Chapter 24*

DETAILED SURVEY AND DESIGN

24.1 INTRODUCTION

Once an alignment has been decided upon through feasibility studies, detailed survey, and design should be carried out to determine the accurate details of road profiles, structures, designs, dimensions, quantities of various items of works, construction methods, maintenance strategies, drawings, cost estimates, and contract documents.

It is expected that i) the estimated quantities and costs at the detailed stage will be within + 8 per cent, ii) the overall quantities and costs from feasibility studies will not be exceeded by more than 15 per cent, and iii) that designs result in lowest risk to and from the structures within the budget allocated by the feasibility criteria.

Even though the alignment will have been fixed during the feasibility stage, the exact alignment, along with accurate cross-sections and long-sections is finally set out only by the detailed survey and designs. Variations from the feasibility survey, including minor changes in the alignment, are inevitable. Non-involvement of an engineering-geologist during the detailed surveying is likely to make the locator/surveyor indifferent to engineering-geological considerations and may result in locations falling unduly outside the boundary of the area for which the hazards and risks were assessed during the feasibility survey. Such a situation will lead to the redundancy of feasibility recommendations.

Traditional detailed surveys do not include engineering-geological and geotechnical information in each cross-section. Design of cut slopes and retaining structures, and the rate analysis for cost estimation, cannot be reliable.

Traditional survey and design techniques are not based upon information on the instability conditions that constitute hazards and risks and, as a result, the designs, often times, lead to acceleration of instability, i.e, increase in hazard levels with consequential increase in risks.

Designs should be based on hazards identified during the feasibility stage. However, since the details of road profiles at 20 metres to 100 metres are required at this stage, hazard information is required for smaller sections, i.e., 20-100 metres. Detailed hazard assessment with the help of 1:5000 scale maps, including slope stability assessments, is therefore required for critical sections (medium and high hazard sections). Geotechnical information, such as the angle of shearing strength, cohesion unit weight, standard penetration value (SPT), stratification, infilling in rocks, waviness of rocks, joint spacing and orientations, water table, and rock and soil types with their depths should be assessed from visual observations for

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most areas and by tests for critical areas. These should be indicated for each cross-section. The properties obtained should account for variations from the geological process in the watershed influencing the road.

Assessment of risk in the design stage involves comparison of costs for the alternative designs which are safe enough for the design life and use of probability, expected value, and decision tree. Only critical structures that are high cost structures may be considered for such analyses.

24.1.1 *Detailed Survey*

Detailed survey involves a number of processes and these are described in this section.

- Obtaining precise details of ground profile along the road corridor, either by establishing a base line and contouring on a large scale, i.e, at contour intervals of 1 metre or less on 1:500 to 1:2000 scale maps or by directly fixing the centre line of the road in the field and obtaining long sections along the centre line and cross-sections across the centre line. The survey should, in any case, be carried out so that cross-sections at 1:100 to 200 and long sections at 1:1000 to 2000 in scale can be plotted.

  A contour survey is preferable since this will allow optimum design of horizontal and vertical alignments through desk work, provided the strip of the survey is wide enough, say at least 100 metres on both sides of the centre line, and additional surveying is done to cover the areas of sections of alternatives that fall beyond the regular strip. Direct fixing of alignments and measuring long and cross-sections are beneficial since more accurate details of profiles are possible, but this does not allow for horizontal shifting of the centre line during geometric designs

- Cadastral surveying to obtain details of land and property acquisition

- Hydrological, hydrogeological, and hydraulic surveying to obtain details of groundwater, rainfall, and stream behaviour

- Engineering geologic mapping on a 1:5000 scale (and high hazard areas) in critical areas to obtain engineering geological and geotechnical details over a wider area influencing the road.

- Geotechnical data collection to obtain soil and rock strength parameters for every cross-section at 20m intervals. This may be done by visual observation and by the use of quick methods, e.g., geological hammer, Schmidt hammer, and charts for most areas; and by laboratory and field tests for critical areas.

- Materials' data collection to identify sources of construction materials.

Detailed design also involves the following steps.

- Optimum design of horizontal and vertical profile of road.
- Design of road formation, cut slopes, and blasting.
- Design of retaining structures above and below the road.
- Design of erosion control measures such as check dams, cascades, and vegetation and plantations.

- Design of landslide stabilization.
- Design of surface drains, sub-surface drains, and culverts.
- Design of bridges.
- Design of river protection.
- Design of pavement.
- Estimation of quantities.
- Rate analysis.
- Estimate of cost.
- Preparation of contract document and specifications.

The sections that follow provide design approaches, supplemented by tables, figures, and design examples with background discussions on the problems to be addressed by the design solutions. Part I should be referred to whenever the user of this application guide wishes to understand the theory behind the application.

24.2 HAZARDS AND RISKS

24.2.1 *General*

The approach to hazards and risks, at this stage, should be different from the approach in the earlier stages since there is no alternative alignment to be compared. However, minor changes and improvements in route location from that determined in the feasibility stage are inevitable. Engineering-geological information is, nevertheless, essential to identify causes of instability from slope effects up and down the road, and to obtain information on the variability of properties of the slope material affecting the road design. Engineering-geological data collection and mapping should be done on a scale of 1:5000 for high hazard areas and other clearly critical areas.

Detailed geotechnical information should be collected for all the critical areas requiring slope stability analysis. A detailed cross-section is required within 2 to 5 metres for slope stability analysis as well as accurate estimation of quantities. Hazard assessment is rather deterministic at the detailed stage because calculation of the factor of safety is involved. However, the uncertainities in the material properties and water table, if considerable, should be dealt with by a probabilistic approach. A decision tree should be used in the choice of alternative designs for high cost and critical structures.

24.2.2 *Detailed Stage Engineering-Geological Studies*

The objectives of the detailed stage engineering-geological and geotechnical studies are to provide the information necessary for the road engineer for more detailed cost and quantity estimates and for detailed design of the least hazardous road.

Walk-over Survey for the Final Alignment Location

A walk-over survey along the route selected from the feasibility studies is performed by a team composed of geologists, road location engineer, and surveyors. The walk-over survey offers the opportunity for small shifts and re-alignments in the route and for adjustment in grade. The survey should minimize the hazardous areas, and, at the same time, the position of structures, side drains, and other facilities should be tentatively decided. Benchmarks, intersection points, and turning points, with sufficient curvature and other control points, are carefully referenced on the ground.

Engineering-Geological Studies and Mapping

The route fixed from the walk-over survey should be plotted at 1:5000 (or larger) scale and a draft engineering-geological map should be produced in the field. At the same time, the rock and soil sampling locations and the sites for detailed geotechnical, geophysical, and drilling works are fixed. Also, the locations necessary for stability analysis are marked in the field for data collection for detailed cross-sections in the detailed survey stage. The engineering-geological studies should pay greater attention to such critical areas as:

- major lanslide areas,
- major gully erosion sites,
- debris or mudflows,
- areas with high cuts (> 20 m),
- areas requiring large retaining walls,
- bridge crossings, and
- areas of major bank scouring

Rock and soil identification and classification as techniques, discussed in Chapter 23, also provide a basis for this stage of the study. Further information on rocks and soils is given in Chapters 9 and 10. Generally, the following parameters should be obtained for soils:

- genetic and unified soil classification,
- friction angle and cohesion,
- unit weight,
- porosity and permeability, and
- tension cracks (depth, width, and length).

Rocks should be studied for:

- composition and texture,
- alternation, interbedding,
- joint sets, spacing and width,
- waviness, infilling ,
- cubic compressive strength,
- weathering grade, and
- tension cracks (depth, width and length).

For critical areas, the measurement and analysis of discontinuities together with the friction angle and stereonet can help in designing the cut slope. The data can also be used for RQD, RMR, and SMR classifications (see Chapter 10).

Different from the feasibility stage, the detailed stage engineering-geological map exhibits the type and extent of landform, dangers, hydrological conditions, land use, rock and soil types, and groundwater conditions. The map should cover a corridor of not less than 100m up and 100m down the road (if there is a river; down to the river). The map should provide a dynamic picture of the terrain and predict the future trend of the geological activity.

The engineering-geological map should be prepared on the basis of the above information and following remarks.

- Landforms such as river terraces (their extent, depth, and order), talus cones, cliffs, fans, badlands, ridge and spur lines, gullies, and rivers should be shown on the map for about 100m up and 100m down the road. Both banks of the river should be shown, as far as practicable.

