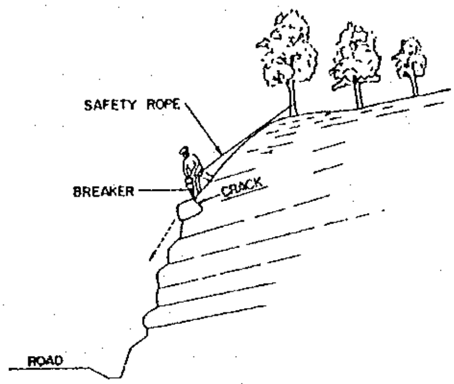


Fig.1.2.1 Removal of Loose Rock via Manpower

2. Use of construction machinery

   In this case, loose rock that is accessible to large construction machinery is removed from a slope using a back hoe (see Fig.1.2.2). This work usually requires a small amount of time to carry out.

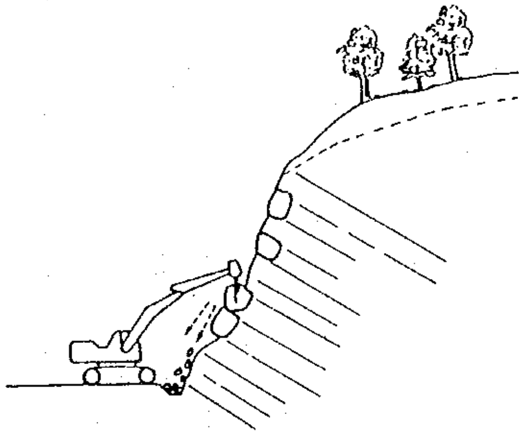


Fig.1.2.2 Removal of Loose Rock via a Back Hoe

When selecting a method to remove unstable rock, it should be based on surveys of those places experiencing rockfalls and the method most suitable for each location chosen.

1.2.2 Rockfall Prevention Net

- Application

1. A rockfall prevention net is used for cut slopes that are being scoured, weathered, or eroded by rain, with there being the probability of a rockfall occurring.

2. In addition to trying to prevent rockfalls, it is also important to try to prevent rocks from falling onto a road and to guide them to the toe of a slope.

3. To prevent rocks from falling onto a road, there are two types of rockfall prevention nets (see Fig. 1.2.3):

    1. a cover-type rockfall prevention net (which is made of wire netting, wires, anchor concrete blocks, and anchor bolts) that prevents rocks from falling by trapping them between a cut slope and the wire netting, and

    2. a pocket-type rockfall prevention net (which is made of hanger wire, netting, steel supports, and anchor concrete blocks) that catches falling rock into a pocket-like area by absorbing the force of falling rock via the wire netting.

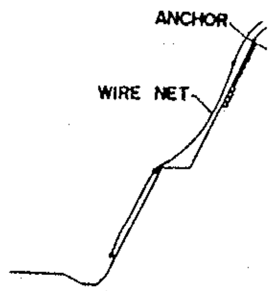


Fig.1.2.3 Types of Rockfall Prevention Nets

- Materials

1. Wire net (diameter: 2.6mm - 4.0mm, netting holes: 50mm x 50mm)

2. Wire 3 x 7 galvanized and stranded with petroleum grease: main wire with maximum tensile strength of

12,000kg (diameter: 16mm), sub-wire with a maximum tensile strength of 7000kg (diameter: 12mm)

3. Anchor bolts 1.5m in length (diameter: 25mm)

4. Anchor concrete blocks (1m wide, 1.2m deep, 2.0m long in direction of rope).
5. H-shaped steel support (pocket-type rock prevention net only).

- Design

(1) Cover-Type Rockfall Prevention Net

The general approach for designing the cover-type rock-fall prevention net is shown by the flow chart in Fig. 1.2.4.

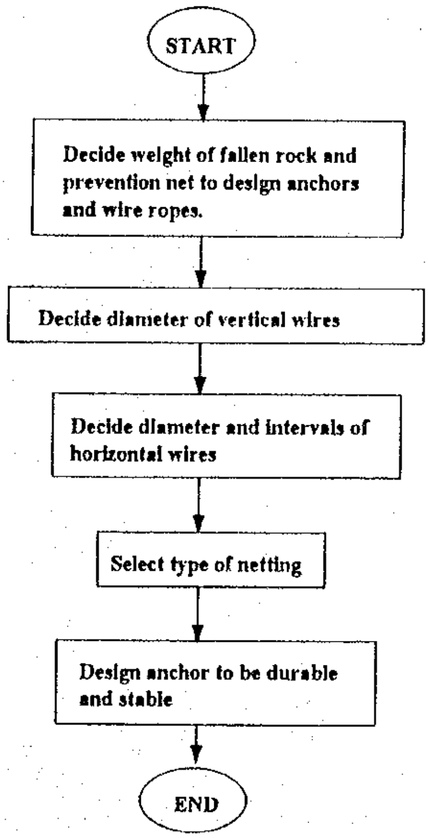


Fig.1.2.4 Design Approach for Cover-Type Rockfall Prevention Net

As for actually designing the net, the seven steps below

below are used to accomplish this.

1. A vertical wire, as shown in Fig.1.2.5, has to support the weight of fallen rock for a slope face width of l together with its own weight. The safety factor, as shown in Table 1.2.1, is set at twice the breaking load of a vertical wire.

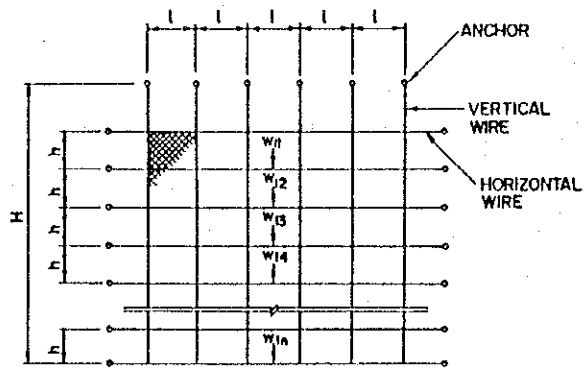


Fig.1.2.5 Calculation of Vertical Wire

Table 1.2.1 Breaking Load of Vertical Wire

Wire rope's diameter	Breaking load
16 mm	12 t
12 mm	7 t
8 mm	3 t

2. The load (WI) acting on a vertical wire is as follows:

$$W1 = W11 + W12 + \dots W1n$$
$$W2 = wN \cdot (\Sigma(l \cdot h))$$
$$WI = W1 + W2$$

Where,
W1: the total weight of fallen rock from sections W11 to W1n with slope width l and length h.
W2: the total weight of the prevention net for each l x h.
wN: the unit weight of the prevention net.
WI: load acting on a vertical wire.
l: length of horizontal wire span.

h: length of vertical wire span.

3. As for the load acting on a horizontal wire, as shown in Fig.1.2.6, it is uniformly distributed and consists of the weight of the prevention net and fallen rock for an area 3h x 1. Furthermore, it is assumed that the sag of the horizontal wire is 1/10 of its span.

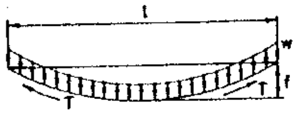


Fig.1.2.6 Load Acting on Horizontal Wire

$$w = \frac{WII}{l}$$

$$WII = W'1 + W'2$$

$$T = \frac{w \cdot l^2}{8f}$$

Where,

W'1: the weight of fallen rock for the area 3h x l (see Fig.1.2.5).

W'2: the weight of the net for the area 3h x l (see Fig.1.2.5).

WII: load acting on a horizontal wire.

4. As for the load acting on the wire netting, the approach is the same as that for the horizontal wire. Regarding the effective tension for wire netting 1m in width, it is 0.5 of the yield strength of the wire netting and is as shown in Table 1.2.2.

Table 1.2.2 Effective Tension Of Wire Net

Strand diameter (mm)	Effective tensioning force (t/m)
4.0	2.09
3.2	1.33
2.6	0.89

5. As for the actual load acting on a vertical or horizontal wire, it is also dependent on the gradient of a slope (see Fig.1.2.7). In the equation below, this is taken into account to correct the values for the loads acting on the vertical and horizontal wires in order to obtain the actual load values.

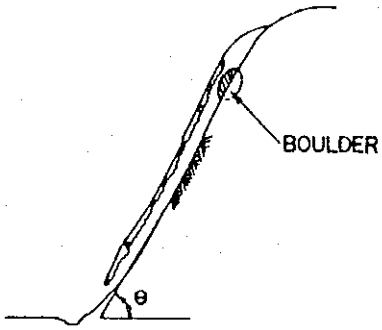


Fig.1.2.7 Slope Gradient

$$W'' = (\sin\Theta - \mu\cos\Theta)\cdot W$$

Where,

W": actual load acting on either a vertical or horizontal wire.

W: WI or WII (load acting on vertical or horizontal wire).

Θ: gradient of slope.

μ: friction coefficient for friction between rock & slope (approx. 0.5).

6. In the case of an anchor, both the loads of the vertical and horizontal wires act upon it, and calculations on intensity and stability should be carried out.

7. As for combined coil, which connects the wire net with the vertical and horizontal wires, it is important that it have the required ductility to sufficiently fulfill this function. As Fig.1.2.8 indicates, it is also important that the combined coil be properly applied to ensure a tight connection between the vertical and horizontal wires with the wire net.

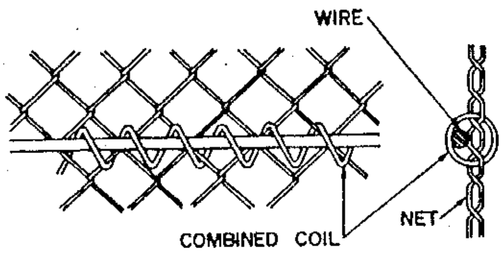


Fig.1.2.8 Combined Coil

(2) Pocket-Type Rockfall Prevention Net

The general approach to designing a pocket-type rockfall prevention net is shown by the flow chart in Fig.1.2.9.

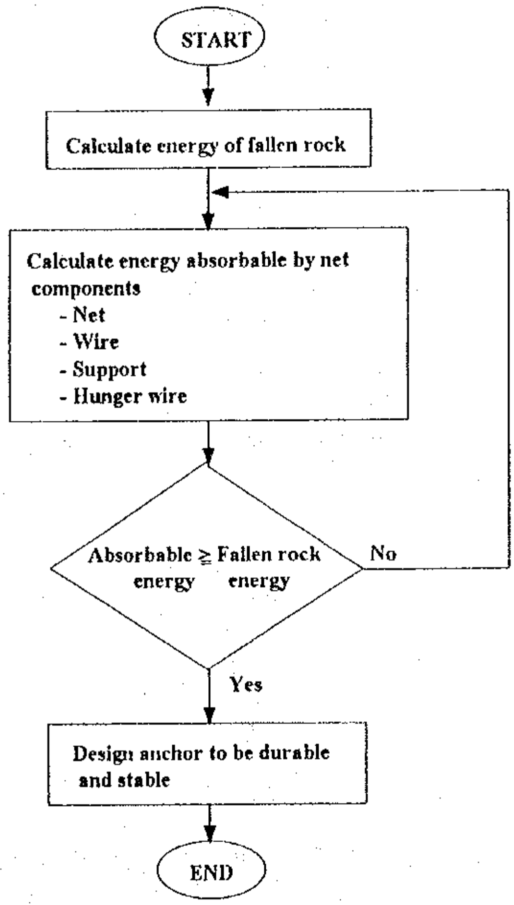


Fig.1.2.9 Design Approach for Pocket-Type Rockfall Prevention Net

As for designing the steel support, wire net, anchor, etc. of the pocket-type rockfall prevention net, this is done as described by the steps below.

1. The area where rocks usually strike is between the middle horizontal wire and upwards. However, as shown in Fig.1.2.10, as result of leeway in the amount of energy that the vertical and horizontal wires can absorb, it has been decided to designate the area where rocks will most likely strike as that between the top and second horizontal wire in the middle of a steel support interval.

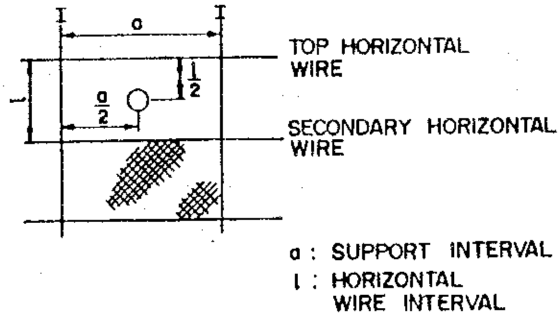


Fig.1.2.10 Location to Be Struck by Rock

2. As for the direction of falling rock, as shown in Fig.1.2.11, it is almost parallel to an object slope. If the influence of sudden jumps by a rock is taken into account, it can be said to be parallel.

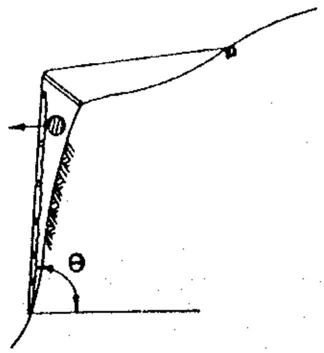


Fig.1.2.11 Direction of Falling Rock

3. The energy that a falling rock conveys to a pocket-type rockfall prevention net is calculated by the equation below, taking into consideration the angle of the net and assuming that rock strikes the net at a 90°angle.

$$E_W = \frac{1W(v\sin\Theta)^2}{2g}$$

Where,

E_W: energy conveyed from a falling rock to the rockfall prevention net.

W: weight of falling rock.

v: speed of falling rock.

Θ: net angle.

g: acceleration due to gravity.

4. The calculation of the potential absorption energy of the rockfall prevention net is done with the following equation:

$$E_T = E_N + E_R + E_P + E_{HR} + E_L$$

Where,

E_T : potential absorption energy.

E_N : absorption energy of net.

E_R : absorption energy of vertical and horizontal wires.

E_P : absorption energy of steel support.

E_{HR}: absorption energy of hanger wire.

E_L : difference in energy before and after the collision of a rock.

5. When calculating the absorption energy of the net, net deformation at maximum load is considered to be approximately 1/2 of span length. However, when taking into consideration the play in the net, this figure should be set at 1/4 of span length. The tensile stress of the net is as shown in Fig.1.2.12.

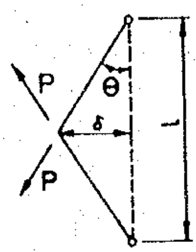


Fig. 1.2.12 Tensile Stress of Net

Where,
P: tensile stress of net.
δ: displacement in net.
l: distance between wires.

Given the above, $\delta = 1/4$ and $\tan\Theta = \delta/(1/2) = 1/2$. Based on this, the absorption energy of the net E_N is equal to the following:

$$E_N = 2P\sin\Theta \cdot \delta = 0.22P \cdot l$$

Here, the diameter of falling rock is expressed as D, while the width of the net struck by a rock is assumed to be 1.5D. Therefore, P can be said to be equal to 1.5D·p, where p is net intensity per meter. Results are given below in Table 1.2.3.

Table 1.2.3 Weight of Fallen Rock & Tensioning Force of Net

Fallen rock's weight (t)	Diameter (m)	P (t/m)	P (t)
0.3	0.596	2.79	2.49
0.5	0.707	"	2.96
1.0	0.891	"	3.73
1.5	1.020	"	4.27
2.0	1.123	"	4.70
2.5	1.209	"	5.06

6. As for calculating the absorption energy of horizontal wire, the tensioning force of the wire net is conveyed to the top horizontal wire and the two horizontal wires below that. Therefore, the absorption energy of horizontal wire is equivalent to the sum of the absorption energy of these wires.

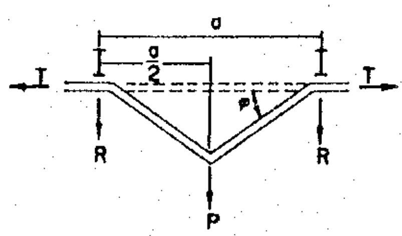


Fig.1.2.13 Tensioning Force of Horizontal Wire

Where,
a: steel support interval.
T: tensioning force of net (P) conveyed to wire.
R: force acting on steel support.

Given the above, the following is true:

$$R = P/2, \quad T\sin\phi = R$$
$$\cos\phi = a/(a + (T \cdot L)/(E \cdot A)$$

Where,
E: modulus of elasticity for horizontal wire.
A: section area.
L: length of wire.

For horizontal wire, 3 x 7 G/O with a diameter of 16mm is used. For the case where L = 30m, a = 3m, and $E = 1 \times 10^{6k} kg/cm^2$, the relationship between P, R, and T is as shown in Table 1.2.4.

Table 1.2.4 Weight of Falling Rock & Tensioning Force of Horizontal Wire

Fallen rock's weight	P (t)	R (t)	T (t)
0.3	2.49	1.25	4.41
0.5	2.96	1.48	4.94
1.0	3.73	1.87	5.78
1.5	4.27	2.14	6.34
2.0	4.70	2.35	6.76
2.5	5.06	2.53	7.11

Given the above, the absorption stress of horizontal wire can be calculated by the equation below.

$$E_R = L(T^2 - T_o^2) / (E \cdot A)$$

Where,
E_R: absorption energy of wire.
E: modulus of elasticity for wire.
A: section area.
L: length of entire wire.
T: tensioning force of wire.
T_o: initial tensioning force of wire (usually 500kg).

7. When calculating the absorption energy of the steel support and hanger wire, the forces acting on the steel support is as shown in Fig.1.2.14

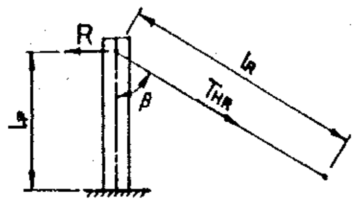


Fig.1.2.14 Forces Acting on Steel Support

Where,
R: force conveyed by wire.
T_{HR}: tensioning force of hanger wire.

Based on this, E_R and E_{HR} can be calculated by the steps below.

1. In the case when the base of a steel support is fixed, the horizontal vector forces (W_P) and hanger wire tensioning force (T_{HR}) are as shown below.

$$WP = \frac{3E' \cdot I \cdot l_R \cdot R}{l_P{}^3 \cdot E \cdot A + 3E' \cdot I \cdot l_R}$$

$$THR = \frac{E \cdot A \cdot l_P{}^3 \cdot R}{l_P{}^3 \cdot E \cdot A + 3E' \cdot I \cdot l_R} \operatorname{cosec}\beta$$

Where,
E': modulus of elasticity for steel support.
I: geometrical moment of inertia of steel support.
l_P: height of steel support.
E: modulus elasticity for hanger wire.
A: area of hanger wire section.
l_R: length of hanger wire.

The stress acting on the bottom portion

of a steel support (δ) is calculated using W_P (horizontal vector forces) and $T_{HR}\cos\beta$ (perpendicular compression). Its design value is set by making it larger than the steel support's yielding stress (δ_y). E_P and E_{HR} are calculated using the equations below.

$$E_P = \frac{W_P{}^2 \cdot l_P{}^2}{3E' \cdot I}$$

$$E_{HR} = \frac{l_R}{E \cdot A}(T_{HR}{}^2 - T_O{}^2)$$

2. In the case where the steel support foundation is a hinge, T_{HR} is designed so that its value will be between $R\cos\beta$ (inclusive) and T_Y (non-inclusive). In this case E_P and E_{HR} are calculated applying the equations below.

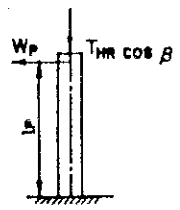


Fig.1.2.15 Acting Force to Support

$$E_P = 0$$

$$E_{HR} = \frac{l_R}{E \cdot A}(T_{HR}{}^2 - T_O{}^2)$$

3. To calculate the difference in energy before and after the collision of a rock into a rockfall prevention net, the equation below is applied.

$$E_L = \frac{W_2}{W_1 + W_2}E_W$$

Where,

E_L: the difference in energy before and after the collision of a rock.

E_W: the kinetic energy of a falling rock.

W_1: weight of falling rock.

W_2: weight of prevention net.

4. Intensity of anchor

Anchor bolts are drilled into a rock foundation and should be designed so that their shearing stress can resist the tensile stress of wire. In addition, anchor bolts should be designed so that they are safe from being pulled out of the ground by tensile force. Below, anchor bolt intensity (R), which resists the shearing force on anchor bolts, is calculated for bolts 22mm and 25mm in diameter (ϕ). Here, the allowable shearing force (σ_A) of an anchor bolt is $800kg/cm^2$ multiplied by a safety factor of 1.5, or $1200kg/cm^2$.

22mm ϕ bolt

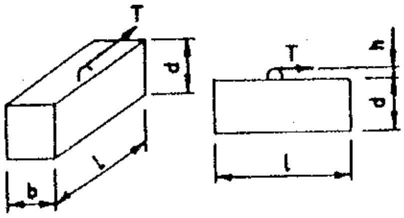
$$R = \underset{(Area)}{3.03cm^2} \cdot \underset{(\sigma A)}{1200kg/cm^2} = 3636kg$$

25mm ϕ bolt

$$R = \underset{(Area)}{3.87cm^2} \cdot \underset{(\sigma_A)}{1200kg/cm^2}$$

As for anchor blocks, which are buried in the soil, sliding, overturning, and bearing capacity have to be considered. The specifications of an anchor block in relation to the tensile force of horizontal wire is as shown in Table 1.2.5.

Table 1.2.5 Shape of Anchor Block



Type	T (t)	b (m)	l (m)	d (m)	h (m)	Comments	
A	6.0	1.0	2.0	1.2	9.03	Internal friction angle of soil	Ø = 30°
B	4.0	0.9	1.6	"	"	Active earth pressure coefficient of soil	ka = 0.3
						Passive earth pressure coefficient of soil	kp = 3
C	3.0	0.8	1.4	"	"	Friction coefficient between soil and concrete	μ = 0.55
D	2.5	0.7	1.3	"	"	Allowable bearing capacity	q = 15 t/m²
						Unit weight of soil	rs = 1.8 t/m³
E	2.0	0.8	1.2	1.0	"	Unit weight of concrete	rc = 2.3 t/m³

1.2.3 Rockfall Prevention Barrier

There are basically three types of rockfall prevention barriers: fence, wall,and a combination of fence and wall. In the case of wall barriers, there are earth-fill, mat-gabion, and concrete types. As for fence, it is almost always made of concrete.

(1) Fence

- Application

Fence is effective for stopping small rocks from reaching a road's surface. There are two kinds of fences: a wire mesh fence and a H-shaped steel fence. These fences are shown below in Fig.1.2.16 and Fig.1.2.17.