- Every existing medium and large landslide should be studied for the material type, depth of the slide, slope of failure, depth of the tension crack and its dip angle, and probable cause of failure (excessive rain, river undercutting, earthquake, etc). Its extent should be plotted on the map with appropriate symbols for various rock and soil slides. For dormant slides, it is necessary to predict the extent and mode of probable failure and the probable area affected by it. The old landslide scars should also be mapped and shown on the map with the above-mentioned information.

- All active gullies should be studied for the possibility of widening or shifting, probable damage to the road length, the sediment load and discharge, depth, width and slope of the banks, as well as the soil or rock on which the gully is developed. The gully slope and direction should be measured with a compass and shown on the map for at least 200m up and down the road.

- The location and extent of alluvial fans, mudflows, and debris flows should be accurately measured and drawn on the map. The fans and flows should be divided into several sub-zones depending upon the probability of occurrence of the danger. The composition variation, shifting, and depth of the fan or flow materials should also be recorded and shown on the map.

- The site for construction material should be marked on the map, and a cursor classification of the construction material should be made. There should be a detailed description of the landform, rock and soil type, dangers, and triggers attached to the above map. The description should be detailed enough to give the data on the rock and soil for the design of cross-sections and profiles. Therefore, the observation points should be put at a minimum interval of 50m in simple terrain. The interval should be varied accordingly for complicated and hazardous terrain in order to get the information in detail.

Geotechnical Studies

Geotechnical studies are carried out for the critical areas marked during the engineering-geological studies and they generally include rock and soil sampling, *in situ* testing, and study of groundwater conditions.

Geophysical Studies

Detailed geophysical studies, consisting of seismic (refraction) and electrical (resistivity) methods (Chapter 2) are effective in assigning the geological and hydrogeological conditions below the surface at relatively low costs. Geophysical methods can be used for a quick estimation of the sub-surface conditions. In many cases, geophysical methods may be applied effectively and quickly, especially in shallow depth, and this can save the cost of drilling.

Drilling

Core drilling should be performed at critical areas, such as bridge sites, etc. and in investigating landslides and debris flows. It is recommanded that they be carried out conjointly with geophysical studies in order to reduce their number. The type of drilling equipment depends on the accessibility of the site.

24.3 ROAD FORMATION DESIGN

24.3.1 *Problems*

The alignment and geometric standards of a hill road are broadly defined by feasibility studies. Many problems associated with stability and costs are likely to appear during the detailed survey and design phase, in spite of attempts to avoid them during alignment selection during the feasibility stage. The problems encountered are:

- heavy cutting in soils and rock,
- heavy rock blasting,
- heavy retaining walls and breast walls,
- heavy filling,
- progressive destabilization of cut slopes and influence area slopes, and
- disposal of excavated rocks and soils.

24.3.2 *Guidelines*

The following criteria are suggested for the design of road formations.

- Proceed with the road formation design with the help of hazard maps or assessments prepared in the feasibility stages.

- Balance cut and fill for each cross-section in areas where fill does not require retaining structures or can be retained with low height (less than 3m) and at low cost, e.g., drystone masonry, retaining wall.

- Lower geometric standards in critical areas.

- Ensure compatibility of vertical grade, horizontal radius, design speed, and sight distance. For example, an 8 per cent vertical grade makes it impossible for a vehicle with a weight of a horse-power ratio of 300 lb/HP to drive at a speed greater than 15 km/hr. There is, thus, no point in designing road formation for a speed of 30 km/hr for such a grade and vehicle. However, allowances need to be made for higher speeds of light vehicles and possible higher speeds of descending heavy vehicles.

- Limit the height of cut to 15 metres in low hazard areas and to 8 metres in high hazard areas, except for unavoidable situations (see Figures 24.1 to 24.3).

- Avoid excessive cutting in steep areas by adopting fill section and providing retaining walls.

- Avoid retaining walls wherever full cut section is possible within the above-mentioned cut height limits.

- Avoid retaining walls of more than 8 metres in height as far as possible.

- Adopt inside-battered retaining walls in fill sections on hill slopes of more than 20 degrees.

- Minimize cut height and retaining wall height by providing breast walls for cut slopes exceeding the limits mentioned above.

- Design stable cut slope or height from the standard tables and charts discussed in Section 24.4.

- Check the slope stability for high hazard areas and for cut slopes exceeding the above limits. (See Chapter 13).

- Provide erosion control measures for unprotected slopes.

- Provide for stabilization of existing or predicted landslides on the uphill and downhill slopes influencing the road.

- Adopt trial and error shifting of alignment, vertically and horizontally, until the cross-section closely conforms to the requirements of geometric standards and the criteria mentioned above.

- Identify areas where it is not possible to maintain the above-mentioned cut height limits. Determine intensive investigation needs and requirements of the specialist to deal with such areas.

- Design out-sloped road formation for low speed (< 30 km/hr) road in sections with longitudinal gradients of less than 5 per cent in rocky areas. Excavation should be considerably minimized by cut sloping. See Figure 24.4

24.4 CUT SLOPE STABILITY AND DESIGN

24.4.1 *Problems*

Cut slope designs are normally based upon geotechnical parameters such as soil and rock properties, terrain, slope, and water tables under static conditions. However, dynamic processes such as land use, rainfall, and runoff conditions in the cut slope and watershed, above and below the road, influence the overall stability of the cut slopes during the life of the road. The instabilities caused by the following activities need to be properly considered while designing slopes.

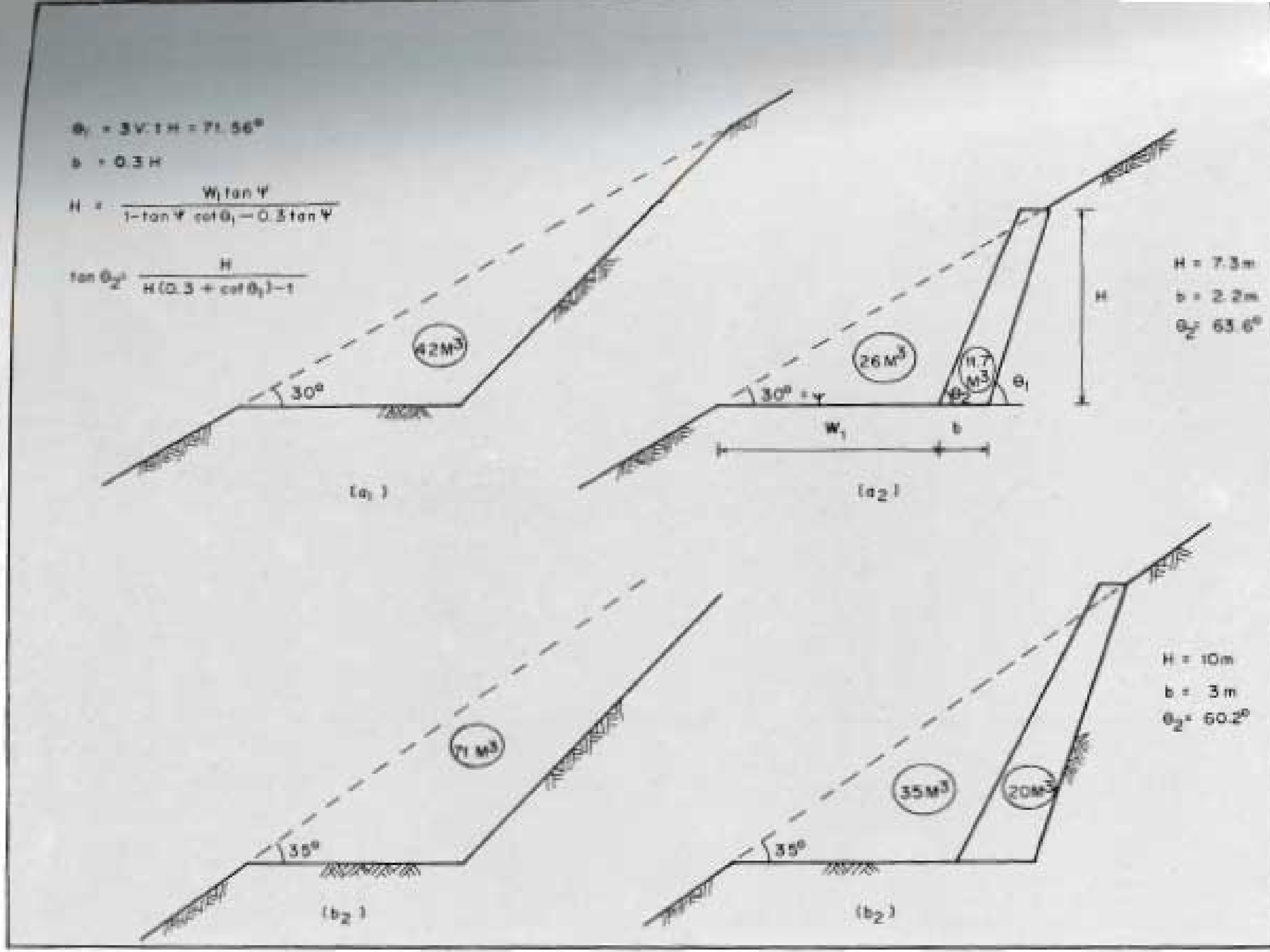


Fig. 24.1 Minimizing height of cut in flatter slopes in soils

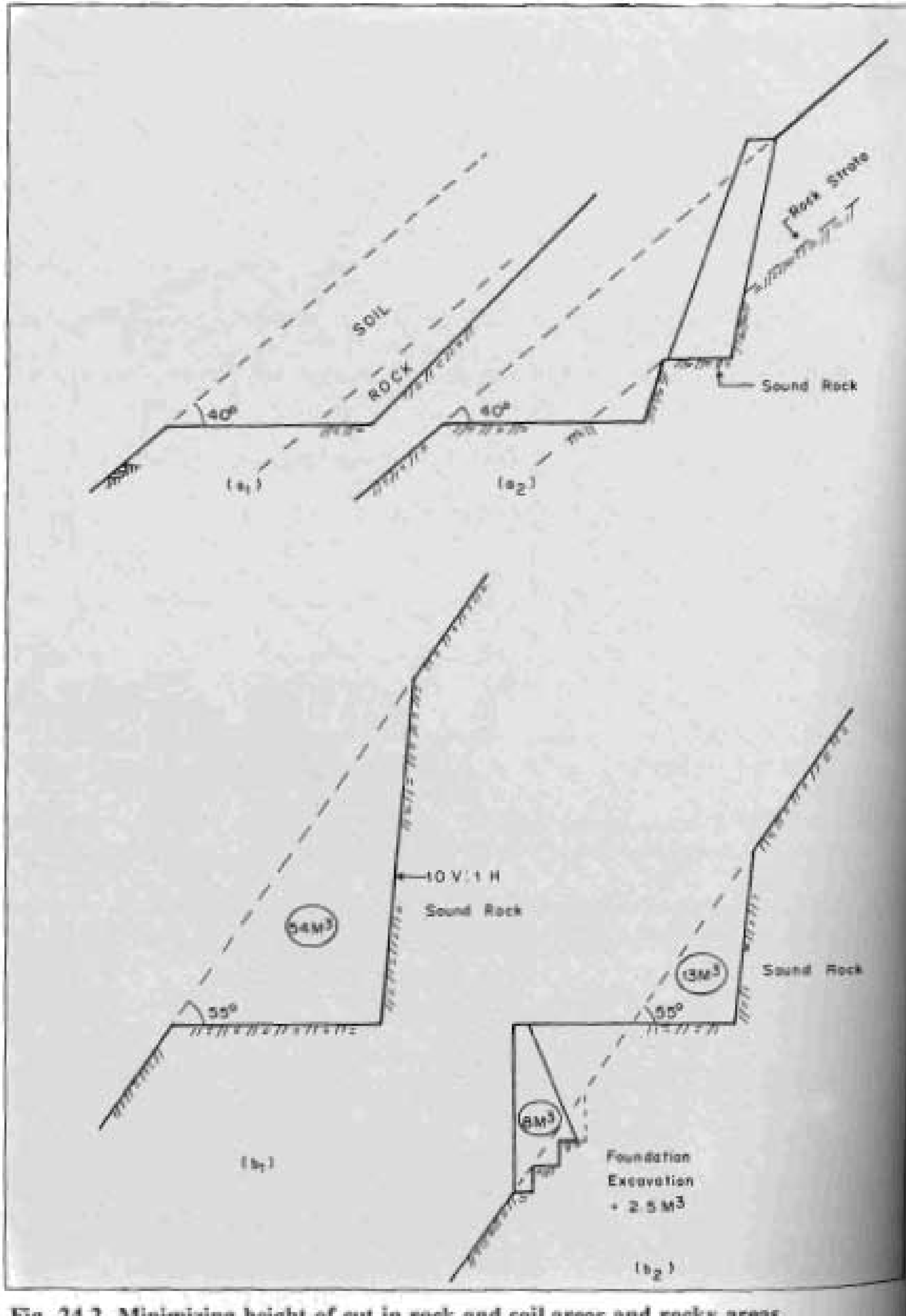


Fig. 24.2 Minimizing height of cut in rock and soil areas and rocky areas

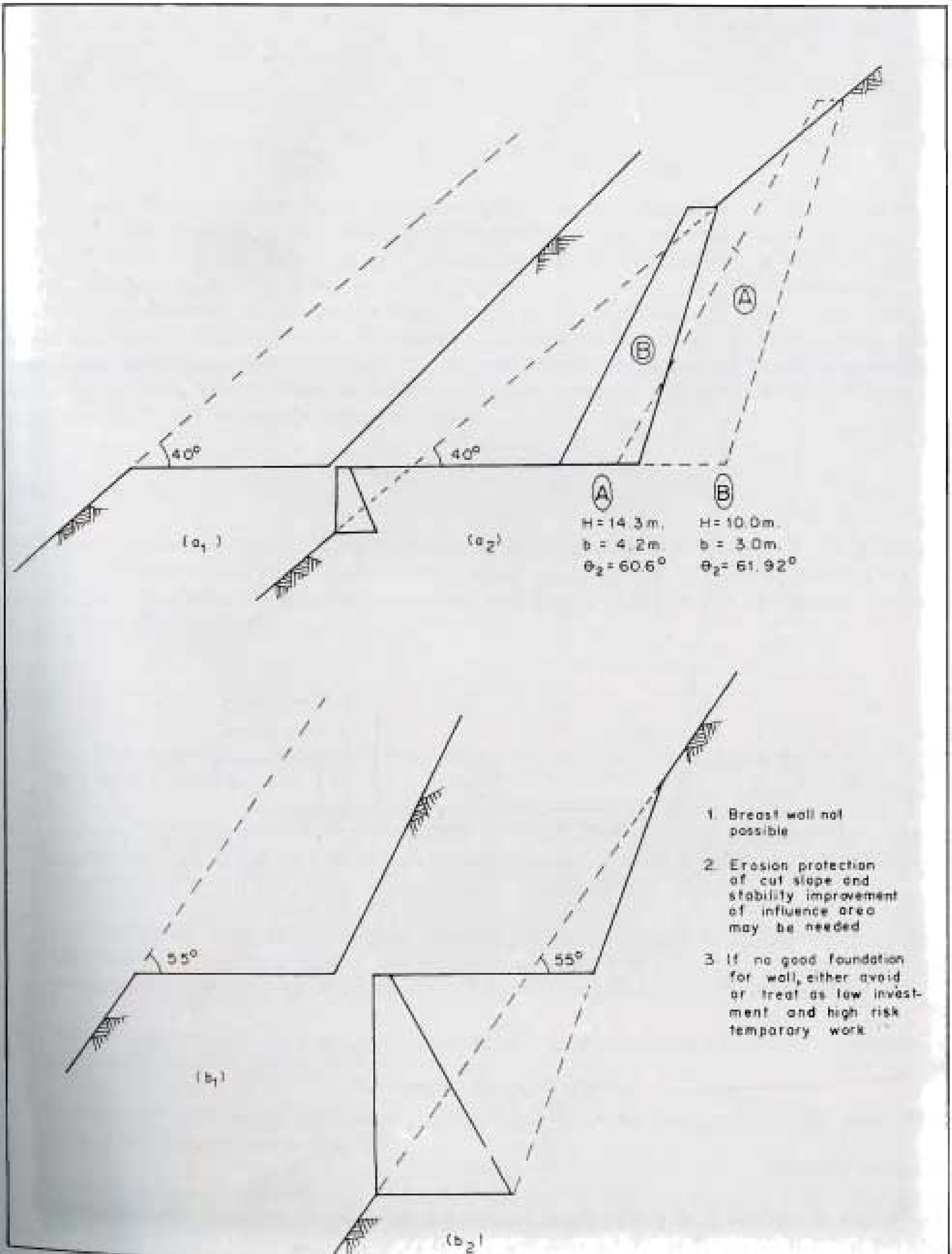


Fig. 24.3 **Minimizing height of cut in steeper slopes in soils**
Requires vertical horizontal shifting of centre line of road.