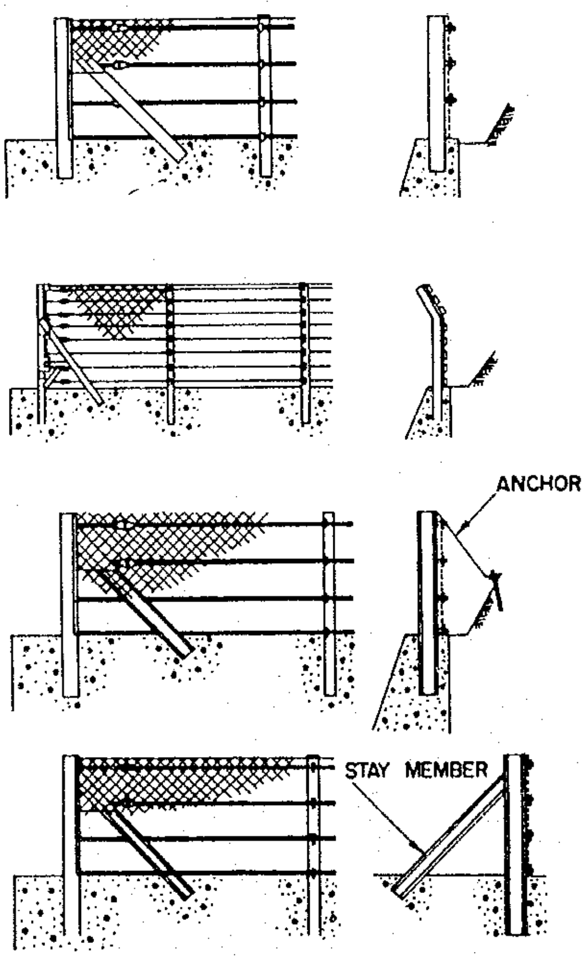


Fig.1.2.16 Wire Mesh Fence

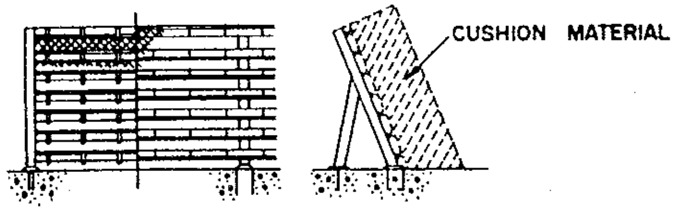


Fig.1.2.17 H-Shaped Steel Fence

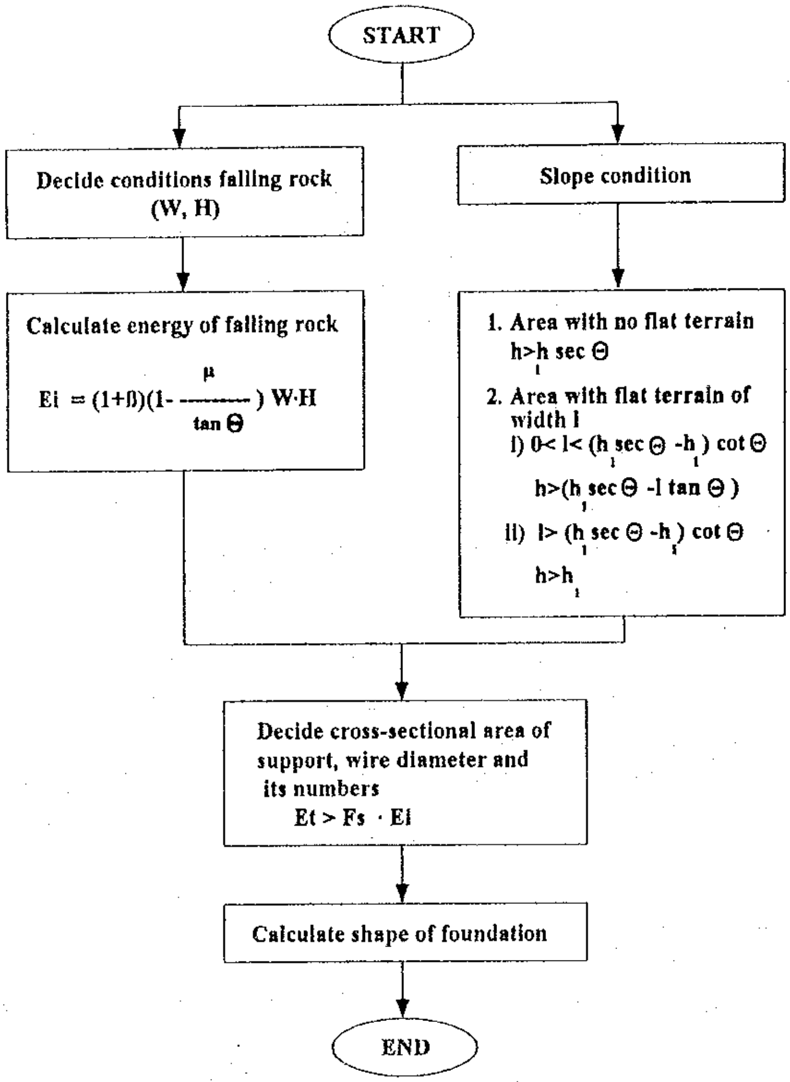
As Fig.1.2.16 indicates, wire and wire mesh are strung between H-shaped steel supports in the case of wire mesh fence. In addition, there are two types of wire mesh fence, i.e., fence with straight and curved supports.

As Fig.1.2.17 indicates, H-shaped steel beams are laid between H-shaped steel supports with wire mesh attached on top in the case of the H-shaped steel fence. In addition, usually old tires or sand is used as cushion material.

- Design

1. Wire Mesh Fence

   A flow chart for designing a wire mesh fence is shown in Fig.1.2.18.



E_i : energy of falling rock
W : weight of falling rock
H : height of falling rock
Θ : slope gradient
μ : coefficient of friction by falling rock

β : coefficient of rotational energy
h_1 : bouncing height by falling rock
l : width of area with flat terrain
Fs : safety factor
Et : potential absorption energy

Fig.1.2.18 Flow Chart for Designing Wire Mesh Fence

For a fence barrier's allowable displacement to absorb the energy of falling rock, the cross section and allocation of members, as well as the stability of the fence barrier's foundation, must be examined.

As for the height of a fence barrier, this depends on how far falling rock bounces. If h_1 is the 90° distance a rock bounces from a slope, then the height of a barrier given a slope gradient of Θ can be calculated by the expressions below.

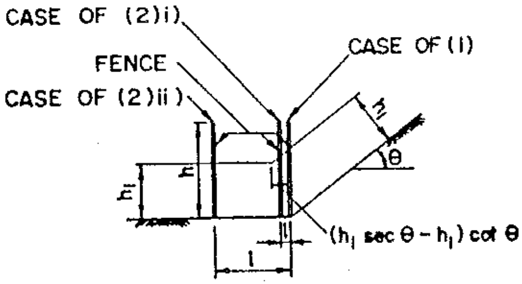


Fig.1.2.19 Conditions for Calculating Distance Falling Rock Bounces

1. Area with no flat terrain: $h > h_1$

2. Area with flat terrain of width l:
   When $0 < l < (h_1 \sec\Theta - h_1)\cot\Theta$: $h > h1\sec\Theta - l\tan\Theta$
   When $l > (h_1 \sec\Theta - h_1)\cot\Theta$: $h > h_1$

The load used to design a fence barrier consists of only the energy from falling rock. The direction that falling rock hits a fence barrier is assumed to be 90° (see Fig.1.2.20).

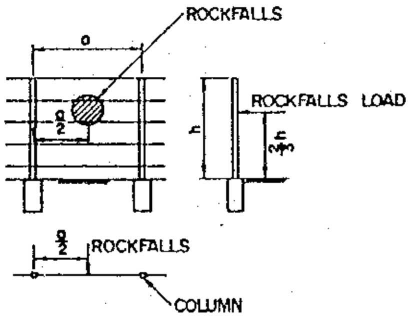


Fig.1.2.20 Direction of Falling Rock

The value used for the energy of a falling rock in design is calculated with the equation below.

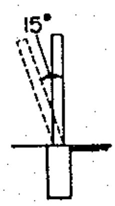


Fig. 1.2.21 Allowable Maximum Displacement

$$E_i = (1 + \beta) \cdot (1 - \mu/\tan\Theta) \cdot WH$$

Where,
E_i: energy of falling rock.
μ: uniform friction coefficient of falling rock.
Θ: slope gradient.
β: coefficient for turning energy of rock
W: weight of rock.
H: distance rock falls.

The allowable maximum displacement of a support is set at 15° (see Fig.1.2.21). The potential absorption energy of a wire mesh fence is calculated in the equation below.

$$E_T = E_R + E_P + E_N$$

Where,
E_T: potential absorption energy of wire mesh fence.
E_R: absorption energy of wire.
E_P: absorption energy of steel supports.
E_N: absorption energy of wire mesh.

The calculation of the absorption energy for supports and wire is carried out in the four steps below.

1. The yielding stress T_y for wire is obtained from Table 1.2.6.

Table 1.2.6 Yielding Stress of Wire

Wire rope's diameter (mm)	Cross-sectional area (cm^2)	Yield tensioning force (t)
Ø 18	1.29	12
Ø 16	1.01	9
Ø 14	0.78	7.5
Ø 12	0.59	5.3

2. The reaction R on a support when the yielding stress T_y is acting on wire is calculated in the equation below. Here, it is assumed that two wires are resisting the force of a falling rock.

$$R = 2T_y \sin\Theta_1$$

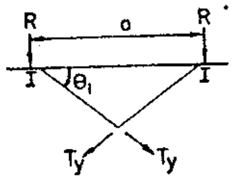


Fig.1.2.22 Deformation of Wire

$$(a/2 + (T_y \cdot L)/2E_w A)\cos\Theta_1 = a/2$$

Where,
a: distance between supports.
L: length of an entire wire.
E_w: modulus of elasticity.
A: cross-sectional area of wire.

3. The force F_y needed to attach a plastic hinge to the bottom of a support is calculated using Fig.1.2.23 and the equation below.

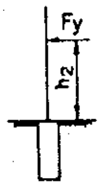


Fig.1.2.23 Height Where Falling Rock Strikes

$$F_y = M_o/h_2 = \sigma_y \cdot Z/h_2$$

Where,
M_o: moment of plasticity.
h_2: height where falling rock strikes.
σ_y: yielding stress of H-shaped steel support.
Z: cross-sectional area of H-shaped steel support.

4. Comparing R and F_y, the following two calculations are made:

1. When $R \geq F_y$

$$E_p = 2F_y \cdot \delta = 2F_y \cdot h_2 \cdot \tan 15° = 0.54h_2 \cdot F_y$$
$$E_r = L(T^2 - T_o{}^2)/E_w \cdot A$$

Where,
T is solved by the next two equations and has the same tensioning force as F_y.

$T = F_y/2\sin\Theta_2$.
$(a/2+T \cdot L/2E_w \cdot A)\cos\Theta_2 = a/2$.
T_o: initial tensioning force (usually 500kg).

2. When $R < F_y$

$$E_p = R^2 \cdot h_2{}^3/3E_H \cdot I$$

Where,
E_H: modulus of elasticity of H-shaped steel support.
I: geometrical moment of inertia of H-shaped steel support.

$$E_R = 2T_y \cdot L \cdot S$$

Where,
S: the stretching ratio of wire when R equals F_y.
$S = T_y/E_w \cdot A$ ($\leq$ 5%).

$$E_N = 2.5\text{tm}.$$

2. H-Shaped Steel Fence

The thinking is the same as that of the wire mesh fence.

3. Foundation

1. Bending Moment

As shown in Fig.1.2.24, the bending moment is calculated by assuming that Point A, which represents the halfway mark of the embedded part of a support, is the focus for turning.

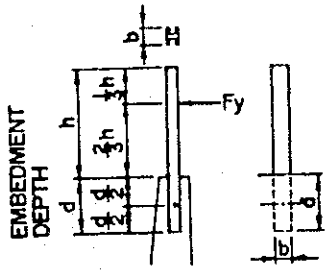


Fig.1.2.24 Installation of Support

$M = F_y(2/3h+d/2)$.
σ: $F_y/A+M/Z \leq \sigma_a$.
$A = b \cdot d$.
$Z = bd^2/6$.
σ_a: allowable bearing stress of concrete.

2. Shearing Force

Shearing force is calculated in the manner indicated in Fig.1.2.25.

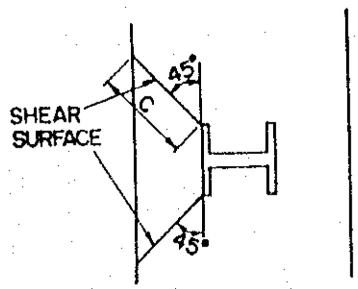


Fig.1.2.25 Distribution of Shearing Force

$t = F_y/2cd \leq t_a$.
t_a = allowable shearing stress of concrete.

(2) Wall

- Application

Wall barriers are relatively effective for preventing medium-size rock from reaching a road's surface, but require a relatively wide area near a road for installation.

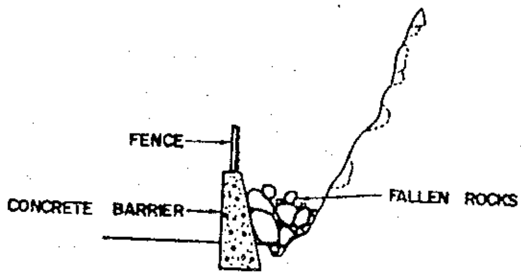


Fig.1.2.26 Wall Barrier

- Design

Since it is difficult to determine the value for external force when a falling rock hits a wall barrier, a dynamics model is constructed and external force calculated given the following conditions:

1. The wall barrier is a rigid body supported by an elastic foundation, and horizontal displacement and rotation occur in the wall when a rock strikes until the kinetic energy of the falling rock equals the deformation energy of the foundation.

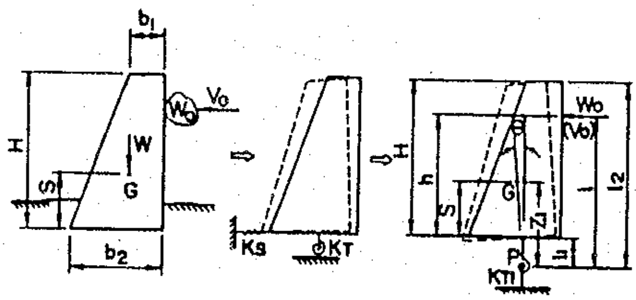


Fig.1.2.27 Dynamics Model

2. The case of a single falling rock colliding with a wall barrier is considered.

3. The effective resistance length of a wall is 4 times

its height. In cases where this is not possible, the actual length of the wall would be its effective resistance length.

4. In design, the angle at which a falling rock strikes a wall barrier is assumed to be 90°.

- Calculation of External Force

a)Calculate distance(Z_1) between rotational center(R) and gravity center(G) of wall

$$m = W/g$$

$$I = \frac{m}{3} \cdot \left(\frac{b_1{}^2+b_2{}^2}{2} + H^2 - \frac{b_2 - b_1}{2(b_1+b_2)} \cdot H_2 - \frac{(2b_1+b_2)^2 H^2+(b_1{}^2+b_2{}^2+b_1 \cdot b_2)^2}{3(b_1+b_2)^2}\right.$$

$$Io = \frac{b_2{}^3 \cdot L}{12}$$

$$A = b_2 \cdot L$$

$$S = \frac{H}{3} \cdot \frac{2b_1+b_2}{b_1+b_2}$$

$$Kv = 0.4\alpha \cdot Eo \cdot Bv^{-3/4}$$

$$Ks = \frac{A \cdot Kv}{4}$$

$$Kr = Io \cdot Kv$$

$$io^2 = I/m$$

$$eo^2 = Kr/Ks$$

$$Z1 = \frac{1}{2S}(S^2+eo^2-io^2)+\sqrt{(}\frac{1}{4S^2}(S^2+eo^2-io^2)^2+io^2)$$

where,

W : weight of wall
g : gravity acceleration
I : geometrical mormrnt of inertia for gravity center of wall
Io: geometrical mormrnt of inertia for bottom

surface of wall

L : length of wall

b_1: top portion width of wall

b_2: bottom portion width of wall

H : height of wall

A : bottom cross-sectional area of wall

S : distance between bottom surface and gravity center of wall

Kv: elastic modulus of ground

α, Eo, Bv : same as above

Ks: same as above

Kr: same as above

b) Calculate velocity(V) of wall at collision point A

$$l_1 = Z_1 - S$$

$$l_2 = l_1 + H$$

$$\alpha' = \frac{4(b_2 \cdot l_2 - b_1 \cdot l_1)(l_2^2 + l_1 \cdot l_2 + l_1^2) - 3(b_2 - b_1)(l_2 + l_1)(l_2^2 + l_1^2)}{6l^2(b_1 + b_2)H}$$

$$V = \frac{Wo}{Wo + \alpha \cdot W} Vo$$

where,

l1: distance between bottom surface and rotational center(C)

l2: distance between top and rotational center(P) of wall

l : distance between collision point(A) and rotational center(P)

Wo: weight of fallen rock

Vo: velocity of fallen rock

c) Calculate rotational angle Θ and bottom displacement of wall by falling rock collision

$$K_{r1} = Ks(eo^2 + l_1^2)$$

$$\delta_s = \frac{Wo \cdot l^2}{K_{r1}}$$

$$\delta_d = \sqrt{\left(\left(1+\alpha \frac{W}{\quad}\right) \frac{\delta_s \cdot V^2}{\quad}\right)}$$

Wo g

$$\Theta = \delta_d / l$$

$$\delta_b = \delta_d - h \cdot \Theta = \delta_d (1 - h/l)$$

where,

K_{r1}: elastic modulus of groung
δ_s : static displacement of point A
δ_d : dynamic displacement of point A

d) Calculate bending morment(M) and horizontal force(H) by falling rock collision

$$M = K_{r1} \cdot \Theta$$
$$H = Ks \cdot \delta b$$

- Calculation of Stability

As in the case of any wall, when calculating wall barrier stability, the stability of the spread foundation has to be considered. Here, stability is studied via the three items listed below, and the safety factor is as shown in Table 1.2.27:

1. examination of foundation bearing capactiy,

2. examination of overturing, and

3. examination of slipping.

Table 1.2.7 Safety Factor and Load

		Normal stability	When falling rock strides	Accumulation of debris
Safety factor	Bearing of ground	3	1.5	1.5
	Sliding	-	1.5	1.5
Overturning		B/6	B/3	B/3
Lord condition		 - Dead weight	 - Dead weight - Falling rocks load -Debris pressure	 - Dead weight - Debris pressure

1.3 Landslide

1.3.1 Calculation of Slope Stability

(1) Present Safety Factor Values

The safety factor for a slope where a landslide is predicted to occur or for the debris after a landslide has occurred is determined by on-site inspection of the soil, boring, etc. Generally, the safety factor (F_s) for when a landslide is occurring is said to be around 0.90. For reference, the safety factor of landslide debris after a landslide has occurred is as described below.

1. In the case of when the soil has stopped moving and the slope is balanced: $F_s = 1.00$.

2. In the case of when sliding is still occurring but only slightly: $F_s = 0.95$.

3. In the case of when there is large sliding: $F_s = 0.90$

(2) Required Safety Factor

The required safety factor is said to be the value needed to stabilize the location where a landslide has occurred. Its value is determined by considering the landslide-induced damage and/or injuries suffered by road users and road facilities, as well as the adverse socioeconomic impacts of the landslide. The value of the required safety factor should be set as described below.

1. For urgent repair work:$1.0 <= Fs <= 1.05$.

2. For long-term measures:$1.05 <= F_s <= 1.20$.

(3) Analysis of Landslides

Based on the results of on-site surveys, the direction, depth, and planar surface of a landslide or possible landslide are estimated.

1. Slope & Landslide Debris Slices

To calculate stability, slopes that might have a landslide and locations covered with landslide debris are divided into slices. As shown in Fig.1.3.1, the slicing is carried out at changes in the topography.

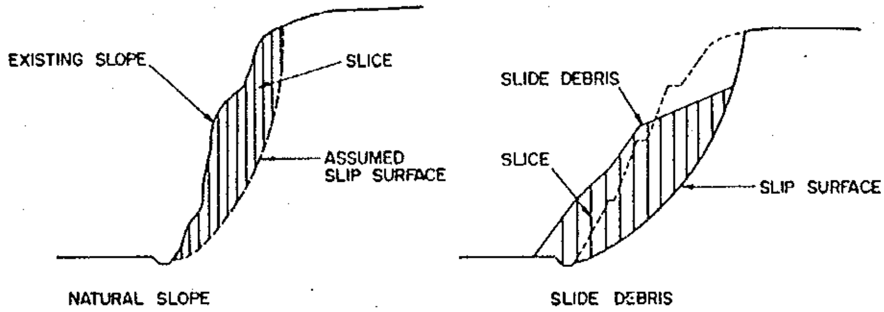


Fig.1.3.1 Slicing of Slope & Slide Debris

2. Estimation of Landslide Face

The direction of a landslide, as shown in Fig.1.3.2, is in most cases along the maximum angle of a slope. Furthermore, the slip surface of many landslides is parallel to their slopes. As for the shape of a landslide's slip surface, in most cases it is circular, and most stability calculations are carried out for this case.

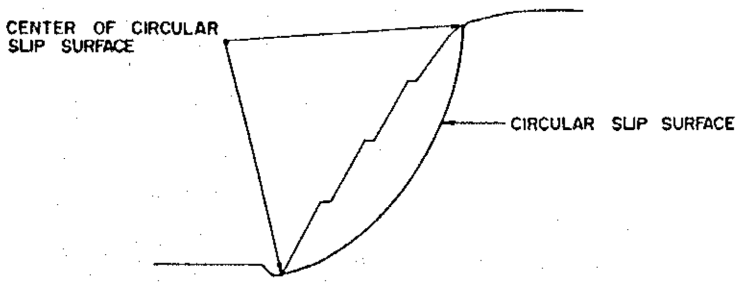


Fig.1.3.2 Hypothetical Slip Surface

3. Pore Water Pressure of Soil

The pore water pressure of a hypothetical slip surface is fixed at the groundwater level value of a bore hole. When carrying out restoration work during heavy rains, since it is in many cases difficult to execute boring, the groundwater level is estimated by making the assumptions below.

1. When normal groundwater level is near or under the slip surface, the groundwater level during

heavy rains is assumed to raise from 10 to 15m.

2. When normal groundwater level is 3 to 5m above the slip surface, the groundwater level during heavy rains is assumed to raise from 5 to 7m.

3. When normal groundwater level is within 5m of boring hole, the groundwater level is assumed to reach the surface during heavy rains.

(4) Calculation of Stability

1. Soil Mechanics Constant

When the soil mechanics constant can not be calculated, the relation between cohesion and internal friction can be applied (see Fig.1.3.3).

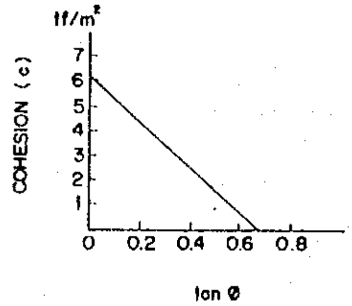


Fig.1.3.3 Relation between Cohesion & Internal Friction Angle

As for cohesion (C), for purposes of convenience, it is set by using the average of the slip surface and slide debris vertical thicknesses (see Table 1.3.1).

Table 1.3.1 Relation between Average Vertical Thickness & Cohesion

Average vertical thickness (m)	Cohesion (t/m²)
5	0.5
10	1.0
15	1.5
20	2.0
25	2.5

Regarding the strength of a weathered slip surface, it as indicated in Table 1.3.2.

Table 1.3.2 Strength of Weathered Slip Surface

Classification		Cohesion	Internal friction angle
Metamorphic rock		0 ~ 0.2	20 ~ 28 (26)
Igneous rock		0	23 ~ 36 (29)
Sedimentary rock	Palaeozoic	0 ~ 0.4	23 ~ 32 (29)
	Mesozoic	0 ~ 1.0	21 ~ 26 (24)
	Palaeogene	0 ~ 2.0	20 ~ 25 (23)
	Neogene	0 ~ 2.5	12 ~ 22 (12.5)

However, in the case where groundwater is near a slip surface, pore water pressure raises. Therefore, for safety reasons, cohesion should be assumed to be zero.