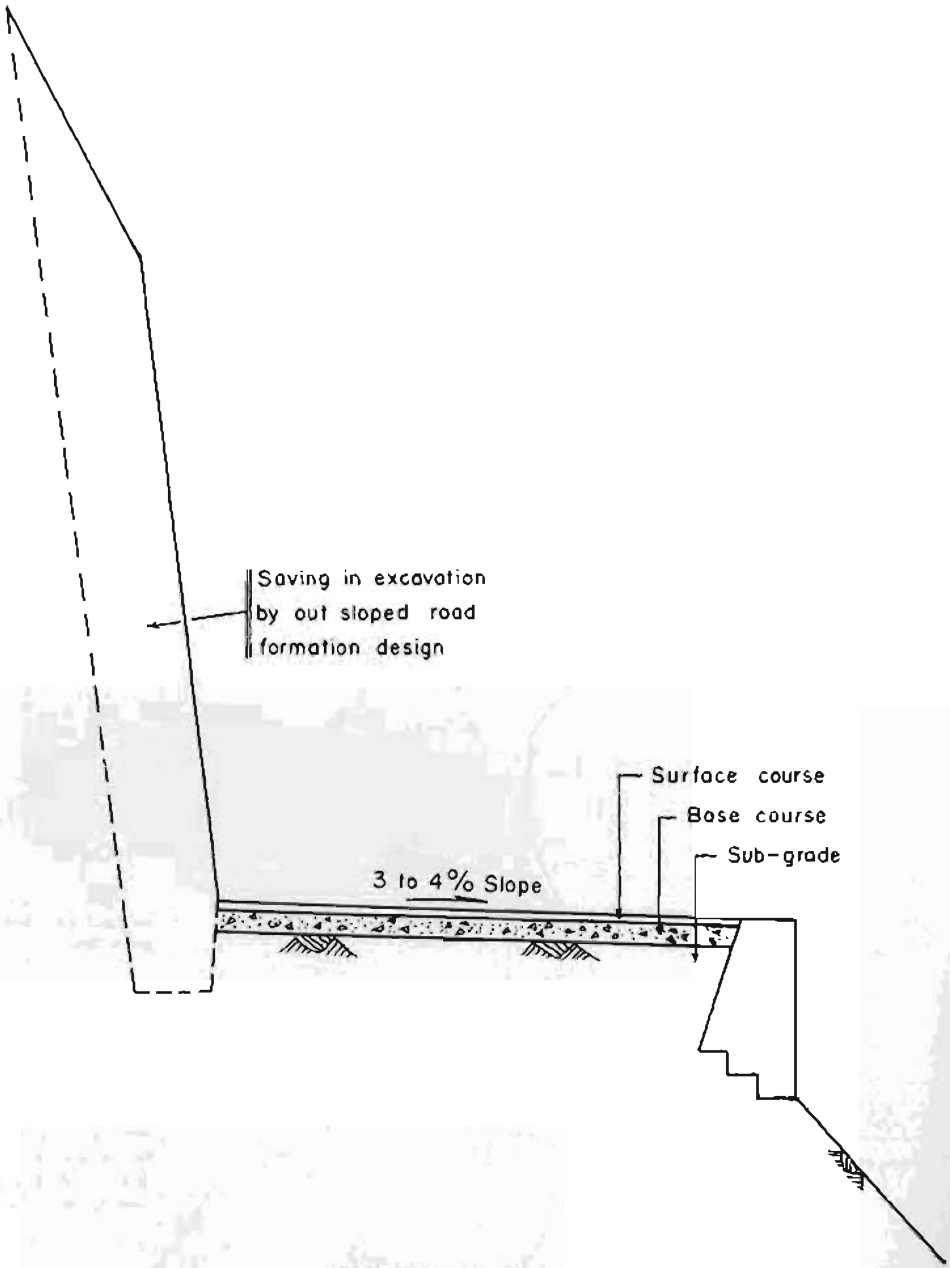


Fig 24.4 Outsloped road

Cut Slope Activities

The failure in this case is primarily the result of cut slopes. Traditionally cut slopes are arbitrarily selected at 1/3 H:IV to 1/2 H: IV for soils and 1/15 H:IV to H:IV for rocks and are steeper than the minimum requirement for equilibrium, i.e., a factor of safety greater than 1.0 for the worst season.

These slopes, called **non-equilibrium** or **marginal equilibrium** slopes hereafter, are bound to fail during or after rains or after undergoing climatic or other conditions (that are worse than at the time of the cutting of the slopes), until a stable angle is achieved and self generation of both plants and vegetation occurs. The total mass wasting due to possible progression of failure longitudinally and laterally, as a result of hill disturbance from cut slopes aided by rainfall and hydrogeologic conditions, could be massive before the natural equilibrium state is achieved. Even slopes initially cut to stable angles, called **equilibrium cut slopes** hereafter, are likely to involve considerable mass wasting, because of progressive destabilization from different forms of erosion on the barren slopes, until self-generation of plants and vegetation occurs and natural equilibrium is reached.

Influence Area Activities

Traditionally, activities on road designs are limited to the strip bounded by the edges of cut slopes and fill slopes. The stability of these slopes is affected by natural and human activities in the influence area up and down the slope. **Slope enrichment** in the watershed area influencing the road is, therefore, necessary for long-term stability.

Guidelines

- Cut slope angles for cut heights of up to 15m in low hazard areas should be selected from Tables 24.1 to 24.5 and Figures 24.5 to 24.8.
- Cut slope angles for medium hazard areas and for heights 16m to 30m in low hazard areas should be selected from Tables 24.1 to 24.5 and from Figures 24.5 to 24.8 and from chart solutions in Chapter 13.
- For high hazard areas and cut heights exceeding 30m in low hazard areas, determine cut slope analysis after special investigations and slope stability analysis by a specialist. Manual method and package computer programmes described in Chapter 13 may be used for stability analysis.
- Reduce cut height to 15m for low hazard areas and 8m for medium hazard areas by providing breast walls and steeper cut slopes.
- Provide vegetative erosion control measures (see Section 24.5) for cut slopes of heights greater than 15m in low hazard areas in soils.
- Provide engineering and biological erosion controls (see Section 24.5) in the cut slopes and beyond the cut slope in the case of road cuts in medium hazard areas.
- Provide landslide and erosion controls in the cut slope and beyond the cut slope for road sections in the high hazard areas.
- Benched cut slopes in soft rock allow vegetation growth and reduce erosion (see Figure 24.1).

Table 24.1 Final design of cut slopes

Soil types/deposits	Cut slope angle, tables and figures to use for determining slope height/angle	Problems requiring special investigation and specialist service
Homogeneous soils (soils that do not exhibit layering or stratification of various materials)	Table 24.2, Figures 24.5 to 24.8	Loose saturated sand, silt and soft clays. Clays underlain by seams of fine water bearing sands.
Stratified deposits (horizontal or dipping layers of various materials underlain by bedrock)	Combination of Table 24.2 and Figures 24.5 to 24.8. Use total height from top of cut to the bottom of each layer to determine the slope of each layer.	Sand and silt layers are sources of water resulting in seepage on the face of cut. Frost action, washout, and hydrostatic pressure from trapped water are likely. Low strength in soft and fissured clay layers. Layers dipping at more than 20° towards the cut cause special problems.
Residual soils (formed by rock weathering in place retaining much of the structural and bedding planes of the parent rock at depth), composed of 3 layers Layer 1: Residual soil Layer 2: Weathered rock Layer 3: Unweathered rock (Residual soils are often stable with slopes steeper than transported soil and thus can be as steep as 45° to 80° in exceptional cases. Also see Figure 24.9 and notes to it)	Layer 1 - use Figures 24.5 to 24.8 Layer 2 - use soil No. 1 of figures 24.5 and 24.6 if the material is dense with no complete joint system. Use soil no 2 or 3 for all other cases. Layer 3 - use Table 24.5 Measure height of cut from top of excavation for any layer and not just the thickness of the individual layer. Account for high water table or poor internal drainage in coarse-grained soils using Figure 24.6.	Dips of bedding towards cut exceeding residual angle of sharing resistance which is 20 to 30 for weathered rock and 30 to 45 for original unweathered rock. Use stereoplot and wedge systems usually to determine stable cut slope in jointed rock masses.
Cemented and special soils (very high degree of interlocking)	Often stands near vertical (1/4:1) Protect from rain or surface runoff since water dissolves or softens the cementing material. Use local experience	Not to be confused with partially saturated sands and silts.
Unweathered rock	Use Table 24.5 for dips towards road less than 30° and for dip steeper cut than the cut slope.	Dips greater than 30° towards the cut.
Rocks with joint systems	Field measurements of strikes and dips of bedding and joints, tension cracks, infilling, waviness, water table etc necessary.	Rock slope stability analysis by computer methods or graphical methods, such as pole nets and stereoplot, by a specialist is required.

Table 24.2 Stable slope angle for sands and gravel with non plastic fines

Soil No.	Description	Maximum Slope Ratio (H:V)			
		Low groundwater (below bottom of excavation)		High groundwater 1/ (seepage from entire slope)	
		dense 2/	loose 2/	loose 2/	dense 3/
1.	Sandy gravel (GW, GP0	0.85:1	1.5:1	3:1	1.3:1
2.	Sand, angular grains well-graded (SW)	1:1	1.6:1	3.2:1	2:1
3.	Silty gravel (GM); univorm sands (SP); and silty sand (SM)	1.5:1	2:1	4:1	3:1

Source: Compendium 13. Slopes: Analysis & Stabilization, Transportation Research Board (TRB), 1980

1/ Based on material of saturated density approximately 125 pcf. Flatter slopes should be used for lower density and steeper slopes can be used for higher density material. For every 5 per cent change in density, change the ratio by approximately 5 per cent.

2/ Approximately 85 per cent of maximum density relative to AASHO T 99.

3/ Approximately 100 per cent of maximum density relative to AASHO T 99.

Soil No.1,
Loose to Dense - 25 to 60 blows per foot of Standard Penetration Test.

Soil No.2,
Loose to Dense - 20 to 50 blows per foot of Standard Penetration Test.

Soil No.3,
Loose to Dense - 5 to 25 blows per foot of Standard Penetration Test.

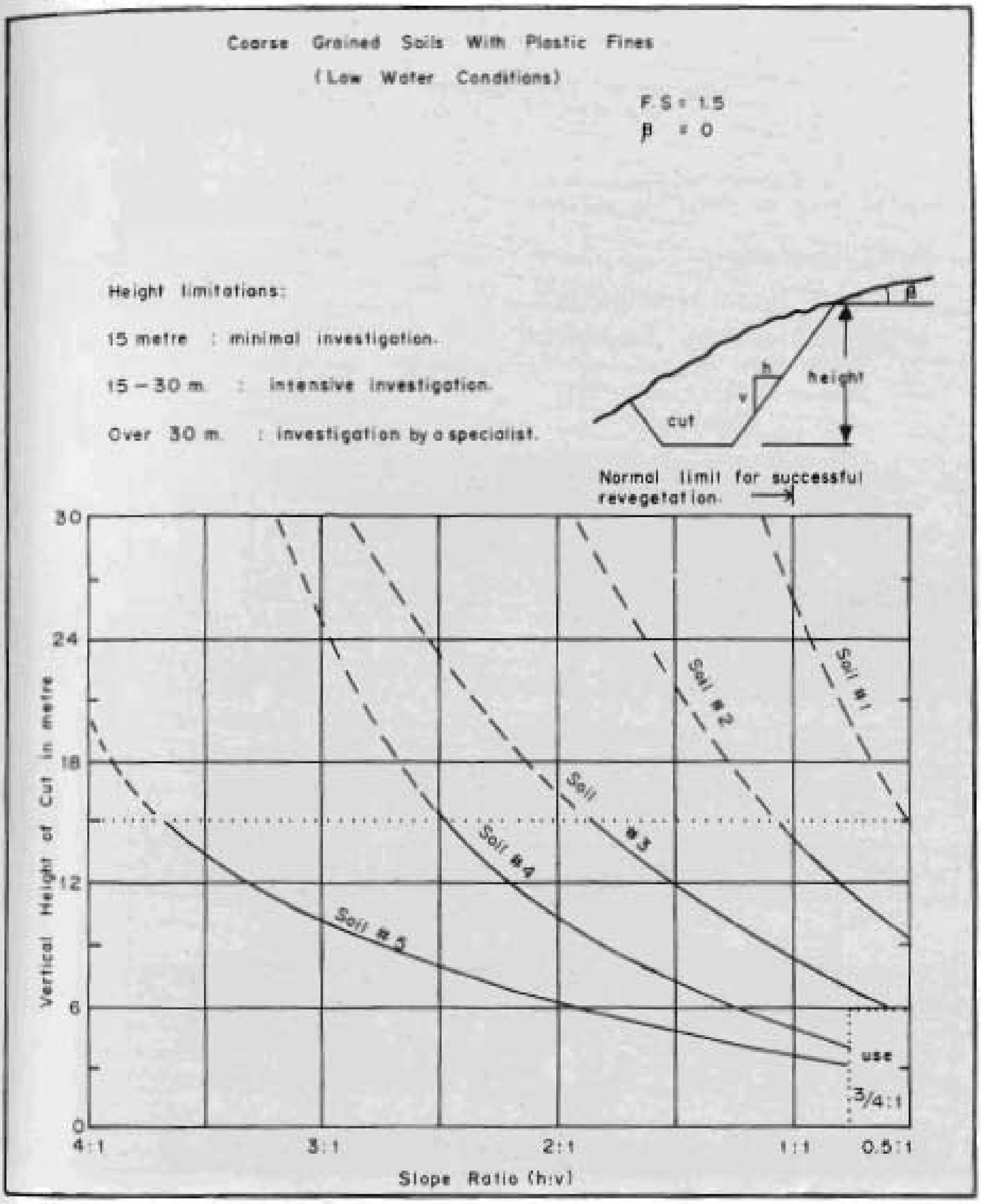
Table 24.3 : Soil number and description for use with Figures 24.5 and 24.6

1. Well-graded material with angular particles. Extremely dense and compact (in excess of 100 per cent of AASHO T 99 relative compaction) with fines that cannot be moulded by hand when moist. Difficult if not impossible to dig with a shovel. May need to be ripped during construction. Standard penetration test blow count greater than 40.
2. Poorly-graded material with rounded or low percentage of granular particles. Dense and compact (approximately 100 per cent of AASHO T 99 relative compaction) with fines being difficult to mould by hand when moist. Difficult to dig with a shovel. Standard penetration test approximately 30 blows per foot.
3. Fairly well-graded material with sub-angular granular particles. Intermediate density and compactness (approximately 90 to 95 per cent of AASHO T 99 relative compaction) with fines that can easily be moulded by hand when moist (PI greater than 10). Easy to dig with a shovel. Standard penetration test blow count approximately 20 blows per foot.
4. Well-graded material with angular granular particles. Loose to intermediate density (approximately 80 to 90 per cent of AASHO T 99 relative compaction). Low plasticity fines (PI less than 10). Extremely easy to dig. Standard penetration test blow count less than 10 blows per foot.
5. Poorly-graded material with rounded or low percentage (50-60 per cent) of granular particles. Loose density (less than 85 per cent of AASHO T 99 relative compaction). Low plasticity fines (PI less than 10). Extremely easy to dig even with the hand. Standard penetration test blow count below 5 blows per foot.