Here, the unit volume of landslide debris is set at $1.8t/m^3$. However, in the case of areas having Shirasu and large rock, or porous and volcanic alterated rock, it is desirable that on-site tests be carried out.

2. Calculation of Stability

As Fig.1.3.4 indicates, the model used to calculate stability does this by dividing up a possible slip surface or slide debris mass into slices.

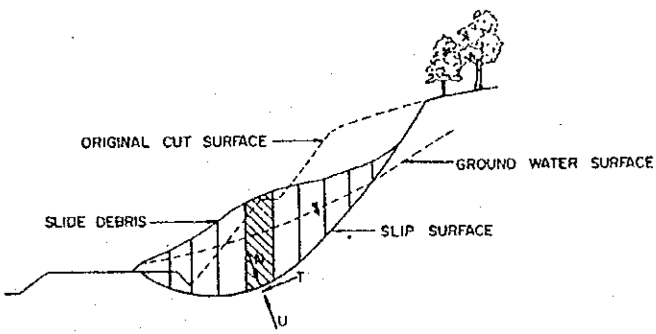


Fig.1.3.4 Stability Calculation Model

The basic equation for estimating stability is shown below.

$$F_S = \frac{\Sigma \{c \cdot l + (W \cdot \cos\alpha - \mu \cdot l)\tan\phi\}}{\Sigma W \cdot \sin\alpha}$$

Where,

F_S: safety factor.
c: cohesion (tf/m^2).
ϕ: shear resistance angle (°).
l: total length of slip surface (m).
W: weight of a slice (tf/m).
α: angle between a slice and slip surface (°).

In Reference Material I, calculations were carried out with a computer software package. In this material, use of the package and the concept of the stability calculations are explained.

1.3.2 Counterweight

As shown in Fig.1.3.5, counterweight work has a heavier unit volume weight than debris at the bottom part of a landslide mass, and it is made up of mat gabion, etc. that can drain spring water in order to prevent future landslides.

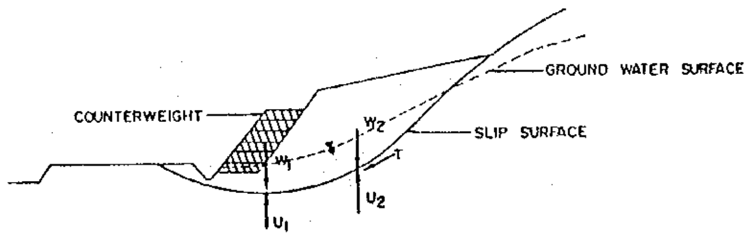


Fig.1.3.5 Counterweight Work

When carrying out counterweight work, the weight of counterweights can result in the ground foundation collapsing (see Fig.1.3.6).

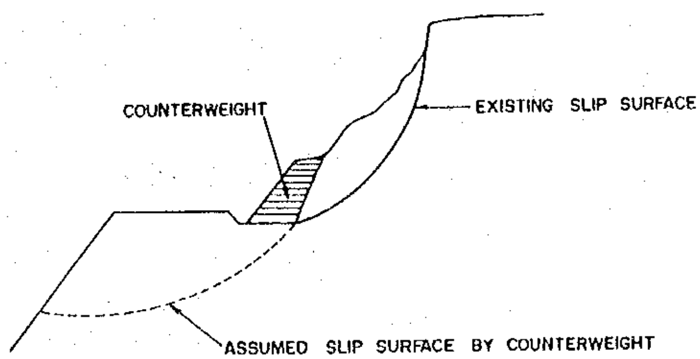


Fig.1.3.6 Collapse of Ground Foundation

Therefore, when executing counterweight work, it is necessary to first carry out stability calculations for slide debris. Then, after confirming the level of safety of the debris, construction and restoration work is begun to address present landslide damage and to prevent future landslides.

1.3.3. Removal of Slide Debris

One of the most reliable methods to remove landslide debris is as shown in Fig.1.3.7. Generally, this method is used for small- and medium-scale landslides.

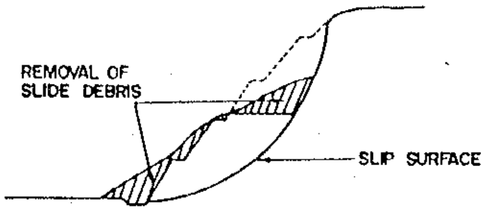


Fig.1.3.7 Removal of Landslide Debris

When disposing of landslide debris, it is necessary first to ensure that it is done under the required level of safety, by making debris stability calculations based on the results of a survey that determines the scale of the landslide, its distribution, and soil strength.

When disposing of debris, either part or all of it can be disposed of. Generally, when partial disposal is carried out, the top part of a debris mass is removed. The thinking behind debris disposal is explained below.

♦ Landslide Debris Disposal Method

1. Location of debris to be disposed: Debris at the top of a landslide mass will be disposed of. However, if debris at the bottom is extremely weak, then soil will be removed from there as well to achieve a safe balance. Therefore, stability calculations shall be carried out to determine the safety factor.

2. Slope gradient at debris removal site: The slope gradient at a debris removal site at the top of a debris mass shall be, as shown in Fig.1.3.8, a gentle 2.0:1 - 4.0:1.

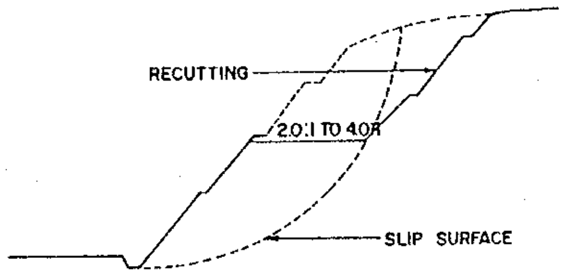


Fig.1.3.8 Slope Gradient at Debris Removal Site

3. Slope work after debris removal: After landslide debris is removed, a slope usually becomes susceptible to seepage and erosion due to rainwater. Therefore, as shown in Fig.1.3.9, drainage and vegetation work shall be carried out.

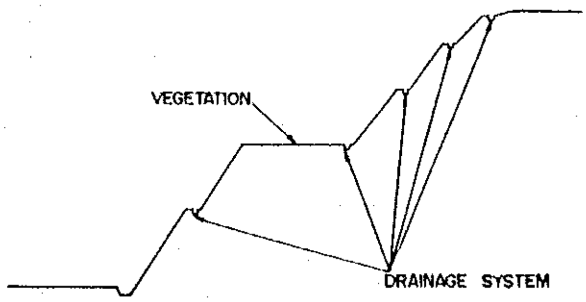


Fig.1.3.9 Slope Work after Landslide Removal

1.3.4 Retaining Wall

(1) Type

A retaining wall is a structure supported by earth to prevent the collapse of soil. Furthermore, it is a structure used during fill-slope or cut-slope work when a soil slope is unable to maintain its stability because of restrictions on right-of-way , topography, etc.

A retaining wall can be divided up into the following types, based on the type of materials used and shape of the wall:

1. gabion wall,
2. stone riprap wall,
3. gravity-type retaining wall,
4. T-shaped retaining wall, and
5. crib retaining wall.

A comparison of these walls is presented in Table 1.3.3.

(2) General Design

- Foundation

The foundation is set using a good bearing stratum of soil, and is able to withstand forces via the ground at its bottom.

Table 1.3.3 Type of Retaining Wall

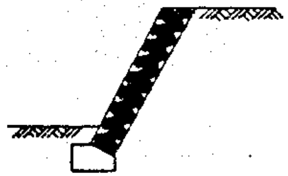
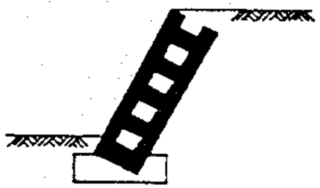
Type	Shape	Recommendable height	Characteristics	Suitable application to :	Economic viability
Gabion Wall		- Less than 3 m high.	- To protect slope from slide, resisting earth pressure.	- Slope protection. - Where the earth pressure to small.	- Viable.
Stone Riprap Wall		- Less than 5 m high.	- To protect slope from slide, resisting earth pressure.	- Slope protection. - Where the earth pressure is small.	- Viable.
Gravity Type Retaining Wall		- Less than 3 m high.	- Easy of construction.	- Where the foundation ground is stiff, not needing pile foundation.	- Viable.
T-Shaped Retaining		- 3 to 10 m high.	- Applicable to high structure taking advantage of earth on footing for stability.	- There the foundation ground is poor.	- Relatively Viable.
Crib Retaining Wall		- Less than 10 m high.	- To protect slope from slide, resisting earth pressure with precast concrete block crib.	- Where the cut slope is much spring water.	- Unviable.

Table 1.3.4 The Bearing Capacity of Foundation Ground by Soil Type

Classification		Allowable Bearing Capacity (t/m²)	Coefficient of Friction between Bearing Ground and Bottom Slab	Remarks	
				qu (t/m²)	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	High density	60	0.6	—	—
	Low Density	30	0.6	—	—
Sandy	High density	30	0.6	—	30-50
	Reasonable density	20	0.6	—	15-30
Clay	Very firm	20	0.5	20-40	15-30
	Firm	10	0.45	10-20	8-15
	Reasonable firm	5	—	5-10	4-8

As shown in Fig.1.3.10, the embedment depth of the foundation must be 50cm or more.

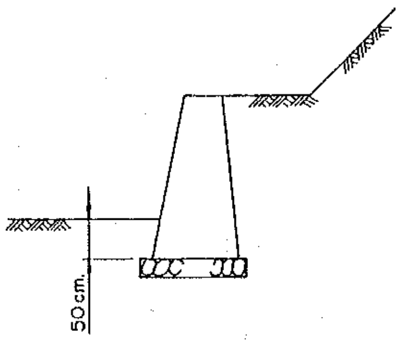


Fig.1.3.10 Embedment Depth of Foundation

In the case when a foundation is to be placed on the top of weak ground, the weak layer of ground is removed and replaced with good quality soil such as crusher-run (see Fig.1.3.11).

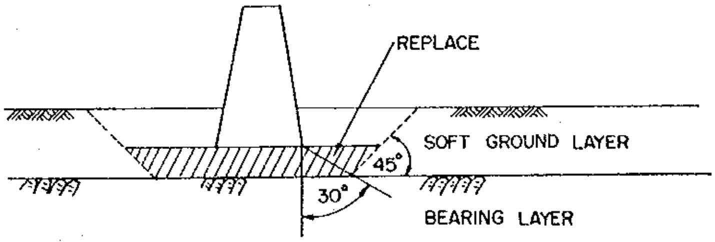


Fig.1.3.11 Replacement of Weak Ground

To increase the sliding resistance of a foundation, it is necessary to take appropriate measures for the bottom of the footing in relation to the foundation ground (see Fig.1.3.12).

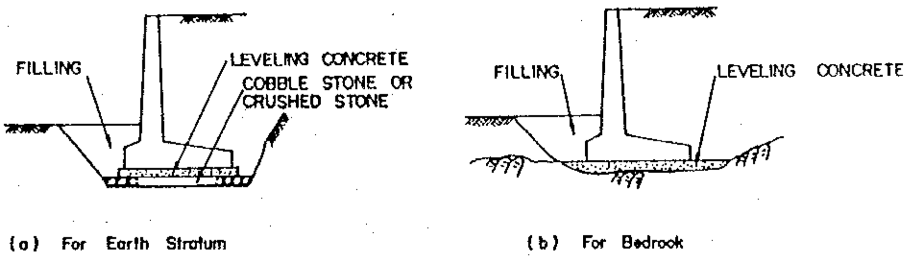


Fig.1.3.12 Measures for Bottom of Footing

As shown in Fig.1.3.12, in the case of the foundation ground being made of soil, cobble or crushed stone is laid at the bottom of the footing to ensure sufficient axial stress. In the case of the foundation ground being bedrock, the face of the bedrock is cut, washed and the footing placed on top of it. Furthermore, as shown in Fig.1.3.13, in the case of sloping ground containing some weak soil, the weak soil is removed and replaced with concrete.

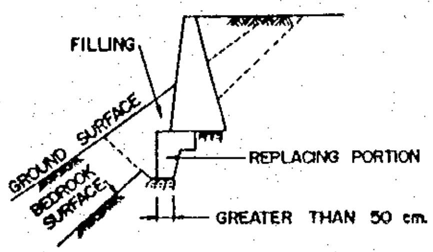


Fig.1.3.13 Replacing Part of Foundation

The bottom of the part to be replaced is cut horizontally with a width of at least 50cm. Furthermore, when the height of the replacement concrete becomes high, a step-like configuration is applied. Finally, stability calculations are carried out and consider the replaced portion.

- Concrete Work

In the case of a T-shaped retaining wall, the footing and wall should be cast together. If this is difficult to do, then they can be cast separately as shown in Fig.1.3.14. Taking into consideration the design of the wall, work should end for the footing and begin for the wall 10cm from the top of the footing.

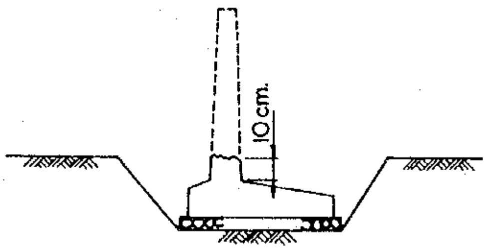


Fig.1.3.14 Construction Joint

In the case of a T-shaped retaining wall or gravity-type retaining wall, since cracking is concentrated on the wall's surface, it is necessary to apply a crack inducing joint as shown in Fig.1.3.15. A crack inducing joint is installed every 10m or less, and the reinforcing of the joints must be continuous.

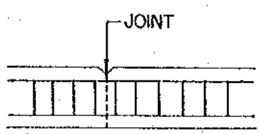


Fig.1.3.15 Crack inducing Joint

The configuration necessary for an expansion joint is as shown in Fig.1.3.16.

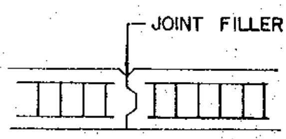


Fig. 1.3.16 Expansion Joint

The intervals for installing an expansion are as follows:

1. 15 to 20m for a T-shaped retaining wall, and

2. 10m or less for a gravity-type retaining wall.

- Backfill Work & Drainage System

The backfill materials for a retaining wall for a fill shall be the same as those used for the subgrade and embankment sections. As for the rolling compaction of backfill materials, sufficient rolling compaction shall be achieved by applying the same machinery used for the subgrade and embankment sections.

In the case of there being difficulty with axial stress for the back of a retaining wall installed at a cut, good quality soil shall be used. Also, the thickness of a strata of backfill will be 20 to 30cm, and sufficient axial stress achieved using tampers and rammers.

A drainage system is to be installed at the back of a retaining wall. The materials to be used should be the highly permeable cobblestone (ϕ: 10mm to 50mm). The thickness of the cobblestone should be 20cm to achieve sufficient drainage. In addition, a drainage pipe every $2m^2$ should be laid.

- Stability Calculations

- Load -

The loads acting on a retaining wall consist of the dead load, surcharge, earth pressure, bouyancy, impact, and water pressure loads. The loads to be used in design are the dead load, surcharge load and earth pressure load.

- Dead Load -

The dead load is the weight of the structure itself and the weight of the earth on the foot slab.

The unit weight of the material to be used in design is indicated in Table 1.3.5

Table 1.3.5 Unit Weight of Material

Materials	Unit Weight (t/m^3)
Reinforced Concrete	2.4
Concrete	2.4
Gravel, Gravelly soil, Sand	2.0
Sandy soil	1.9
Silt, Clayey soil	1.8

- Surcharge Load -

The surcharge load to be used in design is a live load and its value (q) is 1.0 t/m^2.

- Earth Pressure Load -

The earth pressure load acts on the bank of a retaining wall. This load is calculated by the trial wedge shape method. However, in the case of an embankment with a level backfill, it is easy to calculate the coefficient of the earth pressure load by the Coulomb's method.

The caracteristics of a backfill, which are be used in these calculations, is preferably determined by a soil test.

When designing a retaining wall less than 8.0 m in height, however, the values of the internal friction angle are as shown in Table 1.3.6.

Table 1.3.6 Internal Friction Angle

Type of backfill	Ø (degree)
Gravel with sand	35
Sandy soil	30
Silt	25

The concept of the trial wedge shape method is shown in Fig. 1.3.17.

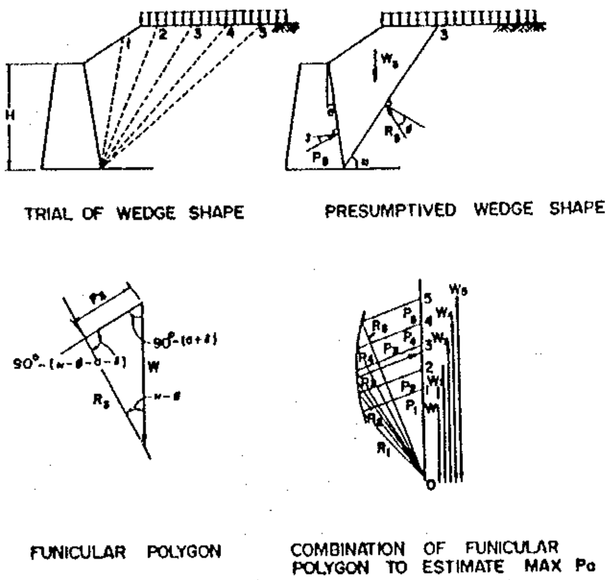


Fig. 1.3.17 Concept of Trial Wedge Shape Method

Where,

- H : height of wall for calculation of earth pressure load (m).
- W : weight of soil within wedge (including surcharge) (t/m).
- R : reaction of sliding face (t/m).
- P : resultant active earth pressure (t/m).
- a : angle between the back of the wall and a vertical line.
- ϕ : internal friction angle.
- δ : friction angle between the back of the wall and soil (refer to Table 1.3.7).
- Θ : angle between imaginary sliding line and horizontal line.

Table 1.3.7 Friction Angle Between Soil and Back of Wall (δ)

Type of retaining wall	Condition	δ (degree)
Gravity type	Soil to concrete	2 / 3 Ø
T-shaped type	Soil to concrete	ß (see Fig. 1. 3. 17)

Horizontal element $Ph = Pa \cdot \cos (a+\delta)$
Vertical element $Pv - Pa \cdot \sin (a+\delta)$

Vertical and horizontal element of earth pressure by trial wedge method is shown in Fig. 1.3.18.

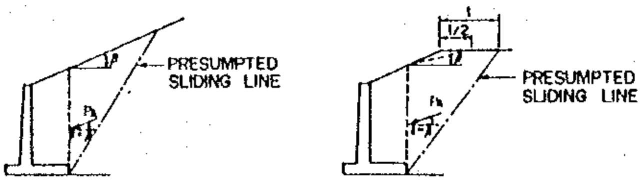


Fig. 1.3.18 Calculation Approach for β

The coefficient of earth pressure is calculated by the trial wedge shape method shown in Fig. 1.3.19.

Earth pressure can be calculated with the following equations after obtaining the coefficient of earth pressure in Fig. 1.3.20

$$Ph = 1/2 \bullet kh \bullet Y \bullet H^2$$
$$Pv = 1/2 \bullet kv \bullet Y \bullet H^2$$

Where,

kh : Coefficient of Horizontal earth pressure
kv : Coefficient of vertical earth pressure
Y : Unit weight of backfill (t/m^3)

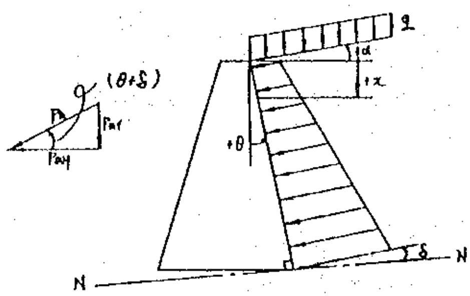
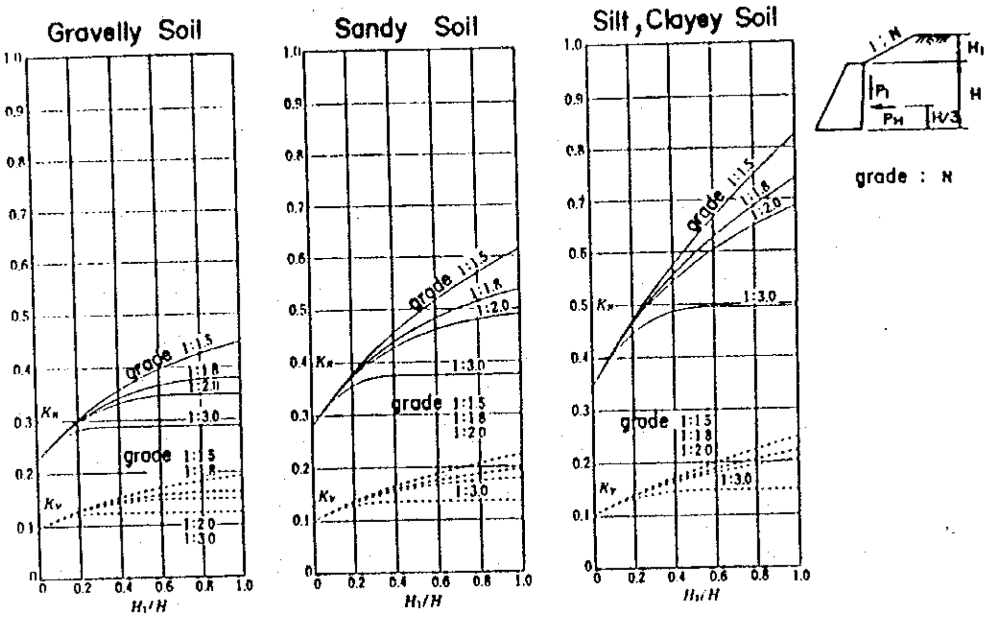


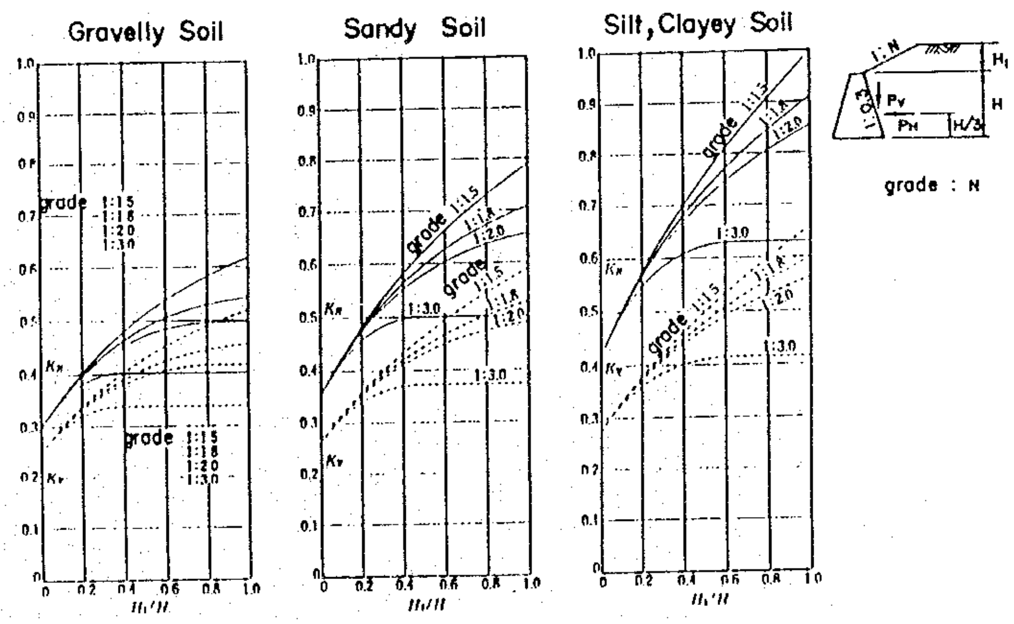
Fig.1.3.20 Vertical and Horizontal Element of Earth Pressure for Colomb

(Soil) $Pa = Ka \bullet Y \bullet x + ka \bullet q$
$Pp = Kp \bullet Y \bullet x + kp \bullet q$

(Silt) $Pa = Ka \bullet Y \bullet x - 2 \bullet c \bullet \sqrt{ka} + ka \bullet q \quad (Pa \geq 0)$
$Pp = Kp \bullet Y \bullet x + 2 \bullet c \bullet \sqrt{kp} + kp \bullet q$

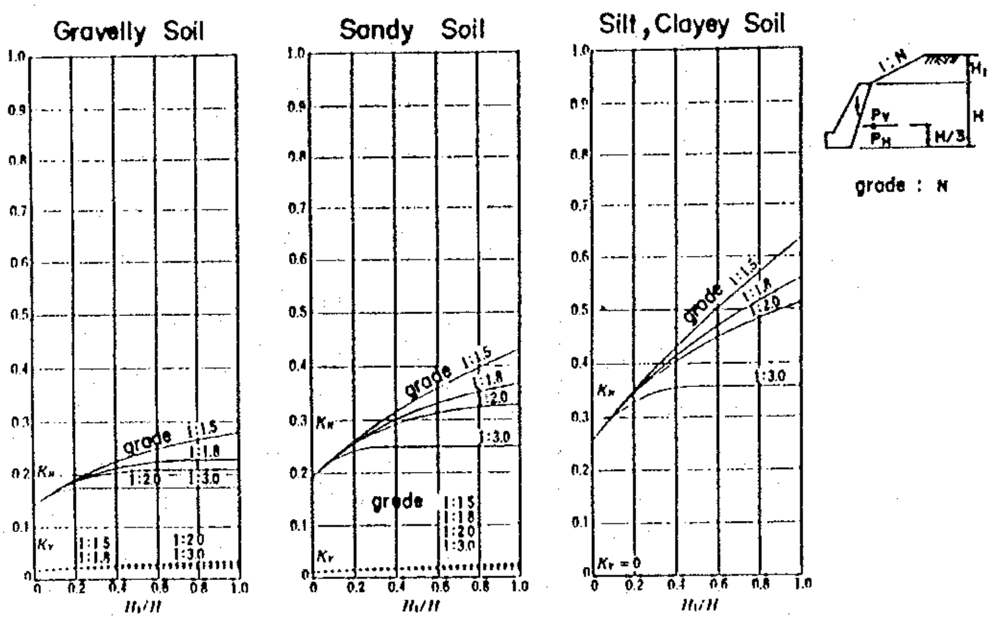


Gravity Type : Back face is vertical

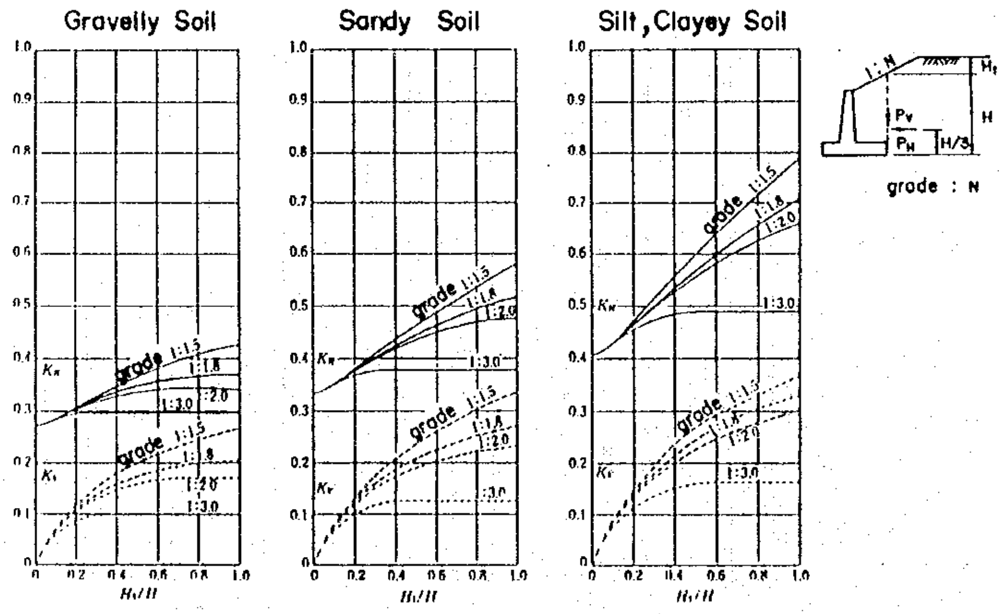


Gravity Type : Back face is 0.3 : 1

Fig. 1.3.19(1) Coefficient of Earth Pressure
Trial Wedge Shape



Supported Type : Back face is 0.3 : 1



T-Shaped Type : Imaginary back face is vertical

Fig. 1.3.19(2) Coefficient of Earth Pressure
Trial Wedge Shape

$$Ka = \frac{\cos^2(\phi-\Theta)}{\cos^2\Theta\cos(\Theta+\delta)\ \left(1 + \dfrac{\sin(\phi+\delta)\sin(\phi-a)}{\cos(\Theta+\phi)\cos(\Theta-a)}\right)^2}$$

$$Kp = \frac{\cos^2(\phi-\Theta)}{\cos^2\Theta\cos(\Theta+\delta)\ \left(1 - \dfrac{\sin(\phi-\delta)\sin(\phi+a)}{\cos(\Theta+\phi)\cos(\Theta-a)}\right)^2}$$

Y : Unit weight of backfill soil (t/m^3)
Pa : active earth pressure (t/m^2)
Pp : passive earth pressure (t/m^2)
Ka : coefficient of active earth pressure
Kp : coefficient of passive earth pressure

- CALCULATION OF STABILITY -

The stability of a retaining wall should be analyzed considering of the four items below.

- Stability against sliding.
- Stability against over turning .
- Stability against capacity of bearing layer.
- Stability of whole system including embankment and foundation.

Stability Against Sliding

The horizontal component of earth pressure results in the retaining wall sliding on its bottom along the bearing layer and producing friction. The passive earth pressure in front of the retaining wall can also be considered as a resisting force, but for safety reasons it is normally neglected in design.

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

$$F_s = \frac{\text{Resisting force against sliding}}{\text{Slide Force}}$$

$$= \frac{(W+P_V)\bullet\tan\delta+C\bullet B}{P_h} \quad \geq 1.5$$

Where,

- Fs : factor of safety.
- Pv : vertical component of earth pressure (t/m).
- Ph : horizontal component of earth pressure (t/m).
- $\tan\delta$: coefficient of friction between bearing ground and bottom of cast-in-place concrete ($\delta=\phi$); for other cases δ = 2/3, but the value of $\tan\delta$ should not exceed 0.6 when bearing ground is soil (normal Table 1.3.4 can be used).
- ϕ : internal friction angle of ground foundation.
- C : cohesion between bearing ground and bottom (t/m), but C should be zero (0) if coefficient of friction ($\tan\delta$) is obtained from Table 1.3.7.
- B : width of bottom of retaining wall (m).

The safety factor for sliding is usually more than 1.5, but can also be determined by the importance of a retaining wall. In the case of the safety factor not being satisfied, the following countermeasures can be taken:

- in creasing wall Bottom length,
- establishment of Protuberance on bottom of wall,
- increasing horizontal resistance with an anchor, and
- inclination of wall Bottom.

Stability Against Overturning

Stability against overturning is determined by the ratio of the resistant moment to the tumble moment.

The distance(d) from the toe to the point where resultant(R) acts can be expressed by the following equation.

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

Eccentricity (e) is the distance from the center of a wall's bottom to the point where resultant (R) acts, and can be expressed by:

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

Further more eccentricity for safety reasons must satisfy the following condition:

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

Where,

- W : dead load.
- Pv : vertical earth pressure component.
- Ph : horizontal earth pressure component.
- a : distance from toe to point where resultant is acting.
- b : horizontal distance from toe to point where Pv is acting (m).
- B : width of bottom of retaining wall (m).
- h : vertical distance from heel of wall to point where Ph is acting (m).

The position of active forces is shown in Fig. 1.3.21.

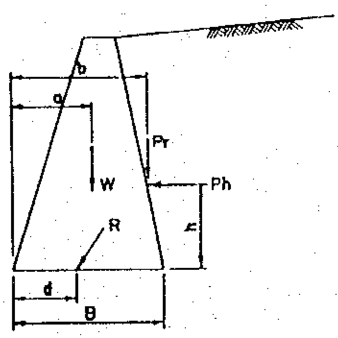


Fig. 1.3.21 Positions of Active forces

- Stability Against Bearing Capacity of Bearing Ground -

Bearing capacity can be calculated by the following equation (see Fig. 1.3.22).

$$q1 \bullet q2 = \frac{W+Pv}{B} \bullet \left(1 \pm \frac{6e}{B}\right)$$

$$q1 \bullet q2 \leq qa = qu/Fs$$

Where,

qa : allowable unit bearing capacity of ground.
qu : ultimate unit bearing capacity of ground.
Fs : safety factor for bearing capacity of ground (= 3.0)

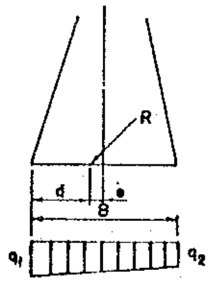


Fig. 1.3.22 Analysis of Bearing Capacity

Stability of the Whole System Including Embankment and Foundation.

For a retaining wall constructed on soft ground, the stability of the whole system should be checked, including the embankment and foundation. Stability analysis using the circular arc method can be applied, as shown in Fig. 1.3.23.

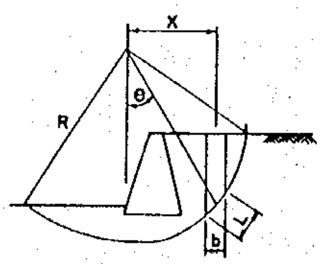


Fig. 1.3.23 Circular Slip on Soft Ground

$$F_S = \Sigma \{C \cdot L + \tan\phi \cdot \cos\Theta (W - \mu \cdot b)\} / \Sigma W \cdot X$$

Where:
F_S: safety factor.
C: cohesion along circular arc.
ϕ: internal friction angle.
W: weight of slice.
μ: pore water pressure.

Retaining Wall on Slope

When a retaining wall is on an incline, the allowable bearing capacity, because of the dip angle and the distance between the fronts of the foundations for the wall and slope, is much less than if the retaining wall was on a flat surface.

As for sliding, it is necessary to consider the stability of the entire slope, which includes the retaining wall and the back of the fill slope.

(3) Design of Retaining Walls

- Gabion Wall

The gabion wall is used to prevent small-scale slope collapses on a slope having seepage water at its toe. Therefore, its structure can only resist small values of earth pressure. Generally, the gabion wall is as shown in Fig.1.3.24.

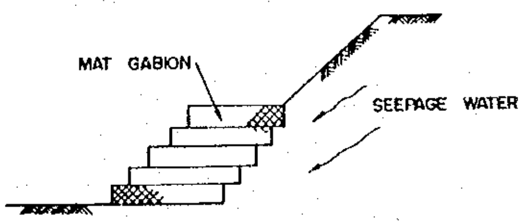
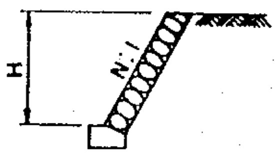


Fig.1.3.24 Gabion Wall

- Stone Riprap Wall

The stone riprap wall is used to prevent small-scale collapses in soil at locations with small earth pressure values. Generally, a stone riprap wall is 5 m or higher. The relationship between the wall's height and front face gradient at fill and cut slopes is as shown in Table 1.3.8.

Table 1.3.8 Relationship between Wall Height & Gradient of the Wall's Front Face



Height of wall (m)		$H \leq 1.5$	$1.5 < H \leq 3.0$	$3.0 < H \leq 5.0$
N: Slope gradient	Fill slope	0.3 : 1	0.4 : 1	0.5 : 1
	Cut slope	0.3 : 1	0.3 : 1	0.4 : 1

Backfill material is used to dispose of water at the back of a stone riprap wall and to reduce the water pressure acting on the wall. Therefore, permeable backfill is desirable. Generally, cobblestone is used and a good size is from 10mm to 50mm. The thickness of the backfill material shall be 30cm.

A weep hole shall be constructed every 2 m^2.

- Gravity-Type Retaining Wall

A gravity-type retaining wall is a wall that resists earth pressure using its dead weight. Furthermore, this wall is designed so that tensile stress does not act on the wall itself by using earth pressure and its dead weight. The height of the wall is to be 3m or lower.

To decide on the configuration of the wall, the following items can be referred to:

1. As shown in Fig.1.3.25, the bottom slab of the wall is generally set at 0.5 to 0.7 of the wall's height.

2. The top of the wall is usually 15 to 40cm. This value is determined by the scale of the wall and on whether or not there is a protective fence.

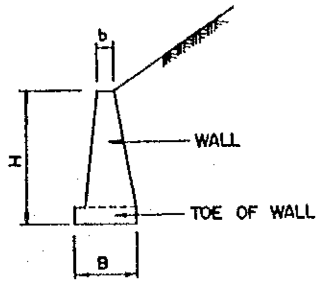


Fig.1.3.25 Configuration of Gravity-Type Retaining Wall

A weep hole is to be constructed every $2m^2$.

Stability calculations for the gravity-type retaining wall are carried out in Reference Materials I.

- T-Shaped Retaining Wall

The T-shaped retaining wall resists earth pressure using the wall's dead weight and the weight of backfill on the wall's heel portion. The height of the wall is usually 3 to 10m.

To decide the configuration of the wall, the following items can be referred to:

1. As shown in Fig.1.3.26, the gradient n of the vertical portion of the wall should be gentler than 0.02:1.

2. The width of the bottom slab of the wall (B) is generally 0.5 to 0.8 of the wall's height.

3. The length of the top of the wall (b_1) is 1/5 the width of the bottom slab (B).

4. The thickness of the ends of the members (C_1,C_2) shall be 30cm or more.

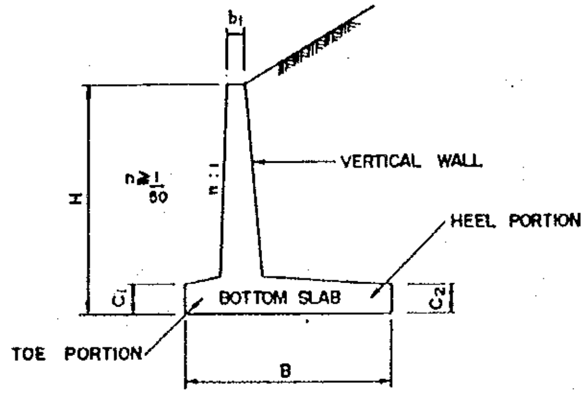


Fig.1.3.26 Configuration of T-Shaped Retaining Wall

5. The point where the footing and the wall meet shall be thick enough to resist shearing force from earth pressure.

6. As shown in Fig.1.3.27, the toe portion will be designed as a cantilever to join the wall. Also, the force acting on the toe portion will be its dead weight and the reaction force of the ground.

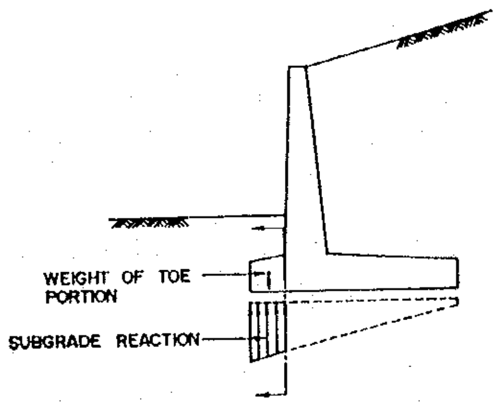


Fig.1.3.27 Force Acting on Toe Portion

7. As shown in Fig.1.3.28, the heel portion is designed as a cantilever to connect with the wall. The force acting on the heel is the reaction force of the ground, the weight of the soil on top of the heel, the vertical component of earth pressure, and surcharge load.

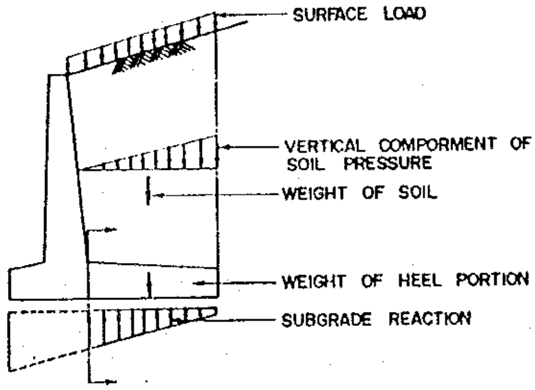


Fig.1.3.28 Force Acting on Heel Portion

A weep hole is constructed every $2m^2$.

- Crib Retaining Wall

A crib retaining wall is a wall that is assembled with precast members. Then, cobblestone must be placed within cribs to make the crib retaining wall heavy. Structurally, this wall is the same as the gravity-type retaining wall. The structural characteristics of the wall are as follows:

1. very good for drainage and applied to places with many springs,

2. the structure is flexible and can adapt to ground displacement, and

3. the wall's members are precast so construction is faster than for other wall types.

As for design, the approach is the same as that for the gravity-type retaining wall. For stability calculations, the water pressure on the back face of members is not considered.

Regarding the assembly of a crib retaining wall, there are two methods: single crib structure and double crib structure (see Fig.1.3.29). The assembly method is determined by stability calculations.

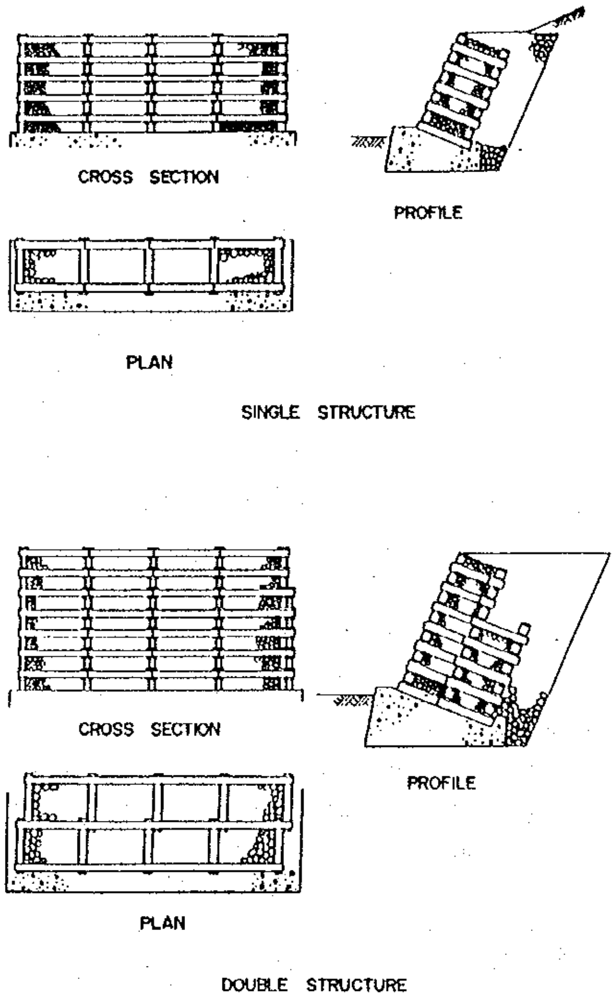


Fig.1.3.29 Assembly Methods for Crib Retaining Wall

The calculation of gravity type wall is presented in Reference Material II.

1.3.5 Horizontal Drain Hole

- Purpose

The purpose of a horizontal drain hole is to prevent landslides from occurring, by reducing the groundwater level for unstable landslide debris and for unstable slopes predicted to have a landslide.

- Method to Confirm Slip Surface

To properly install a horizontal drain hole for groundwater drainage, it is necessary to confirm the slip surface with a soil survey.

The perpendicular boring for the soil survey shall be carried out for at least three spots on the potential or actual landslide surface. Sliding occurs in many cases along the boundaries between colluvium and bedrock and between severely weathered rock and lightly weathered rock. Therefore, boring is carried out to confirm these boundaries and the changes in soil.

- Design of Horizontal Drain Hole

It is planned to construct 4 to 5 drain holes per location. Drain holes will be 66mm in diameter and face upward at an angle of 5 to 10° to easily carry out drainage of groundwater (see Fig. 1.3.30).

As for the method of construction, as shown in Fig.1.3.31, a boring machine will be used to drill holes. In the case of drilling holes where the soil is made up of gravel or soil, casing pipe will be used since they can easily collapse.

Regarding the distance between drain holes on a slip surface, it will be every 5m as shown in Fig.1.3.32.

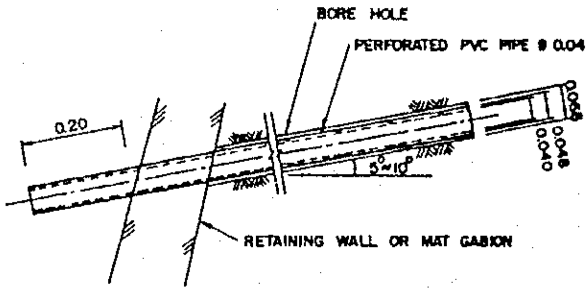


Fig.1.3.30 Angle of Drain Hole

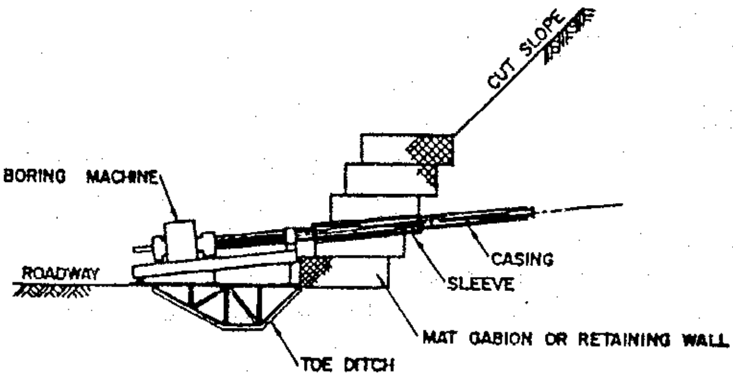


Fig.1.3.31 Boring Machine

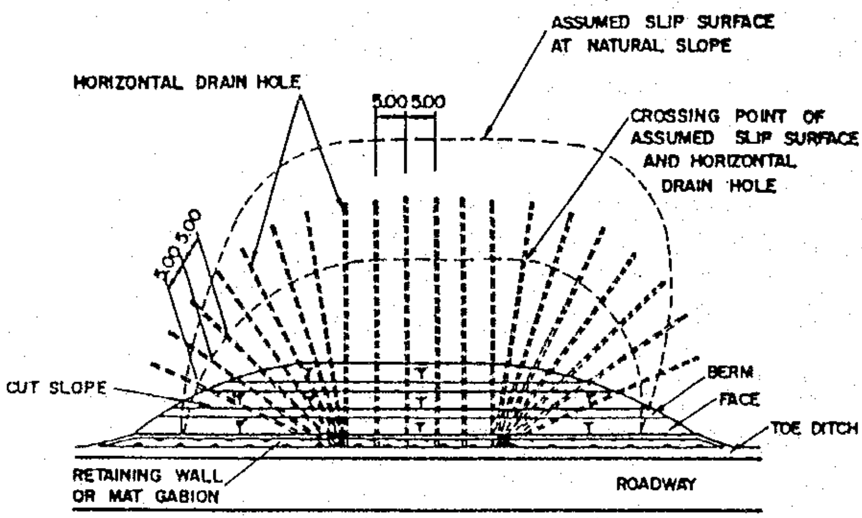
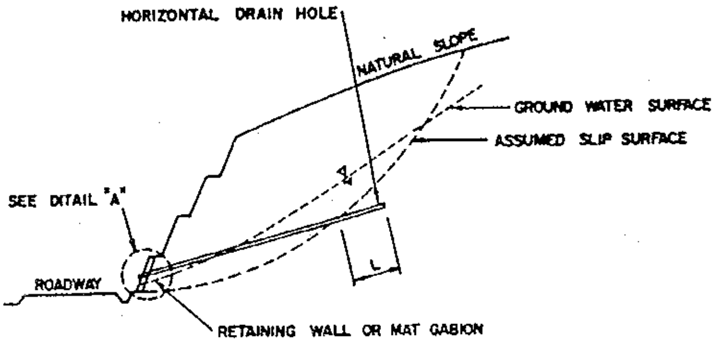


Fig.1.3.32 Drain Holes Arrangement

As shown in Fig.1.3.33, boring depth from slip surface will 10m in the case of a soil slope and 3m in the case of a rock slope.