Notes:

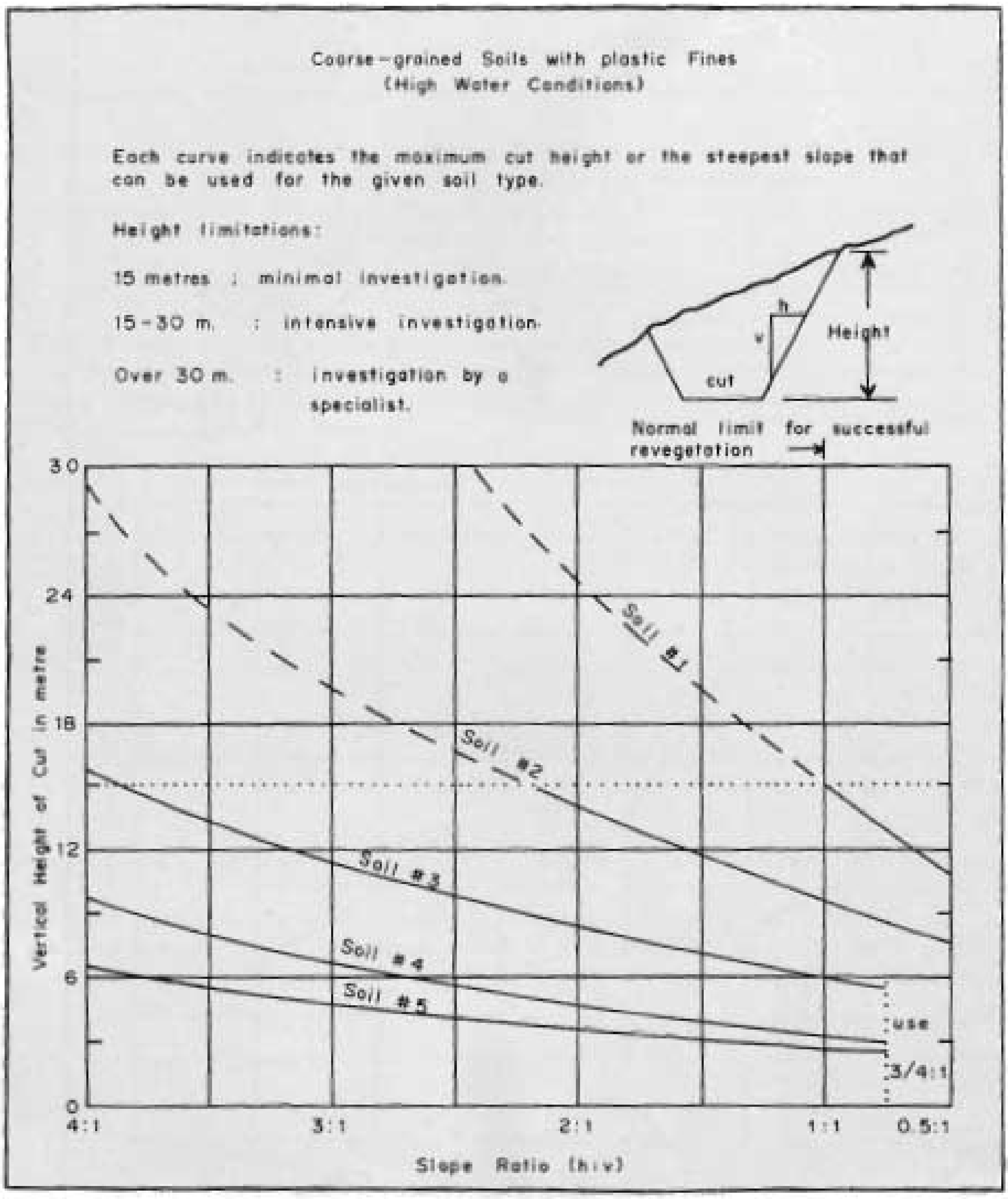
1. Based on material with a moist density of 125 pcf. Flatter slopes should be used with heavier soils. For every 5 per cent increase in density reduce the slope ratio or height approximately 10 per cent.

2. Large gravel and boulders will give misleading results concerning the ease of digging and standard penetration test results.



Source: TRB 1980

Fig. 24.5 Maximum unsupported height of steepest slope of cuts



Source : TRB 1980

Figure 24.6 Maximum unsupported height of steepest slope of cuts

Table 24.4 Soil number and description for use with Figures 24.7 and 24.8

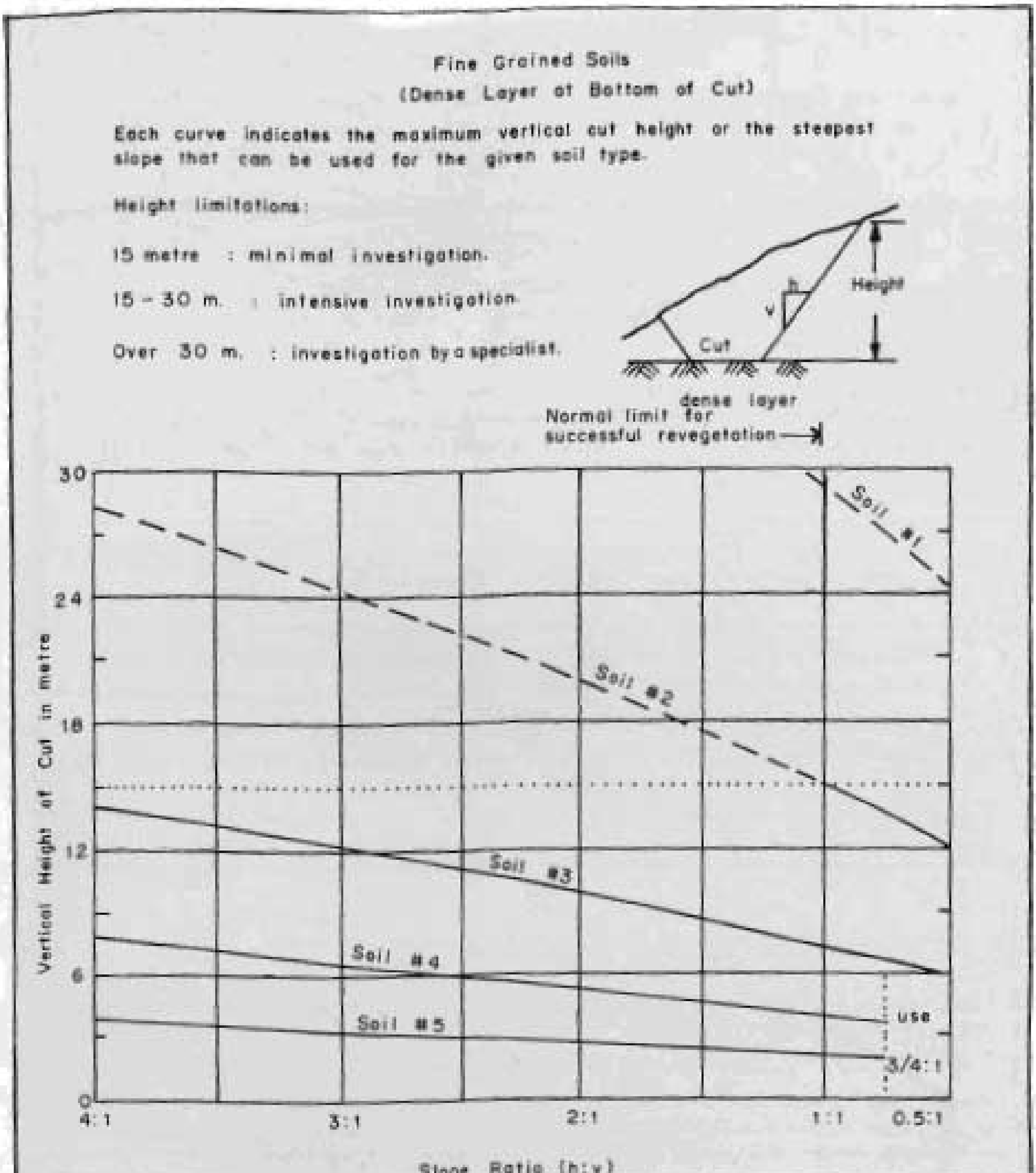
Soil No.	Description
1	Very stiff consistency. The soil can be dented only slightly by finger pressure. Ripping may be necessary during construction. Standard penetration test blow count greater than 25.
2	Stiff consistency. The soil can be dented by strong pressure of fingers. Might be removed by spading. Standard penetration test blow count approximately 20 blows per foot.
3	Firm consistency. The soil can be moulded by strong pressure of fingers. Standard penetration test blow count approximately 10 blows per foot.
4	Soft consistency. The soil can easily be moulded by fingers. Standard penetration test blow count approximately 5 blows per foot.
5	Very soft consistency. The soil squeezes between fingers when fist is closed. Standard penetration test blow count less than 2 blows per foot.