ORIGINAL GROUND CONDITION	L
SOIL	10.00
ROCK	3.00

Fig.1.3.33 Boring Depth from Slip Surface

- Materials

In the bored holes, strainer PVC pipe will be inserted(see Fig. 1.3.34).

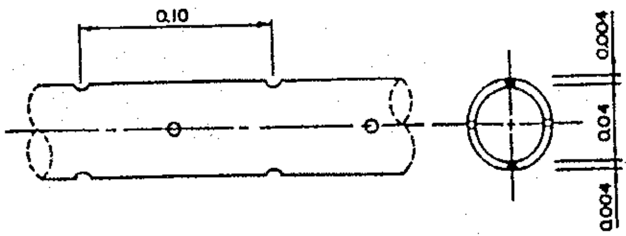


Fig.1.3.34 PVC Pipe

- Reduction of Groundwater Level

Generally, horizontal drain holes reduce the groundwater level by approximately 3m. Therefore, when carrying out stability calculations for groundwater planning, this shall be 3m less than the maximum groundwater level.

Reference Material I

Explanation for Users on Application of Rotational Slope Stability Analysis

1. Introduction

Rotational slope stability analysis was carried out using a software package published by Sarma (1979) and modified by E. Hoek (1985). The package utilizes a general method limited equilibrium analysis that can determine the stability of 2 dimensional slopes of a variety of shapes. For example, slopes with complex profiles with circular, non-circular, or planar sliding surfaces, or any combination of these, can be analysed using this method. In addition, active-passive wedge failures, such as those which occur in spoil piles on sloping foundations or in clay core dam embankments, can also be analysed.

In the analysis, different shear strengths can be specified for each slice and base. The freedom to change the inclination of a slice also allows the incorporation of such structural features as faults and bedding planes. External forces action on each slice can also be included, and the submergence of any part of a slope is automatically incorporated into the analysis.

2. Premises of Analysis

The geometrical coordinates of a sliding mass are defined by (XT_i, YT_i), (XB_i, YB_i), (XT_{i+1}, YT_{i+1}), and (XB_{i+1}, YB_{i+1}), and represent the corners the mass as shown in Figure 2.1. The phreatic surface is defined by the coordinates (XW_i, YW_i) and (XW_{i+1}, YW_{i+1}), which intersect with the sides of the slice.

a). A closed form solution is used to calculate the critical horizontal acceleration Kc required to induce a state of limiting equilibrium in sliding mass.

b). The static factor of safety F is then found by reducing the shear strength values tan ?ϕ and c to tan ϕ/F and c/F until the critical acceleration Kc is reduced to zero.

In order to determine whether the analysis is acceptable, a check is carried out to assess whether all the effective normal stresses acting across the bases and slices of the slices are positive. If negative stresses are found, the slice geometry or the groundwater conditions must be changed until the stresses are all positive. An additional check for moment equilibrium is described, but it has not been incorporated into the program lists at the end of the chapter because it involves a significant increase in computational effort and because it is seldom required for normal slope stability problems.

3. Application of Analysis

3.1 General

The software package is interactive and is therefore easy to use. Below,

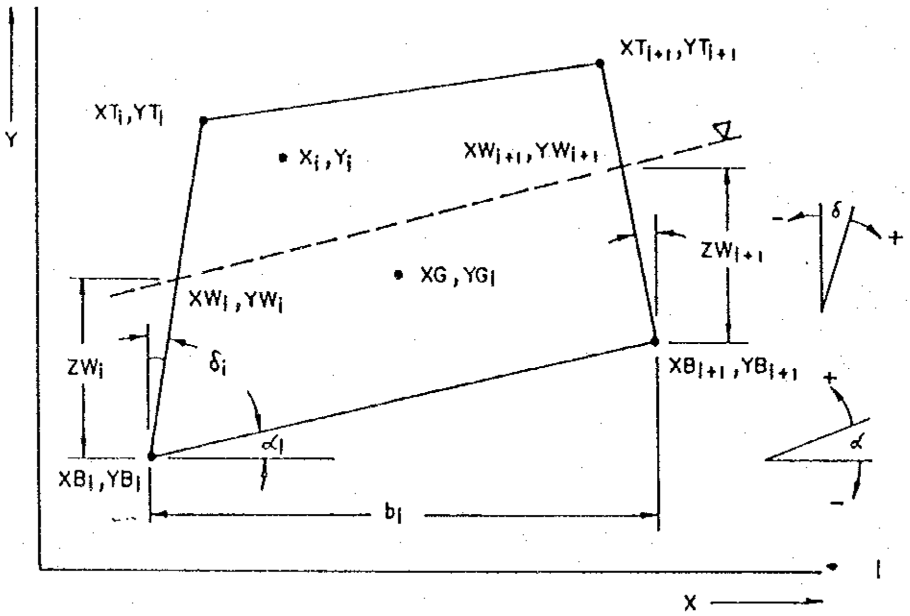
3.2 Using the Software Package

1) Switch on the computer.
2) When you see 'c:\>' on the monitor, insert the disk containing the software package.
3) After inserting disk input 'A:'.
4) After'A:\>' appears on the monitor, input 'CD SARMA'.
5) After'A:\SARMA>' appears on the monitor, input 'SARMA79'.
6) You will see the following message on the monitor.

```
              SARMA NON-VERTICAL SLICE STABILITY ANALYSIS

Copyright - Evert Hoek, 1985.  This program is one of a
series  of geotechnical  programs  developed as working
tools and for educational purposes.  Use of the program
is not restricted but the user is responsible for the
application of the results obtained from this program.

         Note:  In order to operate this program a data
         disk with at least one file with an extension.
         SAR is required. When starting a new disk, ensure
         that such a file is stored on the disk before it
         is used.
Specify drive to be used for data disk(default B:)-
```



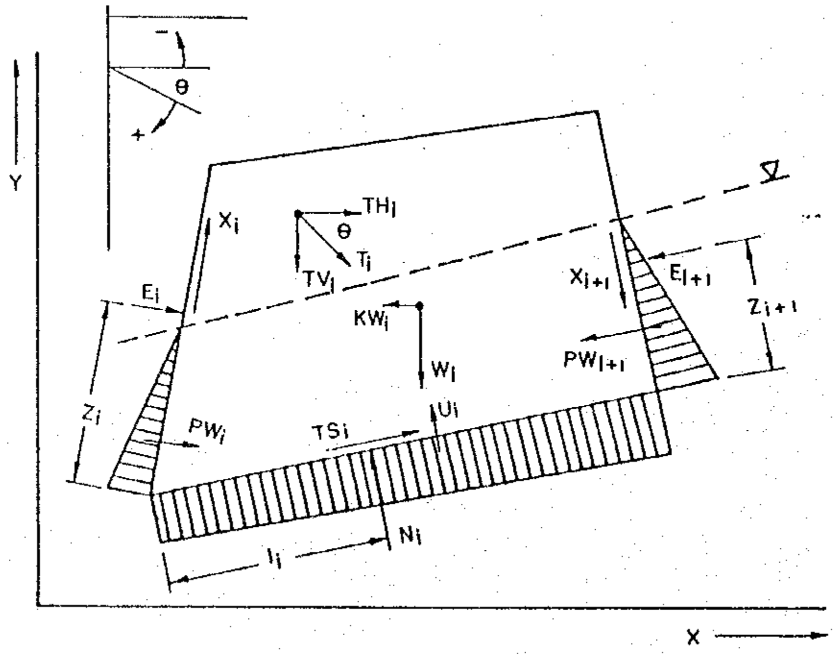


Figure 2.1 Definition of geometry and forces acting on the ith slice.

7) Input 'A'
8) You will see the following message on the monitor.

```
Do you with to read data from a disk file (y/n)?:-
```

If you want to input new data, you should input 'n'. If you need to correct or change some data, you should input 'y'.

A sample analysis is shown in Figure 3.1.

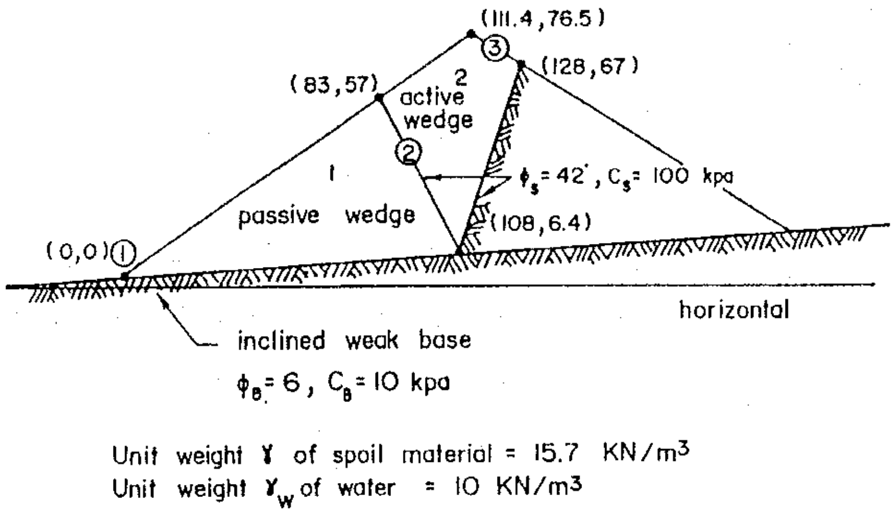


Figure 3.1 Geometry of a spoil pile on a weak foundation analysed by Coulthard (1979)

```
Do you wish to read data from a disk file (Y/N) ? : N
Number of slices to be included in analysis :       2
Unit weight of water =                               10
Are shear strengths uniform throughout slope (Y/N) ?


to terminate input enter [q] in response to any question
```

```
Analysis no.-

Side number                1          2          3
coordinate xt
coordinate yt
coordinate xw
coordinate yw
coordinate xb
coordinate yb
friction angle
cohesion
unit weight of water = 10
Slice number       1          2
rock unit weight
friction angle
cohesion
force T
angle theta

 Note: coordinates must increase from slope toe to crest



 To edit title or data array, use direction keys to move
 highlighted window.  Factor of safety calculation will
 commence automatically when all data has been entered.
```

```
            SARMA NON-VERTICAL SLICE ANALYSIS

 Analysis no.  1;spoil pile on a weak foundation


Unit weight of water = 10

Side number                1          2          3
Coordinate xt       0.00       83.00      111.40
Coordinate yt       0.00       57.00       76.50
Coordinate xw       0.00       83.00      128.00
Coordinate yw       0.00       57.00       67.00
Coordinate xb       0.00      108.00      128.00
Coordinate yb       0.00        6.40       67.00
Friction angle      0.00       42.00        0.00
Cohesion     0.00     100.00        0.00

Slice number                  1          2
Rock unit weight          15.70      15.70
Friction angle             6.00      42.00
Cohesion              10.00     100.00
Force T                    0.00       0.00
Angle theta                0.00       0.00

Effective normal stresses
Base                     147.15     -14.13
Side                0.00       -130.64
 Acceleration Kc =  -0.2087  Factor of Safety = 0.26
 Negative effective normal stresses-solution unacceptable
1 print 2 calculate 3 fos vsk 4 drain 5 file 6 restart
7 quit  8 view
```

4. Example : Slope of Open-Pit Coal Mine

Figure 4.1 illustrates the geometry of a slope for a large open-pit coal mine. A thin coal seam is overlain by soft tuff, and past failures of the slope have show that sliding occurs along the coal seam with the toe breaking out through the soft tuff. In this case a reservoir close by results in a high groundwater surface. Laboratory tests and past analysis of previous failures give a friction angle of 18° and a cohesion of zero along the coat seam and a friction angle of 30° and cohesion of $2t/m^2$ for failure through the soft tuff. The unit weight of the tuff is 2.1 t/m^3 and the unit weight of the water is 1.0 t/m^3.

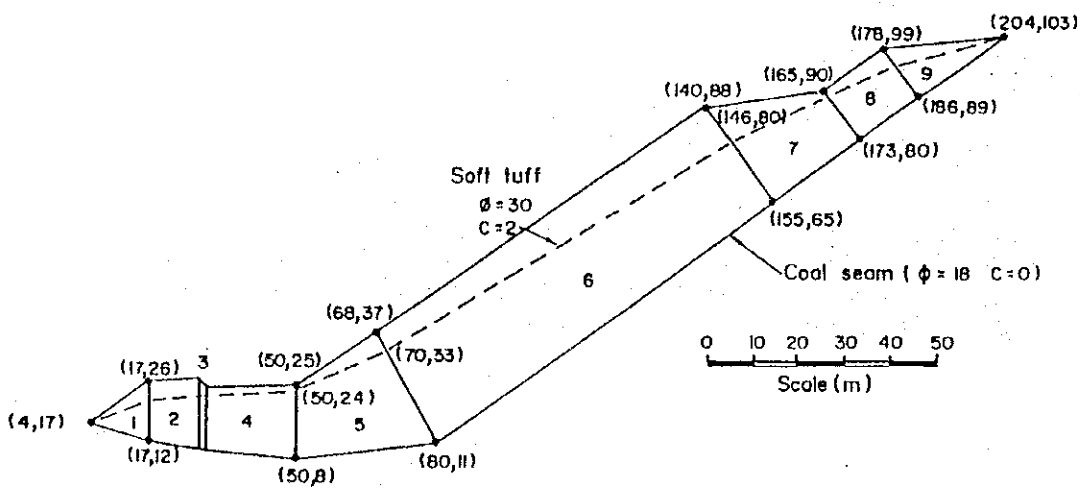


Figure 4.1 Geometry for an open-pit coal mine slope.

SARMA NON-VERTICAL SLICE ANALYSIS

Analysis no. 2:open pit coal mine slope with tuff overlying coal seam

Unit weight of water = 1

Side number	1	2	3	4	5	6
Coordinate xt	4.00	17.00	29.00	30.00	50.00	68.00
Coordinate yt	17.00	26.00	26.00	24.00	25.00	37.00
Coordinate xw	4.00	17.00	29.00	30.00	50.00	70.00
Coordinate yw	17.00	23.00	22.00	22.00	24.00	33.00
Coordinate xb	4.00	17.00	29.00	30.00	50.00	80.00
Coordinate yb	17.00	12.00	10.00	10.00	8.00	11.00
Friction angle	0.00	30.00	30.00	30.00	30.00	30.00
Cohesion	0.00	2.00	2.00	2.00	2.00	2.00

Slice number	1	2	3	4	5	6
Rock unit weight	2.10	2.10	2.10	2.10	2.10	2.10
Friction angle	30.00	30.00	30.00	30.00	30.00	18.00
Cohesion	2.00	2.00	2.00	2.00	2.00	0.00
Force T	0.00	0.00	0.00	0.00	0.00	0.00
Angle theta	0.00	0.00	0.00	0.00	0.00	0.00

Effective normal stresses

Base	28.62	36.69	22.78	29.20	79.89	24.56
Side	0.00	22.67	40.47	47.24	60.56	92.99

Side number	7	8	9	10
Coordinate xt	140.00	165.00	178.00	204.00
Coordinate yt	88.00	90.00	99.00	103.00
Coordinate xw	146.00	166.00	180.00	204.00
Coordinate yw	80.00	89.00	96.00	103.00
Coordinate xb	155.00	173.00	186.00	204.00
Coordinate yb	65.00	80.00	89.00	103.00
Friction angle	30.00	30.00	30.00	0.00
Cohesion	2.00	2.00	2.00	0.00

Slice number	7	8	9
Rock unit weight	2.10	2.10	2.10
Friction angle	18.00	18.00	18.00
Cohesion	0.00	0.00	0.00
Force T	0.00	0.00	0.00
Angle theta	0.00	0.00	0.00

Effective normal stresses

Base	15.38	8.35	3.75
Side	24.46	23.78	9.83

Acceleration Kc = 0.1608 Factor of Safety = 1.20

Large extrapolation - plot of fos vs K suggested to check fos

Case 1. Counterweight

SARMA NON-VERTICAL SLICE ANALYSIS

Analysis no. COUNTERWEIGHT

Unit weight of water = 9.8

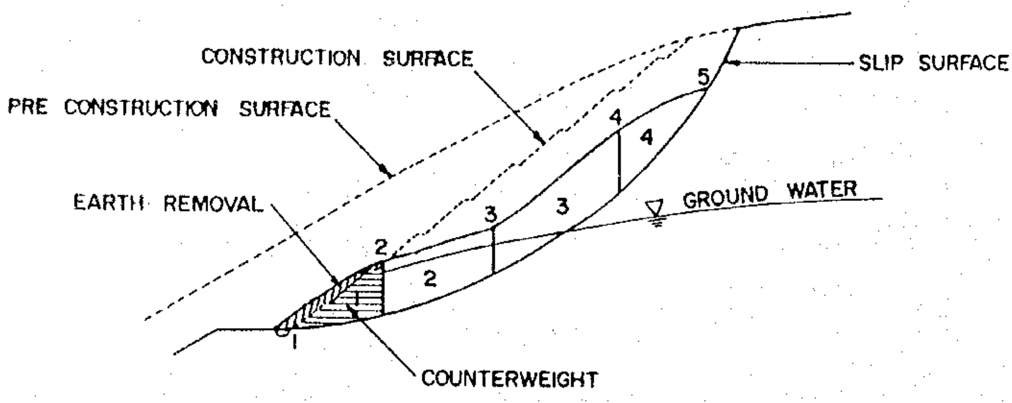
Side number	1	2	3	4	5
Coordinate xt	0.00	19.00	29.50	48.00	61.50
Coordinate yt	0.00	11.50	14.50	29.00	35.00
Coordinate xw	0.00	19.00	29.50	48.00	61.50
Coordinate yw	0.00	10.50	13.50	20.50	35.00
Coordinate xb	0.00	13.50	29.50	48.00	61.50
Coordinate yb	0.00	2.00	8.20	20.50	35.00
Friction angle	0.00	40.00	37.00	37.00	0.00
Cohesion	0.00	5.00	5.00	5.00	0.00

Slice number	1	2	3	4
Rock unit weight	22.50	19.60	19.60	19.60
Friction angle	40.00	37.00	37.00	37.00
Cohesion	13.50	0.00	0.00	5.00
Force T	0.00	0.00	0.00	0.00
Angle theta	0.00	0.00	0.00	0.00

Effective normal stresses

Base		38.50		49.99		78.24		38.45
Side	0.00		4.81		27.87		12.44	

Acceleration Kc = 0.0060 Factor of Safety = 1.02



Case 2. Removal of Slide Debris

SARMA NON-VERTICAL SLICE ANALYSIS

Analysis no. REMOVAL OF SLIDE DEBRIS

Unit weight of water = 9.8

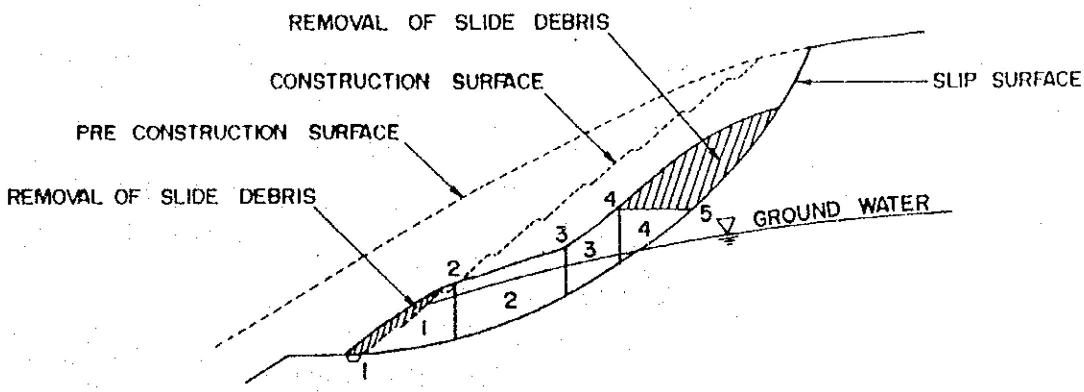
Side number	1	2	3	4	5
Coordinate xt	0.00	13.50	29.50	37.00	48.00
Coordinate yt	0.00	10.00	14.50	20.00	20.50
Coordinate xw	0.00	13.50	29.50	37.00	48.00
Coordinate yw	0.00	8.00	13.00	15.00	20.50
Coordinate xb	0.00	13.50	29.50	37.00	48.00
Coordinate yb	0.00	2.00	8.20	12.00	20.50
Friction angle	0.00	37.00	37.00	37.00	0.00
Cohesion	0.00	5.00	5.00	5.00	0.00

Slice number	1	2	3	4
Rock unit weight	19.60	19.60	19.60	19.60
Friction angle	37.00	37.00	37.00	37.00
Cohesion	0.00	0.00	0.00	0.00
Force T	0.00	0.00	0.00	0.00
Angle theta	0.00	0.00	0.00	0.00

Effective normal stresses

Base		51.81		70.14		77.78		35.37	
Side	0.00		13.72		20.62		16.50		

Acceleration Kc = 0.0589 Factor of Safety = 1.17



Case 3. Horizontal Drain Hole

SARMA NON-VERTICAL SLICE ANALYSIS

Analysis no. HORIZONTAL DRAIN HOLE

Unit weight of water = 9.8

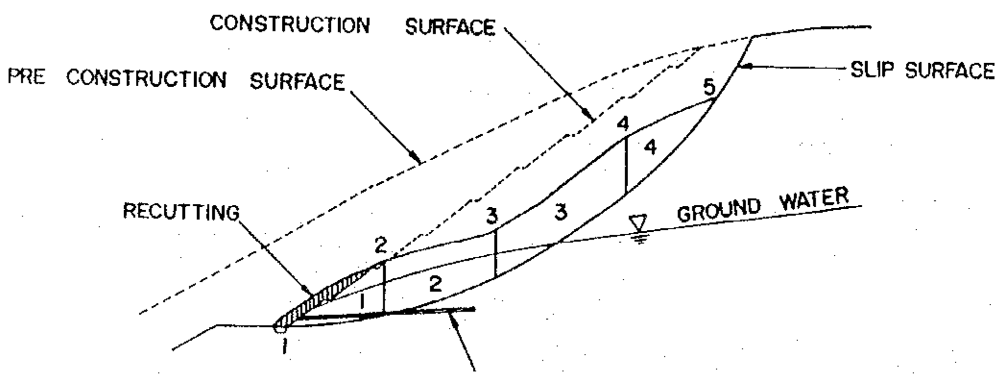
Side number	1	2	3	4	5
Coordinate xt	0.00	13.50	29.50	48.00	61.50
Coordinate yt	0.00	10.00	14.50	29.00	35.00
Coordinate xw	0.00	13.50	29.50	48.00	61.50
Coordinate yw	0.00	7.00	11.50	20.50	35.00
Coordinate xb	0.00	13.50	29.50	48.00	61.50
Coordinate yb	0.00	2.00	8.20	20.50	35.00
Friction angle	0.00	37.00	37.00	37.00	0.00
Cohesion	0.00	5.00	5.00	5.00	0.00

Slice number	1	2	3	4
Rock unit weight	19.60	19.60	19.60	19.60
Friction angle	37.00	37.00	37.00	37.00
Cohesion	0.00	0.00	0.00	5.00
Force T	0.00	0.00	0.00	0.00
Angle theta	0.00	0.00	0.00	0.00

Effective normal stresses

Base		61.48	84.19	86.27	40.18
Side	0.00	28.90	60.14	21.48	

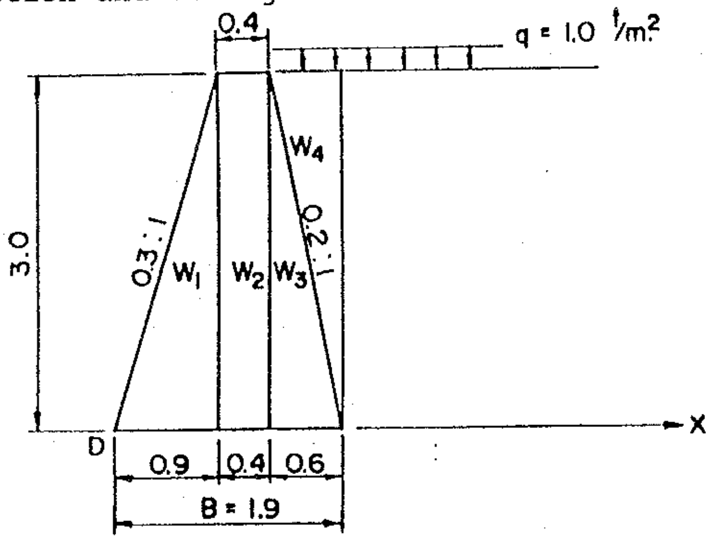
Acceleration Kc = 0.0773 Factor of Safety = 1.19



Reference Meterial II

Example Calculation of Gravity Type Retaining Wall

Cross Section and Configuration



- Unit Weight of Concrete $\gamma c = 2.40\ t/m^2$
- Unit weight of Backfill $\gamma s = 1.9\ t/m^3$
- Internal Friction Angle of Backfill $\phi = 35$
- Cohesion of Backfill $C = 0$
- Coefficient of Friction $\tan\delta = 0.6$
- Height of Earth Pressure $H = 3.0/3 = 1.0$ m

Load

- Earth Pressure
  Coulom Method

$$Pa = 1/2 \bullet Ka \bullet \gamma \bullet H^2 + ka \bullet q \bullet H$$
$$= 1/2 \bullet 0.335 \bullet 1.9 \bullet 3.0^2 + 0.335 \bullet 1.0 \bullet 3.0$$
$$= 3.869\ t$$

$$Pah = Pa \bullet \cos(\Theta+\delta) = 3.185\ t$$

$$Pav = Pa \bullet \sin(\Theta+\delta) = 2.197\ t$$

$$Ka = \frac{\cos^2(\phi-\Theta)}{\cos^2\Theta\cos(\Theta+\delta)\left(1 + \frac{\sin(\phi+\delta)\sin(\phi-\alpha)}{\cos(\Theta+\delta)\cos(\Theta-\alpha)}\right)^2}$$

$$= \frac{\cos^2(35-11.3)}{\cos^2 11.3\ \cos(11.3+23.3)\left(1+\dfrac{\sin(35+23.3)\sin(35-0)}{\cos(11.3+23.3)\cos(11.3-0)}\right)^2}$$

$= 0.335$

Trial Wedge Shape Method

$Ph = 1/2 \bullet Kh \bullet y \bullet H^2$

$Pv = 1/2 \bullet Kv \bullet y \bullet H^2$

$H1/H = 0$
Soil Grade : Gravel with sand

$Kh = 0.30$
$Kv = 0.25$

$Ph = 1/2 \bullet 0.30 \bullet 1.9 \bullet 3.0^2 = 2.565$ t

$Pv = 1/2 \bullet 0.25 \bullet 1.9 \bullet 3.0^2 = 2.138$ t

•Dead Load and Backfill Load

		Unit Weight	Vertical Component	Arm	Moment
		y (t/m³)	V(t)	X(m)	M(tm)
W1	3.0 x 0.9 x 1/2	2.40	3.17	0.600	1.90
W2	3.0 x 0.4	2.40	2.82	1.100	3.10
W3	3.0 x 0.6 x 1/2	2.40	2.12	1.500	3.18
W4	3.0 x 0.6 x 1/2	1.90	1.71	1.700	2.91
			9.82		11.09

•Total of Load (Coulom Method)

	Vertical Component V(t)	Arm X(m)	Resistance Moment Mr(tm)	Horizontal Component H(t)	Arm Y(m)	Moment MO(tm)
	9.82		11.09			
	2.20	1.900	4.22			
				3.19	1.000	3.19
	12.02		15.31	3.19		3.19

•Total of Load (Trial Wedge Shape Method)

	Vertical Component V(t)	Arm X(m)	Resistance Moment Mr(tm)	Horizontal Component H(t)	Arm Y(m)	Moment MO(tm)
	9.82		11.09			
	2.14	1.900	4.07			
				2.57	1.000	2.57
	11.96		15.16	2.57		2.57

Calculation of Stability

•Overturning

$$d = \frac{Mr - MO}{V} = \frac{\overset{(15.16)(2.57)}{15.31 - 3.19}}{\underset{(11.96)}{12.02}} = \underset{(1.052)}{1.007}\ \mathrm{m}$$

$$e = \frac{B}{2} - d = \frac{1.90}{2} - \underset{(1.052)}{1.007}$$

$$= \underset{(-0.102)}{-0.057} < \frac{B}{6} = 0.31\ \mathrm{m}\ \text{...OK!}$$

•Sliding

$$F = \frac{V \bullet \tan\phi + C \bullet B}{H}$$

$$= \frac{\overset{(11.96)}{12.02} \times 0.6 + 0 \times 1.90}{\underset{(2.57)}{3.19}} = \underset{(2.792)}{2.265} > 1.5\ \text{...OK!}$$

•Bearing Capacity

$$q1 \bullet q2 = \frac{V}{B} \bullet \left(1 \pm \frac{6e}{B}\right) \qquad q1, q2 \le qa = qu/Fs$$

$$= \frac{\overset{(11.96)}{12.02}}{1.9} \bullet \left(1 \pm \frac{6 \times \overset{(0.102)}{0.057}}{1.9}\right)$$

$$= \underset{(6.498)}{7.477},\ \underset{(4.267)}{5.200} < 20\ (t)\ \text{...OK!}$$

():Trial Wedge Shape Method

2. Collapsing of Bridge

2.1 Discharge Capacity

2.1.1 Calculation of Discharge Capacity

To decide a waterway's cross section at a bridge crossing, the cross section of culverts and other structures that intersect with road, and the capacity of drainage facilities, it is necessary to calculate discharge capacity. In calculating discharge capacity, the amount of rainfall is used to derive river discharge, etc. Various methods have been advocated to obtain discharge capacity, but the process shown in Fig.2.1.1 is applied.

- Determination of Catchment Basin

The area of a catchment basin for drainage facilities, etc. under study is measured and determined with a topographical map. The area shall be fixed by considering the influence of watersheds, roads, structures such as railways, changes in the drainage basin due to other drainage facilities, etc. If the catchment basin (or drainage area) has an area of 25km^2 or more, then a 50-year rainfall return period is applied. Then, using the specific yields of flood flows in Fig.2.1.2, discharge capacity can be directly obtained.

- Rainfall Return Period

Although it is desirable that no matter how hard it rains the rainfall be completely drained, this is not economically advisable. Therefore, the necessary drainage capacity to protect and maintain roads is decided by the importance of the subject structure, ease of maintenance, etc. Here, the rainfall return period has been set as follows:

Road surface & slope drainage, etc. (excludes inflows from land adjacent to roads, natural slopes): 3-year return period.

A waterway or river with a catchment basin 25km^2 or less in area: 20-year return period.

- Run-off Coefficient

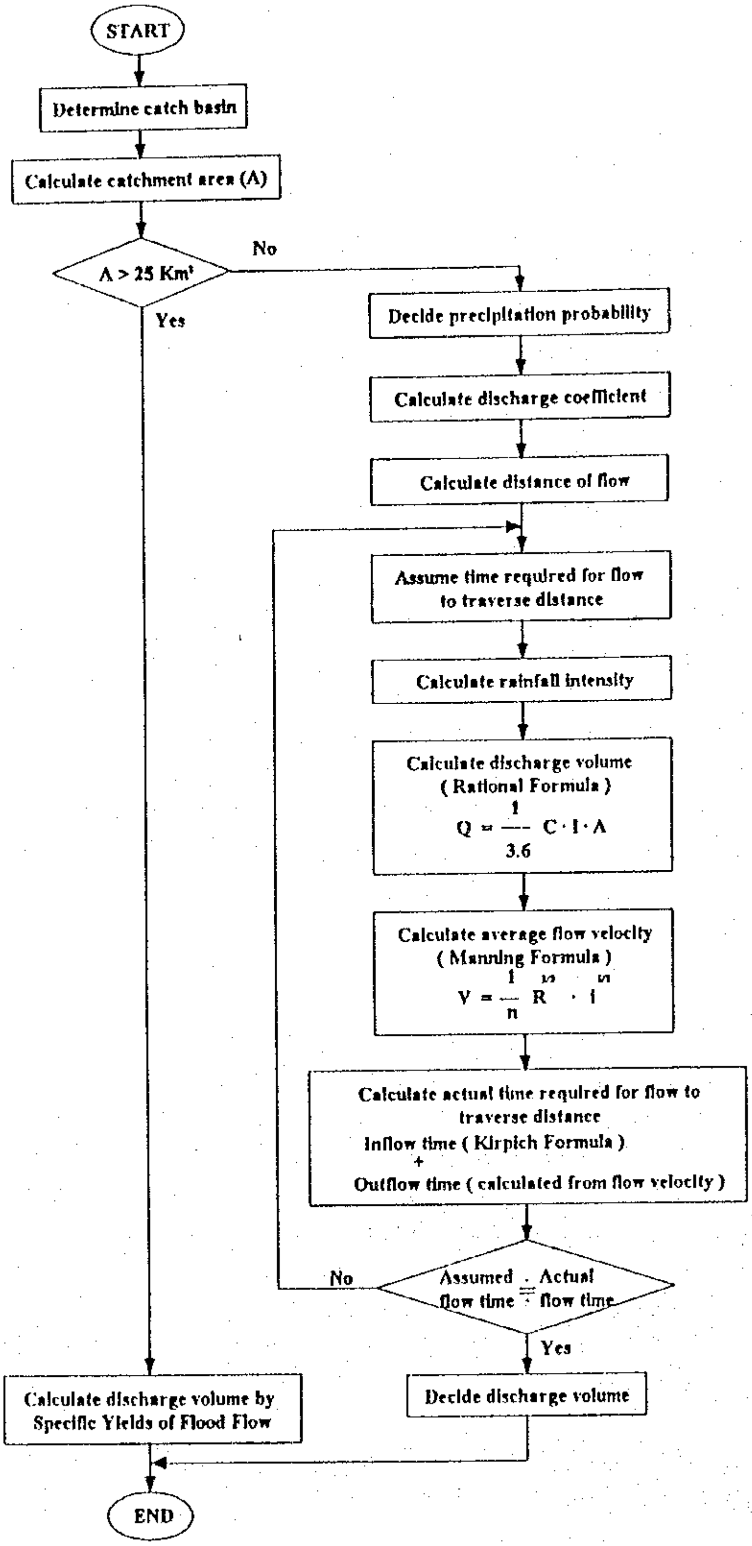


Fig.2.1.2 Specific Yields of Flood Flows

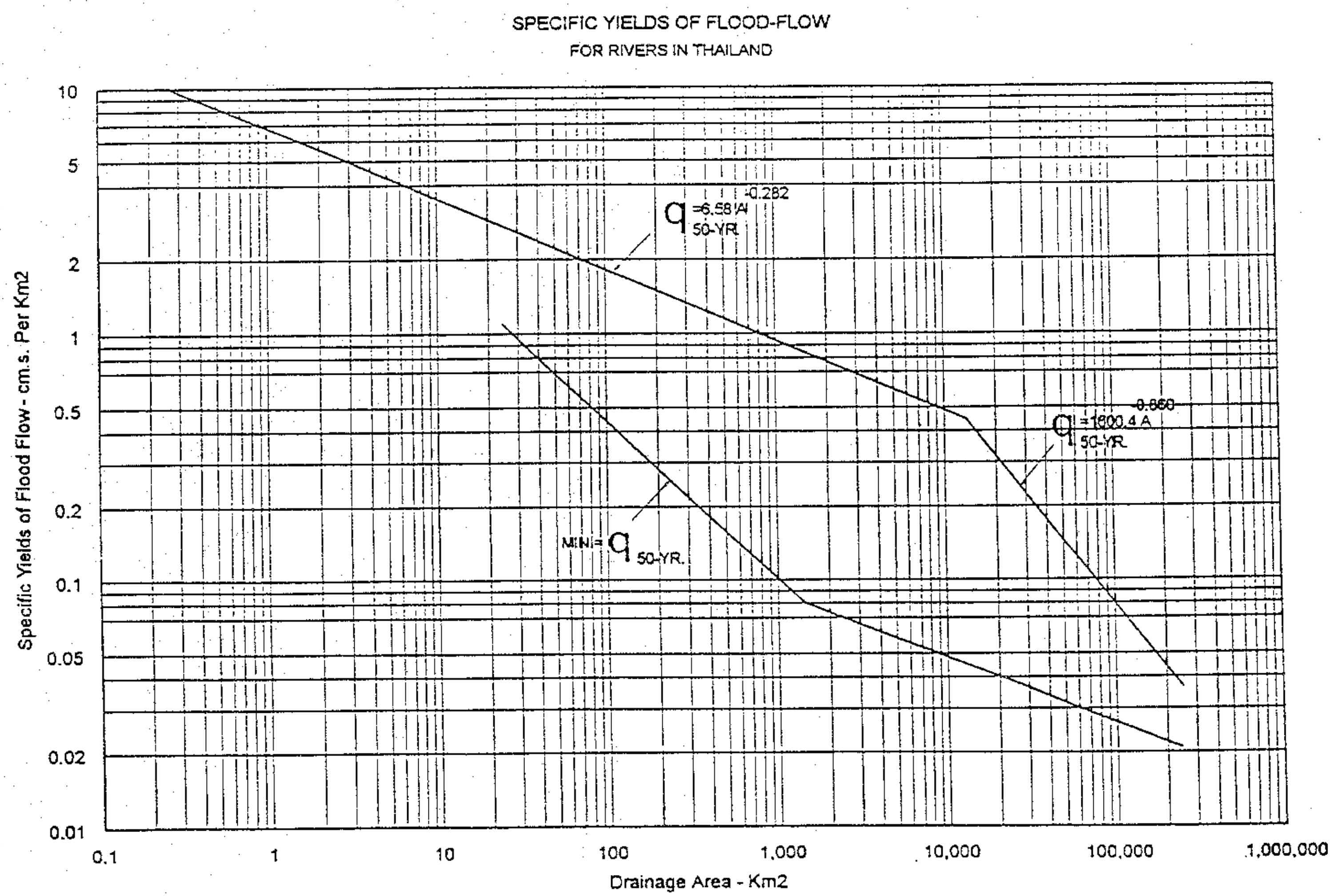


Fig.2.1.2 Specific Yields of Flood Flows

The run-off coefficient varies with rainfall and the characteristics of a drainage area. Although it is difficult to set its value without any discussion, it has been decided to fix its value based on the different types of land use shown below.

Mountainous catchment: 0.6

Flat catchment: 0.5

Paddy field: 0.7

In the case of when a catchment basin's use is not uniform, a weighted average using land-use ratios can be applied. Therefore, it is necessary to divide up the area of a catchment basin by land use.

- Setting Rainfall Intensity

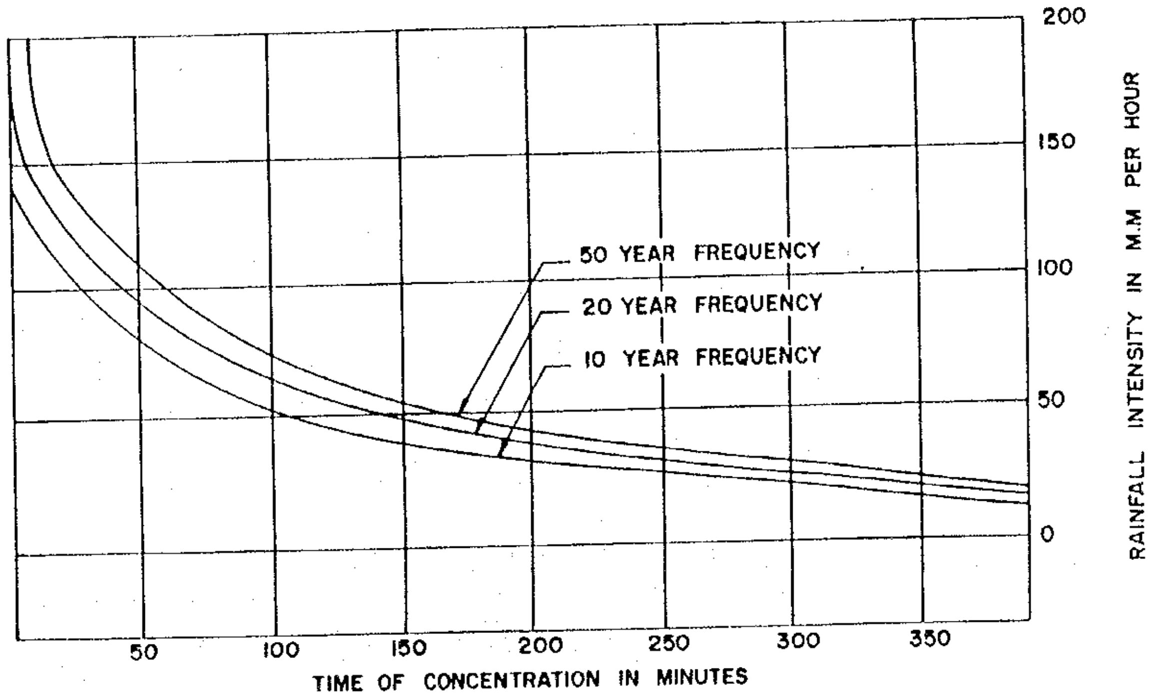
When calculating discharge capacity with the rational formula, it is necessary to derive the rainfall intensity I(mm/h) that corresponds with the time it takes for rainfall to flow down from the furthest point on a catchment basin (time of concentration). Applying DOH graphs based on past observations that relate rainfall amount to arbitrary time continuums, rainfall intensity was derived. These graphs were for 17 stations located throughout the country. In Fig.2.1.3, there are examples for Chiang Mai and Songkla. However, since there are no graphs for a 3-year return period, a 10-year return period is used.

- Calculation of Discharge Capacity

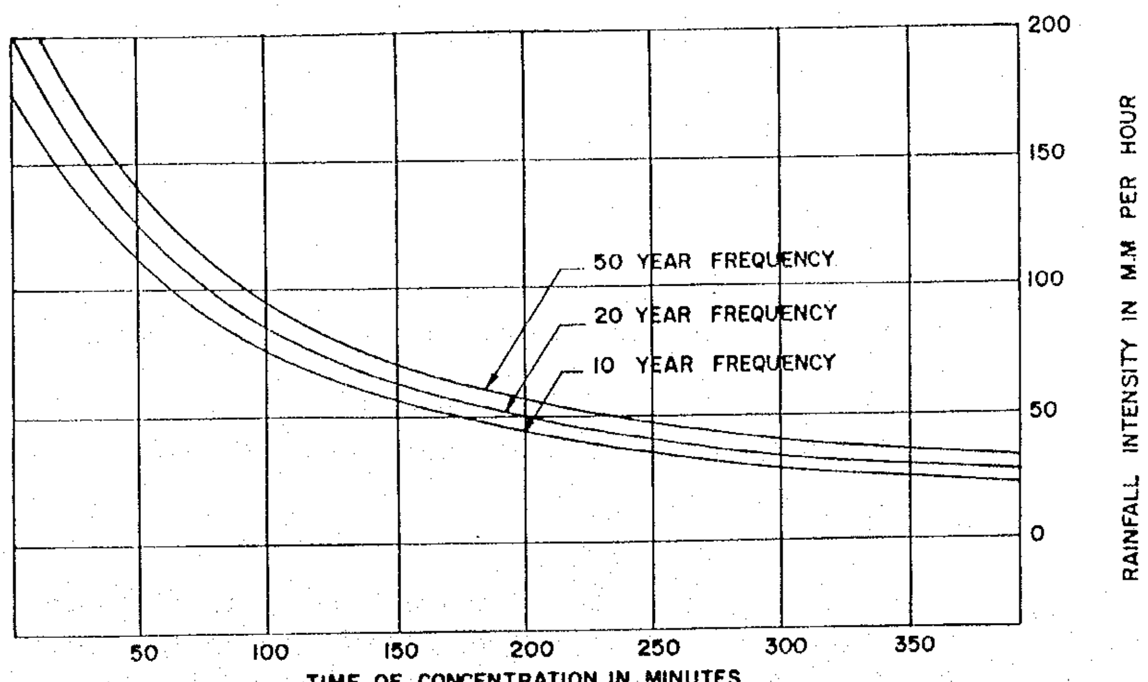
In the case of a 50-year return period, as stated before, discharge capacity is directly obtained from the specific yields of flood flows in Fig.2.1.2. In other cases, the rational formula shown below is applied to calculate discharge capacity.

$$Q = \frac{1}{3.6} C \cdot I \cdot A$$

Where,
Q: discharge volume (m^3/sec).
C: run-off coefficient.



CHIANGMAI STATION



SONGKLA STATION

Fig. 2.1.3 Intensity Duration Curves

I: rainfall intensity within time of concentration (mm/h).

A: catchment area (m^2).

As for deriving the value of the time of concentration, it is obtained by comparing iteratively an assumed value with an estimated value. If the assumed value is smaller than the estimated value, and the difference is within 20% of the assumed value, the calculation process is finished.

- Time of Concentration

Time of concentration is divided into two parts: the time it takes rain to flow down to and through drainage facilities from the furthest point on a catchment basin (inlet time t_1) and the time it takes a river to carry the rain to a predetermined point (outlet time t_2). Therefore, in the case of road surface water or a bridge, culvert, etc. on a river with no predetermined flow route, $t = t_1$. On the other hand, in the case of a bridge, culvert, etc. on a river with a predetermined flow route, $t = t_1 + t_2$.