Notes:

1. For ML soils with PI 3 or less, use slopes as defined by soil # 3 Table 24.2.

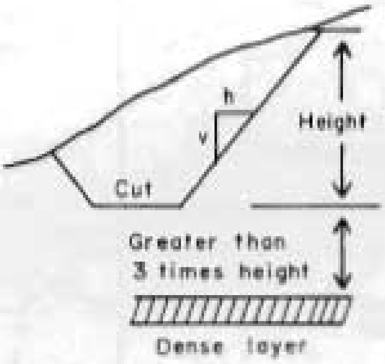
2. Figure 24 assumes a hard layer at the bottom of the proposed cut and is based on material with a moist density of 125 pcf, flatter slopes should be used with heavier soils. For every 5 per cent change in density, change the slope ratio or height approximately 5 per cent.

3. Moisture should not be adjusted when checking consistency. An undisturbed sample should be used taken at a depth great enough to represent constant moisture content throughout the various seasons.



Source: TRB 1980

Figure 24.7 Maximum unsupported height of steepest slope of cuts



Fine-grained soils (dense layers at great depth)

Soil Type	Maximum Height 1/ in feet	Slope ratio (h:v)
1	80	0.5:1
2 2/	40	0.5:1
3 2/	20	0.5:1
4 3/	10	1:1
5 3/	5	1:1

Notes:

1/ If it is necessary to exceed this height consult with a geotechnical engineer. Benching will not improve stability as these slopes are practically independent of the slope ratio.

2/ If the slope of the natural ground exceeds 20°, then the natural slope may be unstable. A detailed field investigation is necessary to check this condition prior to any design or construction.

3/ If the slope of the natural ground exceeds 10°, then the natural slope may be unstable. A detailed field investigation is necessary to check this condition prior to any design or construction.

Source: TRB 1980

Fig. 24.8 Maximum unsupported height of steepest slope of cuts

Table 24.5 Average slope values for bedrock excavation for planes of weakness dipping less than 30 degrees

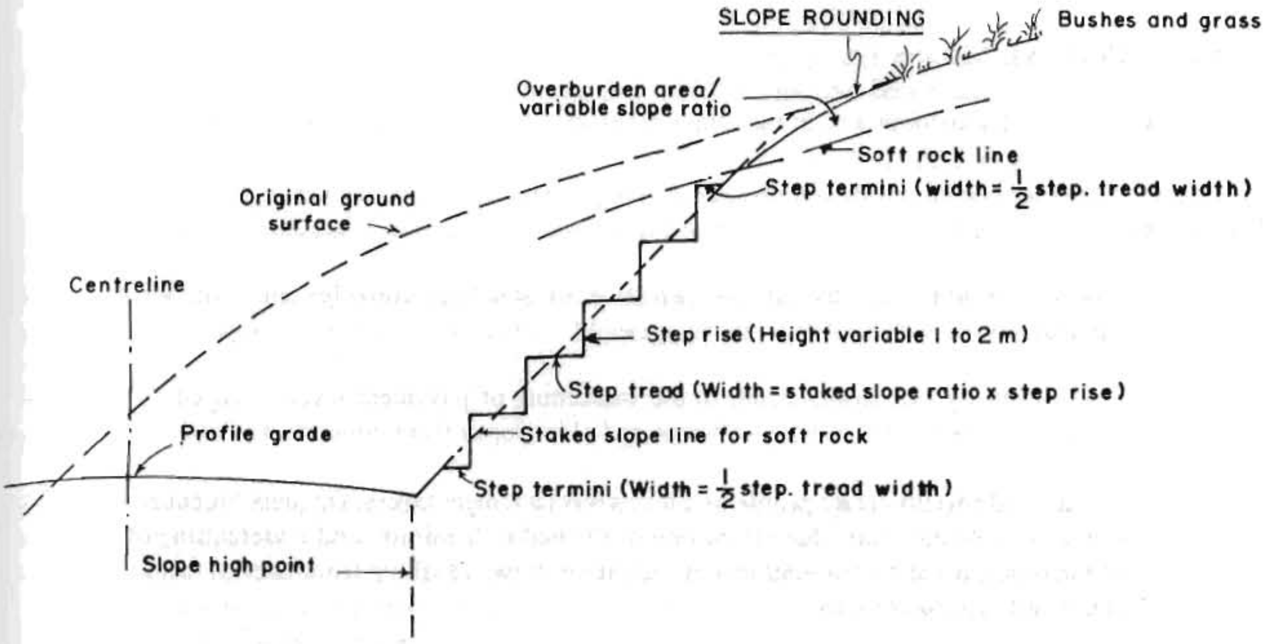
	Description	Maximum Slope Range (H:V)
1	**Igneous**	
	Granite, trap, basalt, and volcanic tuff	1/4:1 to 1/2:1
2	**Sedimentary**	
	Massive sandstone and limestone	1/4:1 to 1/2:1
	Interbedded sandstone, shale, and limestone	1/2:1 to 3/4:1
	Massive claystone silt stone	3/4:1 to 1:1
3	**Metamorphic**	
	Gneiss, schist, and marble	1/4:1 to 1/2:1
	Slate	1/2:1 to 3/4:1
	Serpentine	Special investigation

Notes:

1. Variable cut slope angles are recommended for stratified sedimentary deposits where there are layers of different weathering properties.

2. Steeper cut slope angle may be used along concave road and flatter angle along convex road portions respectively.

3. Vertical cuts may be used in case of erodible rocks such as conglomerate, soft shale/sand rock, etc.



Source: Schuster and Krizek 1978

Fig. 24.9 Idealized cross-section showing stepped cut slope design in soft rock

Notes:

* Some material such as decomposed granite will ravel continuously due to weathering, filling ditch lines. Flattening the slope usually does not control this problem unless vegetation can be established. A successful solution is to step the slope in small benches, say two feet wide and with height determined by the slope. The material will then ravel, filling the benches and at the same time protecting the underlying material, aiding plant growth to start and preventing further ravelling. This procedure can be applied to many types of material.

In badly jointed or weathered rock some means of removing the hazard must be provided. This can be done by:

- o rock bolting,
- o grouting of fissures,
- o horizontal berms or a wide ditch to catch falling rock,
- o fences, wire mesh, or walls to catch falling rock, and
- o pneumatically applied mortar to prevent ravelling.

In addition, the prevention of water pressure building in seems must be prevented by the proper drainage or sealing of the entrances. Any design must start with a complete picture or the joints, fissures, and bedding planes of the area of concern.

24.5 DRAINAGE

Refer to Figures 24.10 to 24.12 for illustrations of different drainage constructions.

24.5.1 *Problems*

The problems generally encountered in mountain roads in developing countries are outlined below.

Side drains

- Side drains are sized by *ad hoc* selection of standard drawings and without hydraulic calculations.

- Under-capacity side drains result in the weakening of pavement layers, caused by increased moisture content, and erosion of berms and side slopes from concentrated runoffs.

- Unlined side drains create problems such as wet pavement layers, frequent blockage by failure of the sides of the drain, deep trenching of the bed of the drain, and undercutting of the sides of the road caused by non-uniform and unsteady flows resulting from uneven scour, friction, and runoff concentrations.

- Long side drains (in excess of 100m) in high rainfall areas create massive damages to the road from the interruption to flow by debris and landslide materials.

- Lack of protection on the downhill slope below the outlet of the side drains at the switch-backs enhance gullying and massive failures where the material is colluvium, highly weathered rock, and other loose material.

- Side drains alone are not enough to keep the sub-grade of the pavement in dry condition (or at low moisture content), and to keep the slope stable in areas where the slope is wet and the sub-surface water table is above the road.

Culverts

Refer to Figures 24.13 to 24.15.

- Culverts are also not normally designed from hydraulic considerations. Cost effectiveness and hydraulic efficiency of these culverts are rarely possible.

- Transverse relief culverts, i.e., pipe or box or other culvert solely aimed at discharging the runoff from the gutter (side drain) are generally spaced at 150 to 200 metres despite the area and conditions of watershed.

- Many stream beds are on slopes which are marginally stable in colluvial soils and are prone to massive slope failure from low frequency floods, i.e., 25 year or more return period floods. Therefore, culverts designed to carry a 10 year or less return period without outlet protection, when discharging greater floods, erode the downhill slope heavily and also damage the foundations of the road structure.