1. Inlet Time

It is standard practice to calculate inlet time using the Kirpich formula below.

$$t_1 = \frac{60 \cdot [0.87L^3]^{0.385}}{H}$$

Where,

t_1: inlet time (min.).

L: distance from the furthest point on a catchment basin to a river (km).

H: difference in height between the furthest point on a catchment basin and a point near a river.

2. Outlet Time

Outlet time is considered to be roughly equivalent to the total upstream distance of a river from the point where discharge capacity is being calculated divided by average flow velocity. Average flow velocity is calculated by applying the Manning formula below.

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

Where,
v: average flow velocity (m/sec).
n: roughness coefficient (sec/m^3).
R: A/P = hydraulic radius.
A: cross-sectional length of water flow.
P: length of wetted perimeter.
i: hydraulic gradient.

The values of Manning's roughness ratio are as follows:

Cast-in-place concrete: 0.015.

Precast concrete: 0.013.

Rivers in mountainous terrain (with large boulders & boulders): 0.040.

Rivers in flat terrain (grass, bush): 0.035.

Rivers in flat terrain (grass, pebble): 0.045.

Based on the above, oulet time can be calculated as follows:

$$t_2 = \frac{50}{3} \cdot \frac{L}{v}$$

Where,
t_2: outlet time (min.).
L: length of flow (km).

2.1.2 Calculation of Allowable Discharge Capacity

As indicated below, the allowable discharge capacity for the cross section of a waterway at a bridge crossing and the cross section of drainage facilities that intersect road such as culverts must be greater than or equal to discharge volume.

$$Q_a >= Q$$

Where,
Qa: allowable discharge capacity at bridges & culverts.
Q: discharge volume at bridges & culverts.

The approach to allowable discharge capacity for bridges and culverts is explained below.

Bridge: Allowable discharge capacity is calculated by multiplying average flow velocity obtained with the Manning formula by a river's cross section at a bridge site. Here, the surface of the river is for flood conditions, and there should be a distance of 60cm or more between the bottom of the main girder and the river's surface.

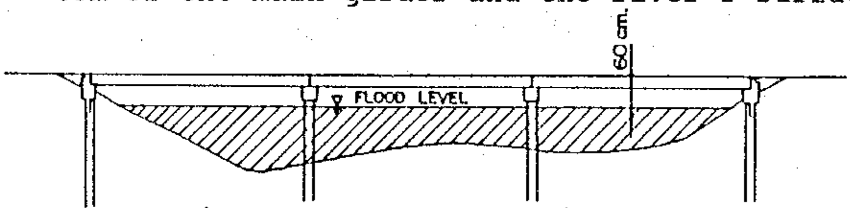


Fig.2.1.4 Cross Section of River

Culvert: The thinking here is the same as that for a bridge, except that clogging up by debris is taken into account in deciding that the water level of a culvert should only reach 80% of its height.

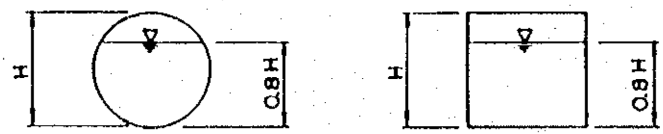


Fig.2.1.5 Culvert Cross Section

2.1.3 Ratio of River Flow Blockade by Pier

As shown in Fig.2.1.6, the ratio of river flow blockade by piers is the ratio of the total width of piers located in a river to the width of the river in a flood. It is necessary that a bridge be planned so that this ratio is 5% or less.

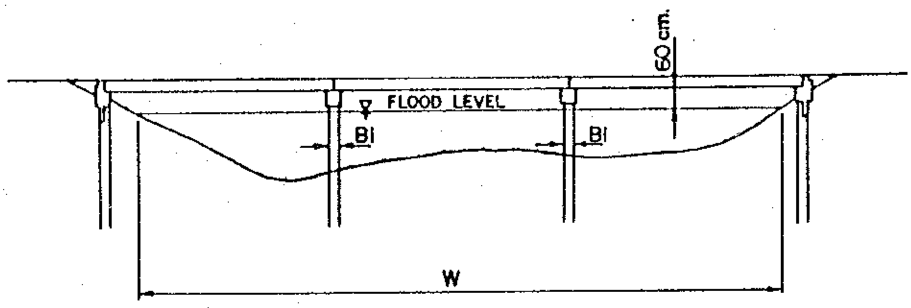


Fig.2.1.6 Conceptual Illustration on Blockade Water Flow

$$n = (B/W) \cdot 100 <= 5\%$$

Where,
n: ratio of river flow blockade by pier.
B: total width of piers in river.
W: width of river in a flood.

2.2 Bailey Bridge/Trestle

A Bailey bridge or trestle is applied in urgent repair work. Particularly, they can be applied when traffic has been stopped by either a bridge being washed out or the collapsing of a slope.

(1) Materials

In the case of a Bailey bridge, steel materials are generally used. Therefore, when other materials are applied, a verification of their ability to sufficiently stress shall be carried out.

When common steel materials such as H-shaped steel members are used, those that are most easily obtainable shall be selected. In the case of using wood materials, since the quality varies greatly with the type of wood, the same type of wood shall be applied.

(2) Decks

Since reinforced beams have to directly support the load of vehicles, they must have sufficient strength and durability. There are steel, concrete, and wooden decks. Therefore, the appropriate type of deck should be chosen taking into account the load of vehicles.

(3) Load

When designing, the following loads shall be considered:

1. dead weight (weight of structure),
2. live load (vehicle load), and
3. impact load (load from the stopping & starting of vehicles).

$$I = 50 / (L + 125)$$

Where,
I: impact load (maximum 30 %).
L: length of the span (m).

(4) Distance between Girder Bottom & River

The distance between the bottom of a bridge's girder and a river shall be 60cm or more.

(5) Foundation

The foundation that will support a Bailey bridge or trestle shall be sufficiently strong for this purpose. Furthermore, when piles are used for support, they will be driven 3m or more into a soil layer that is gravelly or hard silt and has a value of 30 N or more, or diluvial clayey soil with a value of 10 N or more.

2.3 Abutment Protection

To protect bridge abutments and the backs of bridge abutments, revetments shall be used at back-fills. Standard revetments are usually concrete; however, stone riprap with mortar is also permissible for repairs or if there are supply problems.

Whatever method is used, be it either concrete foundation work (which must go at least 1m under the ground) or sheet pile work, the purpose is to strenghten the toe of a slope on a river embankment (see Fig.2.3.1).

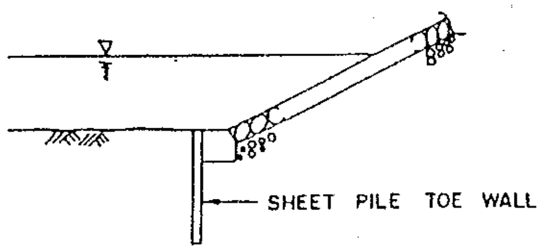


Fig.2.3.1 Strengthening of Slope Toe on River Embankment

Furthermore, as shown in Fig.2.3.2, it is desirable to protect the riverbed facing the above-mentioned protection work using gabion mat, etc.

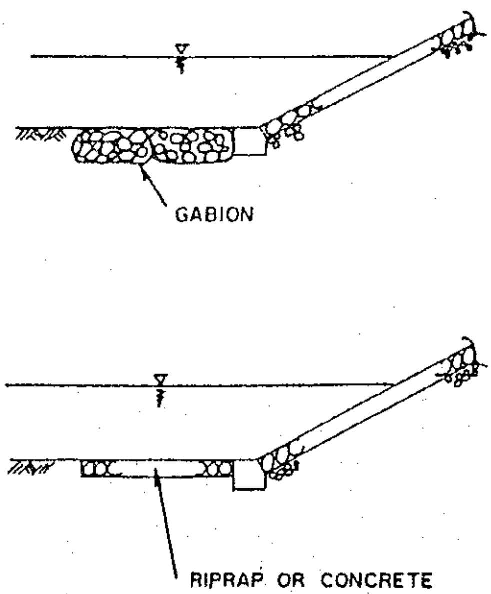


Fig.2.3.2 Protection of Riverbed & Toe of Slope

2.3.1 Concrete Revetments

- Application

Revetments are reinforced concrete structures that are applied to slopes with gradients gentler than 1:1.

- Materials

Concrete for structures, reinforcing bars, joint filler, cobblestones,
and PVC pipe for draining water.

- Design

As for the cement, sand, and gravel mix of concrete, it shall have a weight ratio of 1:2:4, and the slump shall be 10cm or less.

As for the thickness of concrete, for a slope gradient of N:1, it shall be as follows:

N >= 1.5: 10cm (DOH standard).

<= N >= 1.5: 20cm.

To prevent surface cracking, contraction joints shall be installed every 2m or less in a vertical direction and every 3m or less in a horizontal direction (see Fig.2.3.3). Furthermore, as shown in Fig.2.3.4, the joints will be plugged with mastic joint filler.

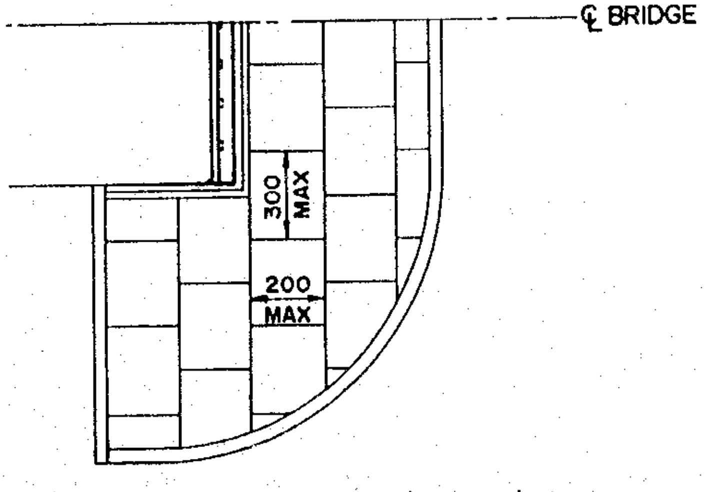


Fig.2.3.3 Contraction Joints

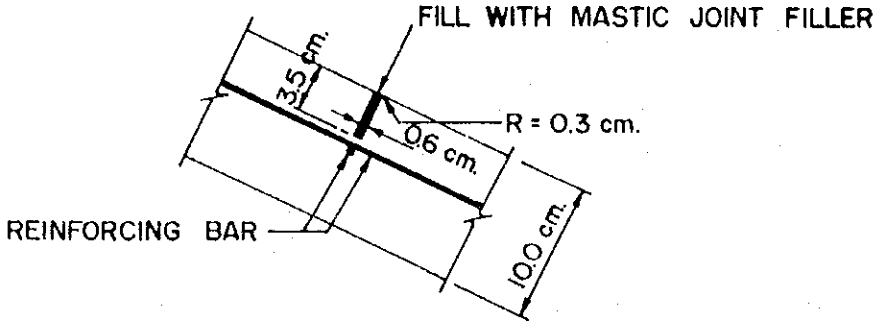


Fig.2.3.4 Mastic Joint Filler

To alleviate water pressure at the back of an abutment, as shown in Fig.2.3.5, back-filling cobblestone 20cm in thickness shall be laid from the front to the back of the abutment. Furthermore, every $2m^2$ PVC pipe 50mm in diameter shall be installed to drain water.

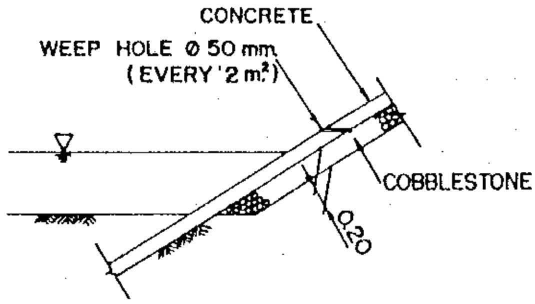


Fig.2.3.5 Cobblestone

To protect a riverbed from scouring, the depth of the portion of foundation to be embeded shall be at least 1m and be made of concrete (see Fig.2.3.6).

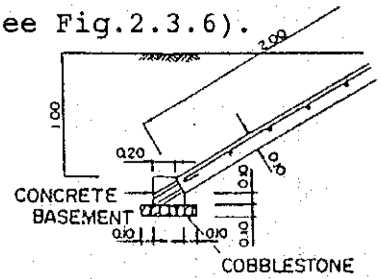


Fig.2.3.6 Concrete Foundation

2.3.2 Stone Riprap Revetments with Mortar

- Application

This type of revetment is made of stone riprap and is applied to slopes with a gradient gentler than 1:1. In the case of the gradient being steeper than 1:1, than a stone riprap wall is applied as described later.

- Materials

Cobbelstone or crushed rock, mortar, concrete for small structures, PVC pipe for draining water.

- Design

The cement and sand mix for mortar is by volume 1:2.

To alleviate water pressure at the back of an abutment, back-fill cobblestone is laid from the bottom to the top of the abutment, with PVC pipe installed every $2m^2$ to drain water (see Fig.2.3.7).

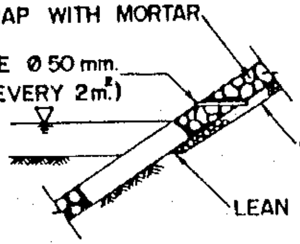


Fig.2.3.7 Cobblestone

The thickness of riprap and the back-fill cobblestone for a slope gradient of N:1 is as follows:

Slope Gradient	Thickness of Riprap	Thickness of Back-fill Cobblestone
N >= 1.5	25cm	10cm
<= 1.0 N < 1.5	45cm	20cm

In order to protect a riverbed from scouring, the portion of foundation to be embeded will be at least 1m and be made of concrete (see Fig.2.3.8).

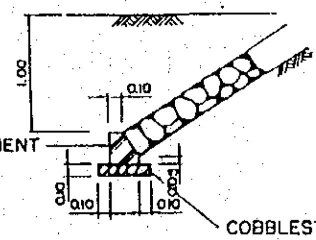


Fig.2.3.8 Concrete Foundation

2.4 Pier Protection

Bridge piers disturb the river flow in the vicinity of a bridge pier. For this reason, scouring of the riverbed in the area of a pier foundation occurs (see Fig.2.4.1). There are cases where this results in the piers tilting to one side and the bridge collapsing.

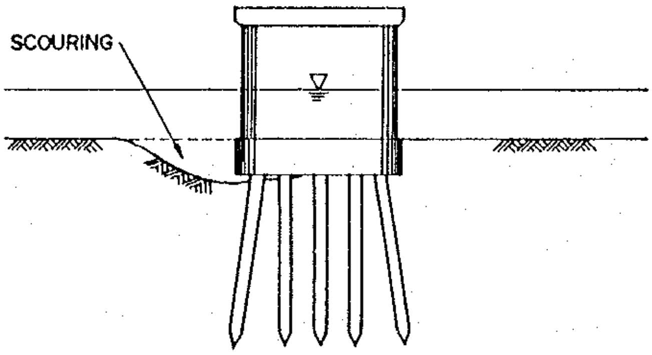


Fig.2.4.1 Scouring of Pier Foundation

Therefore, work to protect piers from on-going or expected scouring of their foundations shall be carried out.

- Confirmation of Bearing Capacity

Most of bridge piers are of the pile-bent type in Thailand. First it is necessary to confirm whether a bridge pile is a bearing- pile or friction-pile type.

In the case of a bearing-pile type pier, as shown in Fig.2.4.2, since the bottoms of the piles support the acting load, partial scouring will not have a large effect on bearing capacity.

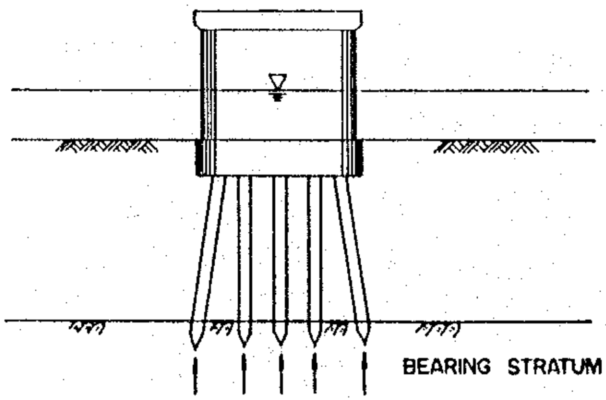


Fig.2.4.2 Bearing-Pile Type Pier

In the case of a friction-pile type pier, as shown in Fig.2.4.3, since the friction along the faces of the piles supports the acting load, it is necessary to carry out sufficient verification of bearing capacity in regards to partial scouring.

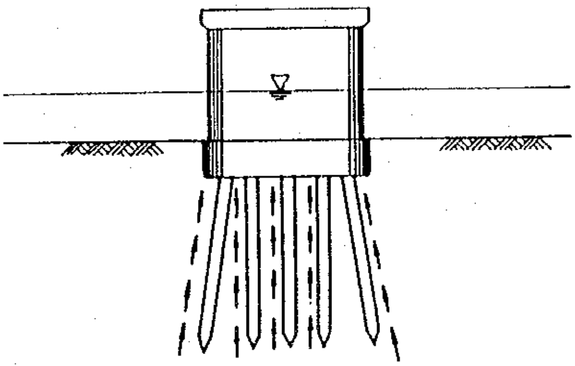


Fig.2.4.3 Friction-Pile Type Pier

Below, the decrease in friction for a sample area scoured in Fig.2.4.4 is calculated in the next equation.

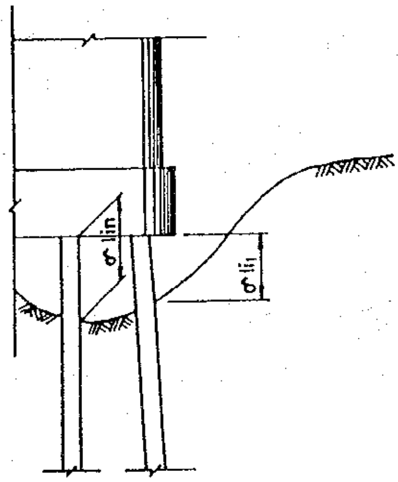


Fig.2.4.4 Change in Friction at Pile Face

$$\delta P = D\,\Sigma l \cdot f$$

Where,

δP: change in friction for scoured area.

D: radius of pile.

Σl: length of scouring = l_i

f: friction at pile face.

- Types of Protection Work

Pier protection work consists of the following types of work:

1. sand mat work,
2. crushed stone work,
3. mat gabion work,
4. cyclinder gabion work, and
5. concrete revetment work.

When selecting a type of work in the design stage, before considering material availability and economics, it is necessary that the situation of the affected area be sufficiently understood.

- Area to Be Protected

The area to be protected consists of the entire scoured area, and is to be completely covered.

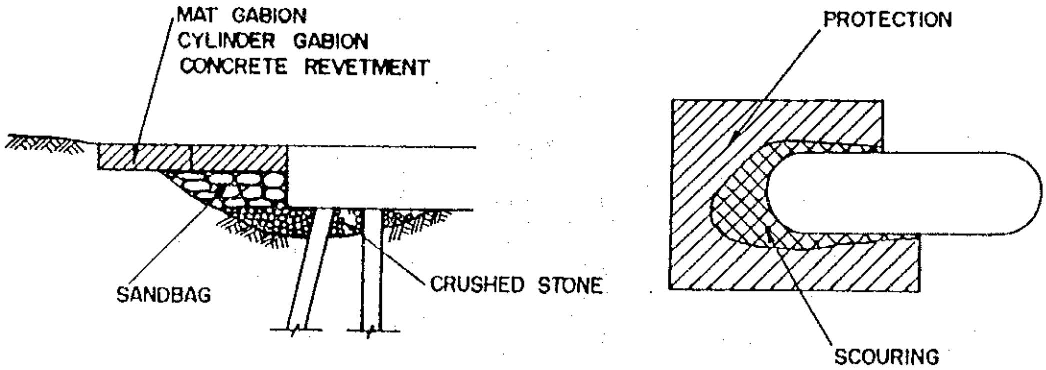


Fig.2.4.5 Area to Be Protected

2.5 Protection of Bridge Access Roads

When a bridge is located in a flood plain, since there is the fear of the approach roads being scoured by the stream paralleling embankment (see Fig.2.5.1), it is necessary to carry out work to protect slopes on the upstream sides of the access roads.

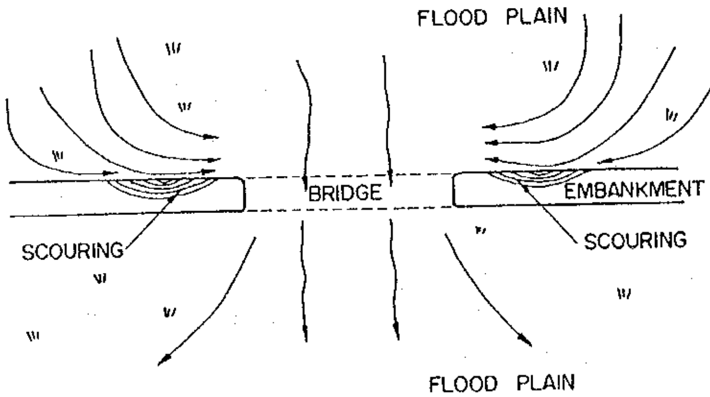


Fig.2.5.1 Flood Flow at Waterway Opening

(1) Selection of Work

Protection work consists of the following types of activities:

1. concrete revetment work,

2. articulated concrete revetment work,

3. stone riprap revetment work, and

4. cribwork with stone riprap.

Concrete revetments should be used at places where the water flow is rapid and rough. As for the other types of protection work, material availability, consistency with work around the bridge abutments, esthetics, etc. should be considered. However, in the case of an articulated concrete revetment, slope gradient must be gentler than 1.5:1 and the height can be no higher than 3m. The slope gradient for all other work is to be gentler than 1.0:1.

No matter what kind of protection work is chosen, the portion of foundation to be buried underground shall be at least 1m. Furthermore, it is desirable that the riverbed at the front of the protection work be protected from scouring by mat gabion, etc.

Below, the characteristics of the different types of protection work are explained.

(2) Concrete Revetments

Same as in Section 2.3.1.

(3) Articulated Concrete Revetments

- Application

This structure is applied to slopes gentler than 1.5:1 and used with precast concrete blocks.

- Materials

Precast concrete block, cobblestone, PVC pipe for water drainage.

- Design

As shown in Fig.2.5.2, the precast concrete block is 12cm thick and interlocking.

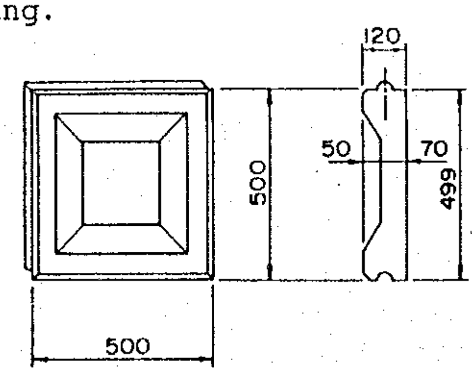


Fig.2.5.2 Articulated Concrete Revetments

- Water Pressure

To reduce water pressure at the back, a 20cm thick layer of cobblestones is laid from the front end of the protection work all the way to the back, and PVC pipe with a diameter of 50mm is installed every $2m^2$ to drain water.

- Foundation

As shown in Fig.2.5.3, a concrete foundation is installed and the buried portion at least 1m deep.

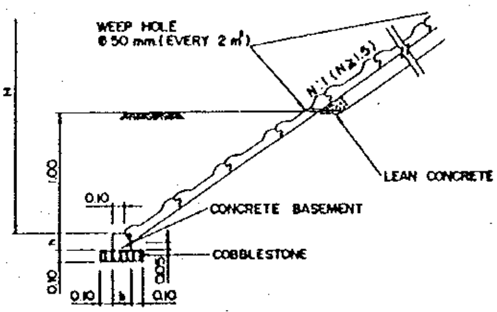


Fig.2.5.3 Concrete Foundation

(4) Stone Riprap Revetments with Mortar

Same as Section 2.3.2.

(5) Cribwork with Stone Riprap

- Application

This structure is applied to slopes with a gradient gentler than 1.0:1 and the cribs filled with stone riprap and mortar.

- Materials

Precast concrete crib bock, cobblestone or crushed rock, mortar, concrete for small structures, PVC pipe for water drainage.

- Design

The span of a crib is to be 1m. The dimensions of the crib members are to be 15cm x 15cm or 15cm x 20cm. Also, an anchor pin 50 to 100cm long will be used at crib junctures to prevent sliding, and the iron wires protruding from the crib members tied and mortar poured in.

The contents of a crib will consist of stone riprap with mortar. For the surface, rocks will be laid unevenly to slow water flow and prevent erosion.

To alleviate water pressure at the back of a slope, cobblestone 10cm in thickness are laid from the front of the protection work all the way to the back. In addition, PVC pipe is installed every $2m^2$ to drain water.

In precast concrete cribwork, the type of foundation laid is as shown in Fig.2.5.4.

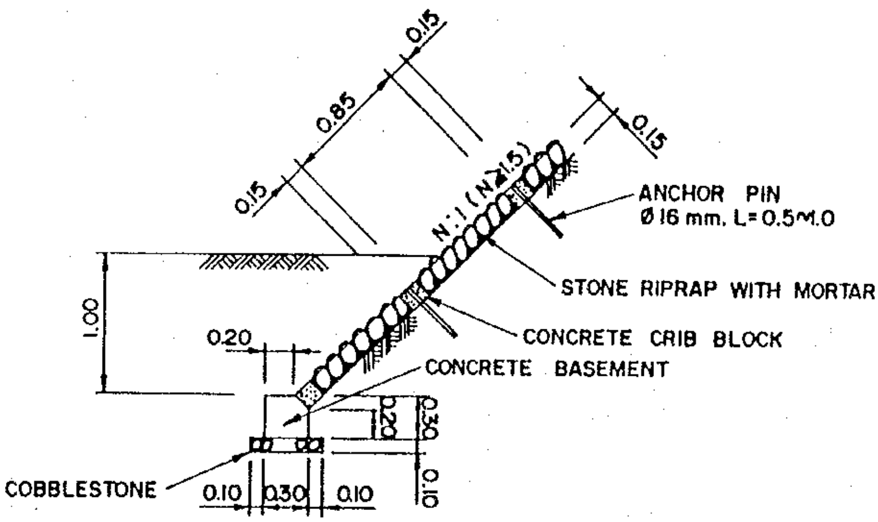


Fig.2.5.4 Concrete Foundation

2.6 Protection of River Bank

In the case of a winding river, riverbank protection is installed in order to prevent damage to a bridge feared from scouring and to rectify river flow. The scope of this protection work, as shown in Fig.2.6.1, must at least cover the area from where the outside bend of the river begins and ends.

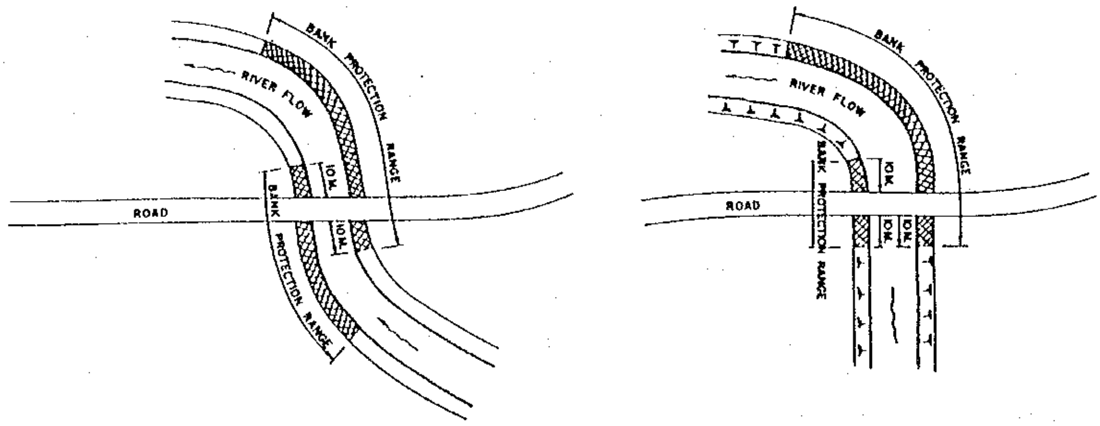


Fig.2.6.1 Scope of River Bank Protection Work

- Selection of Work

In protecting an approach road, there are the following types of protection work:

1. concrete revetments,
2. articulated concrete revetments,
3. stone riprap revetments with mortar,
4. cribwork with stone riprap,
5. gabions, and
6. dumped rock.

Concrete revetments should be applied to areas where the water flow is fast and rough. This means that they should be applied to the outer bend of a river. As for gabions and dumped rock, they should be applied to low places of relative importance, such as a low water channel in an emergency situation. Regarding the other types of work, they should be selected taking into consideration material availability, consistency with previous work, esthetics, etc. However, for an articulated concrete revetment, the slope gradient must be gentler than 1.5:1 and its height no higher than 3m. The slope gradient of all other work is to be gentler than 1.0:1.

Excluding work with gabions and dumped rock, the portion of foundation to be underground shall be at least 1m or more in depth. Furthermore, in order to prevent scouring, it is desirable that gabion mat, etc. be laid in the riverbed at the front portion of the protection work.
Below, the different kinds of protection work are explained.

(1) Concrete Revetments

Same as the last section.

(2) Articulated Concrete Revetments

Same as the last section.

(3) Stone Riprap with Mortar

Same as last section.

(4) Gabions

- Application

Since the corrosion of the wire netting of gabions shortens their service life, they are either applied to locations of low priority or urgent or temporary work is carried out (see Fig.2.6.2).

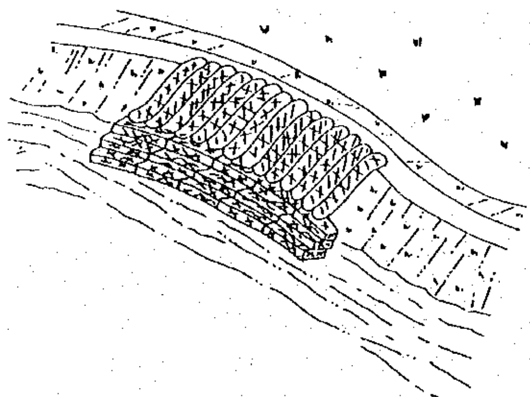


Fig.2.6.2 Gabion Work

There are two types of gabions: gabion mat and cyclinder gabion. Below, these two types are described.

1. Gabion Mat

   Gabion mat is applied to slopes with a gradient gentler than 1.5:1 that require a change in their configuration. It is especially suitable for machine-based work.

2. Cylinder Gabion

   Cylinder gabion is applied to a slope gradient gentler than 0.5:1. It also has a flexible structure and can be applied to an uneven slope. It is especially suitable for manual work.

- Materials

Wire net for gabions, cobblestone, and crushed rock.

- Design

The size of the net hole shall be 10cm. However, depending on the cobblestone available, this could vary.

In the case of gabion mat, a mat is laid from the toe and the shoulder of a slope (see Fig. 2.6.3).

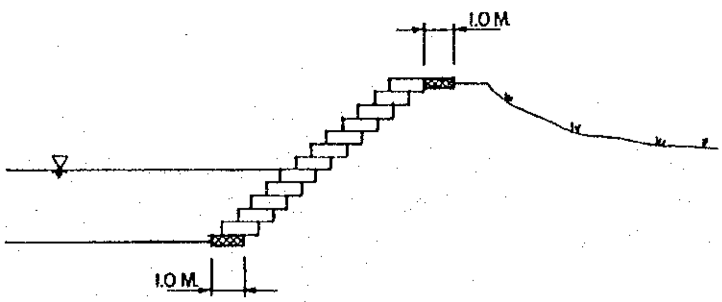


Fig.2.6.3 Gabion Mat

In the case of cylinder gabion, it is installed so that it extends 1m out from the shoulder of a slope and 1.5m out from the toe of the slope (see Fig. 2.6.4).

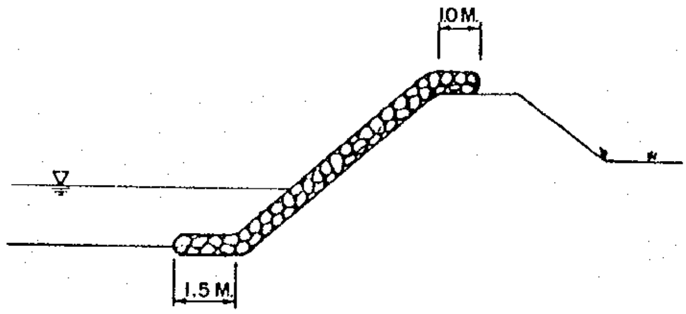


Fig.2.6.4 Cylinder Gabion

(5) Dumped Rock

- Application

This structure crumbles rather easily, but it is also easy to implement. It is therefore used at locations of a relatively low priority during urgent and temporary repair work.

The slope gradient should be gentler than 1.5:1 or stability could be adversely affected.

- Materials

Cobblestone or crushed rock 150 to 300mm in diameter.

- Design

As shown in Fig.2.6.5, cobblestone or crushed rock is laid over an entire slope, and extends 1m out from the shoulder of the slope and 1.5m out from the toe of the slope. It is then pounded into the ground.

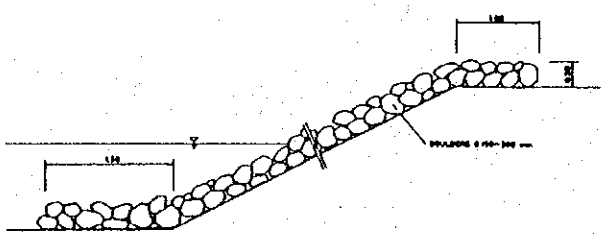


Fig.2.6.5 Dumped Rock

2.7 Guide Dike

- Application

In the case of a bridge being in a flood plain, when there is the fear of the approach parts or the abutments of a bridge being scoured by flood-induced eddies, then a guide dike can be applied if there sufficient space and materials such as soil (see Fig. 2.7.1).

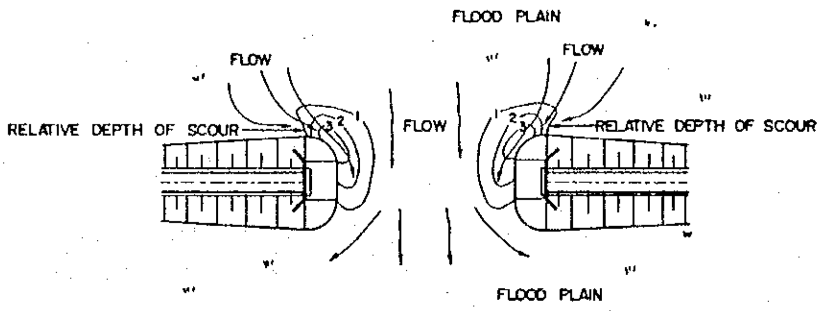


Fig. 2.7.1 Flood Flow and Scouring at Bridge Opening

As shown in Fig.2.7.2, a guide dike keeps eddies away from the abutments. Therefore, even if a guide dike collapses, the abutments are still in place and this serves to keep the river flow on its proper course.

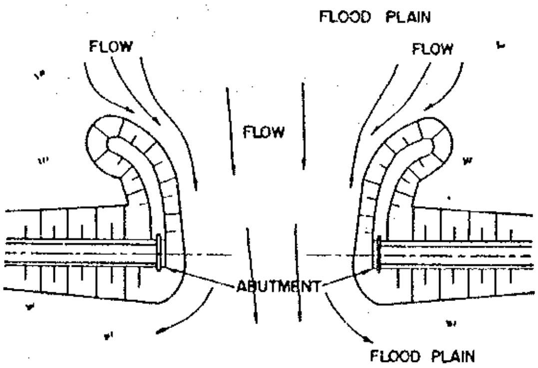


Fig. 2.7.2 Equipment of Guide Dike

- Materials

Fill used for fill slopes, revetments.

- Design

The design of a guide dike shall be carried out as follows:

1. The front slope (which faces the abutment) shall have a gradient of 2.0:1.

2. The back slope shall have a gradient of 1.5:1 and extend to the surface of the road.

3. The levee crown will be 2m.

4. The length will be 20m.

5. Each layer of a fill slope shall be 30cm in thickness.

6. Slope protection work shall be carried out as inexpensively as possible and so that later repairs can be easily made. Dumped rock and sand bags seem to be appropriate. Fig.2.7.3 shows a standard guide dike.

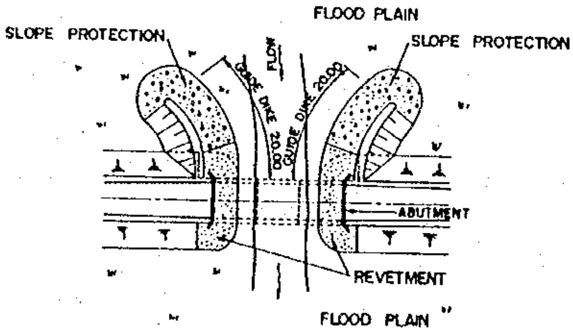


Fig. 2.7.3 Standard Guide Dike

3. Collapsing of Road Embankment

3.1 Culvert Replacement

- Application

In the case of an existing culvert either lacking the capacity to handle the discharge of a waterway or being forecasted to be clogged by debris, it should be replaced with a structure having a larger cross section.

- Materials

Pipe culvert, box culvert, bridge.

- Design

In the case of a culvert with insufficient discharge capacity but having no problems with debris, either the cross section should be enlarged or the vertical gradient made steeper. As shown in Fig.3.1.1, there should be a leeway of at least 20% inside a culvert.

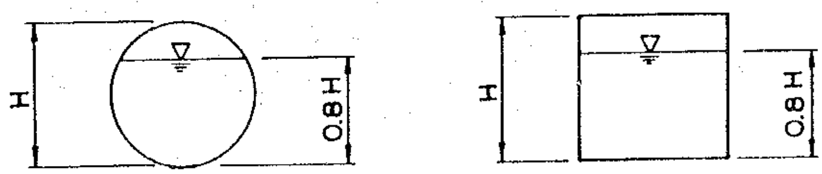


Fig. 3.1.1. Required Leeway in a Culvert

However, as shown in Fig.3.1.2, the multiple pipe culvert is uneconomical for handling increases in discharge capacity and should in principle be avoided.

From a maintenance viewpoint, it is desirable that a culvert be 1.5m or larger in diameter to allow the easy removal of debris.

In areas where there are mud and debris flows, it is necessary of course for culverts to have cross sections large enough to handle these flows together with the appropriate leeway. This is done by investigating the boulders upstream and measuring the largest boulders. The cross section of culverts are then designed to be 1.5 to 2 times the diameter of this boulders. In addition, multiple pipe culverts should be avoided and single pipe culverts with large cross sections used.

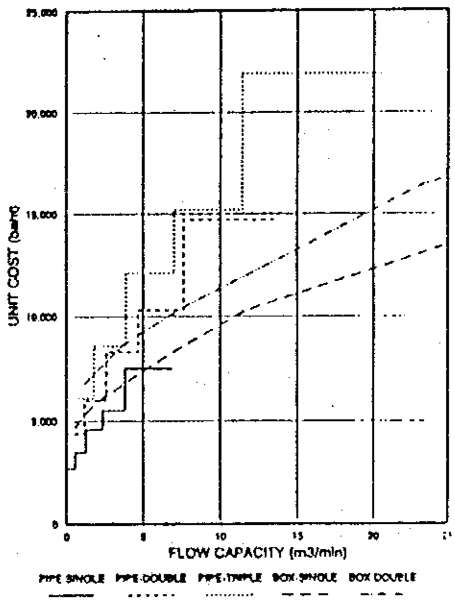


Fig.3.1.2 Relationship between Discharge Capacity & Cost

At places where there is floating debris, a box culvert with a large cross section or a bridge must be used. In the case of a bridge, the distance between the bottom of the bridge and flood waters should be at least 60cm or more.

3.2 Repairing Road Shoulder Damage

- Application

This type of work is carried out to restore the shoulder of a road damaged by scouring caused by flooding.

- Materials

Soil, planting material, sandbags, wicker work, etc.

- Design

As Fig.3.2.1 and Fig.3.2.2 show, a road shoulder is prone to collapse in a flood when the road is located in either a flood plain or on sloping ground.

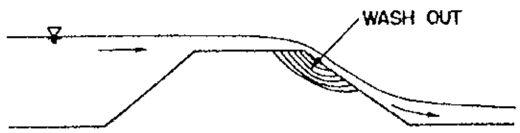


Fig.3.2.1 Road in a Flood Plain

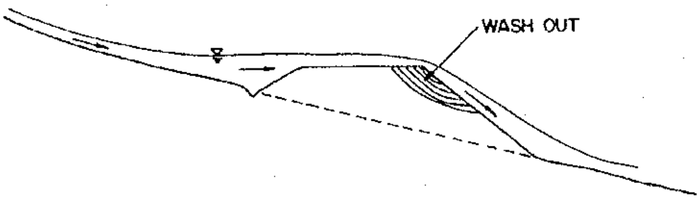


Fig.3.2.2 Road on Sloping Ground

For road shoulders that have collapsed in refilling work, stepping is carried out where the gradient is steeper than 4:1 and where the soil is very loose.

When carrying out the repair work, attention shall be paid to the following:

1. the thickness of a layer shall be 20cm,

2. compaction should be done for each layer using a compactor (vibrating roller, tamper, rammer),

3. a completed slope should be compacted by tamping, and

4. vegetation work (eg, stripe sodding) should be carried out quickly on the subject slope, but, if it is

the rainy season and there is the fear of another collapse, wicker work and sandbags should also be applied.

Furthermore, the amount of rainfall in an area should be examined and consideration given to building gutters, toe ditches, vertical ditches, culverts, etc. The thinking concerning these facilities has been discussed previously.

4. Road Flooding

4.1 Raising of Raising Roadway Embankment

- Application

At sections where a road is made impassable due to flood waters, the road shall be raised above the flood water level to prevent this from happening if feasible.

- Materials

Soil, paving material, materials for slope and culvert work.

- Design

When raising a road, it is necessary to confirm in advance the height of the maximum flood water level above the road's surface. Also, the vertical alignment should be carried out by connecting other spots.

In principle, the distance between the flood water level and a road's surface shall be 50cm.

To drain rainwater sufficiently, it is important to enlarge culvert cross sections and build new culverts sufficiently large.

Before raising an embankment, when using an existing culvert that is not long enough, the culvert should be lengthened as shown in Fig.4.1.1.

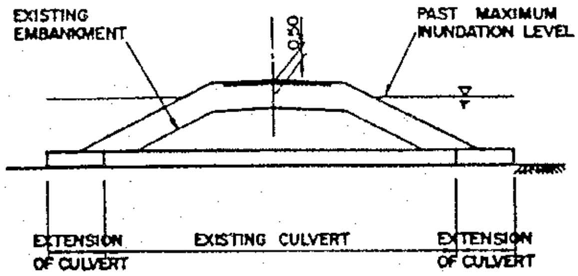


Fig.4.1.1 Lengthening Existing Culvert

At places where there is asphalt concrete paving, this shall be removed and stepping carried out on the slope for refilling work. Stepping shall be executed as described in the previous section. Also, stepping shall be executed where the surface is weak. Details of the work are as follows:

1. the thickness of a fill layer should be 20cm,

2. compaction should be done for each layer with a compactor (vibrating roller, tamper, rammer),

3. a completed slope should be compacted by tamping,

4. vegetation work such as stripe sodding shall be carried out quickly.

4.2 Culverts

- Application

A culvert is applied to the horizontal waterway of a road.

Culvert use depends on the amount of rainwater, but mud and debris flows and floating debris (eg, trees) require that the cross sections of a culvert be larger than usual.

- Materials

Concrete for structures, cribs, reinforcing bars, concrete pipe, etc.

- Design

Culverts and inlets/outlets shall follow DOH standards.

As discussed in Section 3.1, in terms of cost and discharge capacity, the single pipe culvert should be used and the multiple pipe culvert in principle avoided.

Concerning the cross section of a culvert, they shall be designed to have excess capacity of 20%. Also, for maintenance reasons, culverts should be 1.5m or larger in diameter.

In places where there are mud and debris flows, culverts shall be built so as to handle these flows together with the necessary leeway. This shall be done by determining the maximum size of boulders that flows downstream and to build the cross sections 1.5 to 2 times that size. Also, large single pipe culverts should be used and not multiple pipe culverts.

As for inlets, when the channel, a training wall should be built to water smoothly guide to an inlet (see Fig.4.2.1). The standard length is 10m.

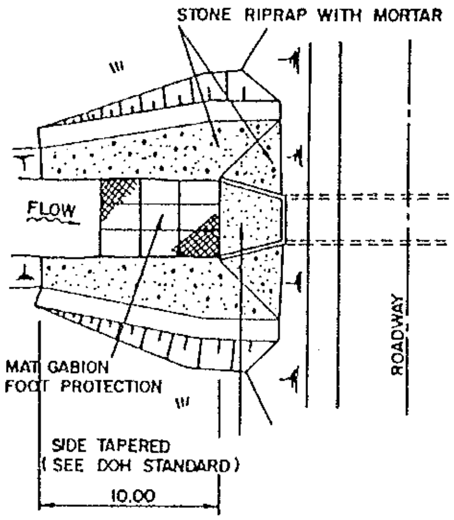


Fig.4.2.1 Inlet of Culvert

As for the front base of an inlet, mat gabion is used to prevent scouring. In the case of when there is a difference in the height of an outlet and the downstream river, an open (or closed) chute is installed to prevent scouring (see Fig.4.2.2).

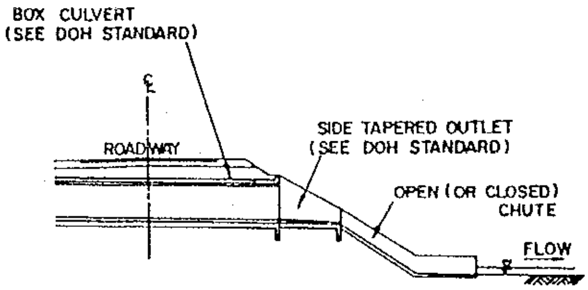


Fig.4.2.2 Open Chute

When part of the bed of a waterway is being eroded away, stone riprap with mortar is used to prevent further scouring (see Fig.4.2.3).

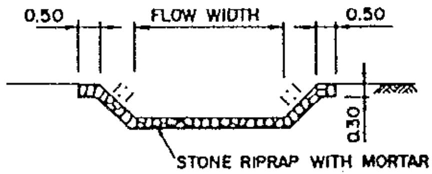


Fig.4.2.3 Stone Riprap for Stream