- Undue slope instability is caused by the scour from the pipe culverts laid at modified bottom locations (Figure 24.13b), i.e., the gradient of the pipe the less than the gradient of the stream.

- Location of transverse relief culverts (Figure 24.13c) on erodible soils and loose soils creates severe slope failure if not accompanied by adequate erosion protection and slope lining.

- Absence of debris risers, debris deflectors, and debris racks (see Figures 24.16 to 24.18) create major failures of roads due to blockage of culverts by debris and large boulders.

Landslides

- Absence of sub-surface drainage has resulted in a perpetual cycle of landsliding and clearance. Many shallow slides can be stabilised by draining the landslide mass by using sub-surface drains (variously named as counterfort drains, trench drains, French drains, and Y - drains by various agencies) and providing a low height toe support and, sometimes, unloading the top of the landslide mass.

- Deep horizontal drains, though generally effective in the stabilization of deep-seated slides are not applied because of lack of drilling equipment and skills.

- Engineers are not yet well adapted to the use of vegetative measures for minimising large-scale failures in the long run due to progressive development of soil erosion and gully erosion.

Gully Control

- Absence of gully controls such as check dams and cascades, in the upstream and downstream of streams or gullies from the road, results in drainage diversions and considerable damage to the hill slope and the road itself.

24.5.2 *Guidelines*

Chapter 19 provides background on theoretical and empirical designs of drainage. Figures 24.11 to 24.28 in this chapter provide examples of typical designs of drains, gully controls, minor culverts, and low-cost drainage crossings. In the next section are some guidelines on the design process of drainage.

Side Drains

- o Design standard side drains by estimating 10 year return periods for the sections of a watershed divided into several homogenous sections. Special measures should be taken to protect the wet areas and debris depositional areas..

- o In critical areas that are susceptible to considerable additional runoffs from drainage diversions from the land use on the uphill side, drains should be designed to carry additional flows of at least 25 year return period floods.

- o Outsloped roads may be considered for paved sections of low speed road with longitudinal grades of up to 5 per cent.

- o Hydraulic design of side drains is carried out with the help of the Manning Equation and checked for non-eroding velocities (see Chapter 19).

- o Side drains should be built so that they are not only adequate in capacity to carry surface runoff but also strong enough to withstand the loads exerted by vehicles crossing each other and travelling very near the drain and deep enough to allow draining of pavement underlayers.

- o Side drains should be generally lined to prevent seepage into the pavement underlayers and scouring of the drain channel, unless sound rock is encountered along the drain invert. Recesses must be provided in the lining to allow the draining of water from behind the lining.

- o Drystone lining is not preferable, although the initial cost is low, since it allows seepage into the road pavement structure and needs intensive maintenance.

- o Side drain outlets should be designed against any possible scouring by the falling water.

Pavement Drainage

- o Provide sub-surface drain parallel to and below the surface-side drain in wet areas (see Figure 24.12).

Culverts

- o Design minor culverts for a 10 year return period flood.

- o Provide transverse relief culverts at a spacing not exceeding 100 metres in long grades and high runoff areas.

o Design major culverts for a 25 year return period flood.

o All protection works below the side drains and culverts in the switchback areas should be designed for at least a 25 year return period flood.

Debris Control

o Provide grating, trash rock, debris deflectors, and debris risers, as appropriate in the inlets of culverts. See Figures 24.16 to 24.18.

o Provide scour and erosion protection below the outlets of culverts and side drains in switchback areas.

Gully Control

o Provide check dams and cascades, as appropriate, in the upstream and downstream of the streams that are susceptible to flow diversions and accelerated erosion. See Figures 24.19 to 24.24.

Landslide Drainage

o Stabilize shallow landslides in wet areas by sub-surface drains. See Figures 24.25 to 24.26.

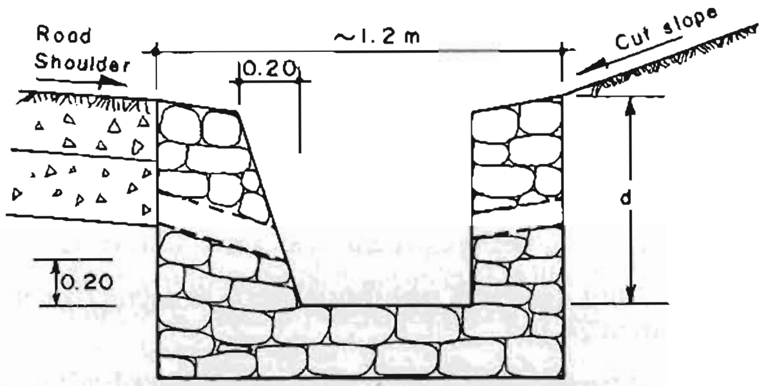
o Use horizontal or vertical drains to stabilize deep-seated landslides by pore-water pressure reductions in critical areas of important roads. Refer to specialized texts for the design of horizontal drains. See Figures 24.27 and 24.28.

Biotechnical stabilization

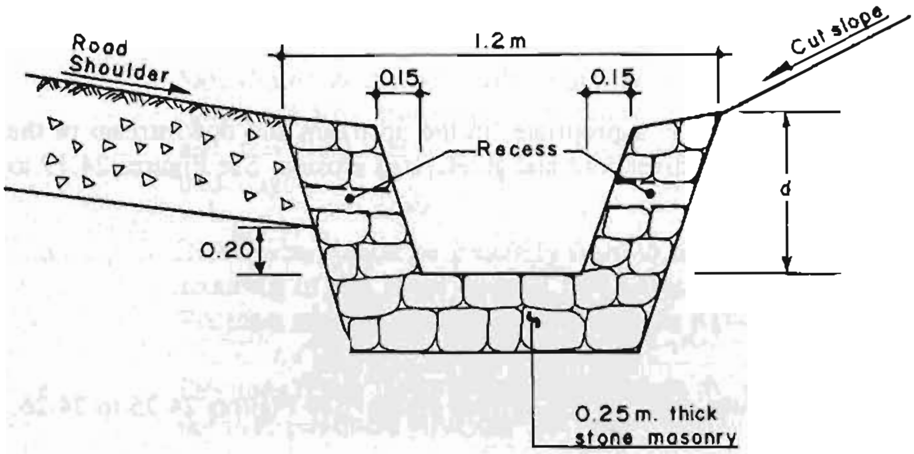
o Plant the cut slope and the slope beyond the cut slope in highly erodible areas with fast growing, deep rooting, and dense cover types of grass adaptable to local situations.

o Plant appropriate species on all stabilized landslide surfaces or slopes.

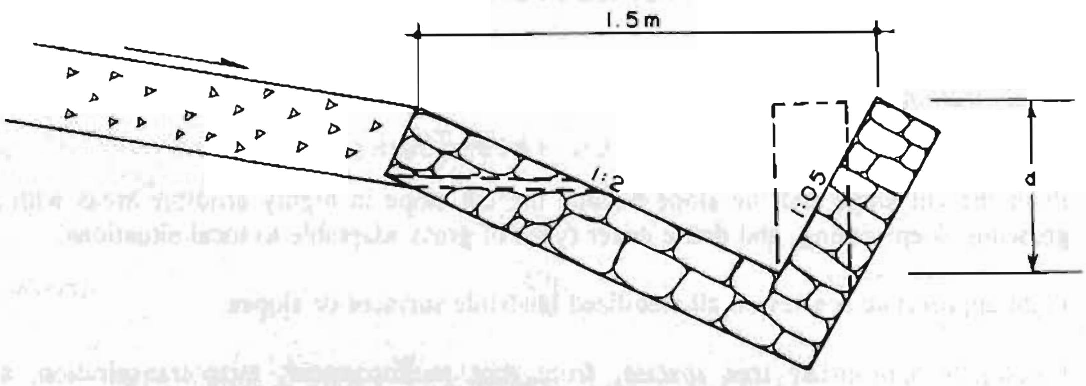
o Investigate appropriate tree species, from root reinforcement, evapotranspiration, stem infiltration, and local acceptability considerations, suitable to stability enhancement of the area around the road and carry out plantation programmes (refer to Chapter 16).



(a) Rectangular drain to withstand traffic surcharge adjacent to drain

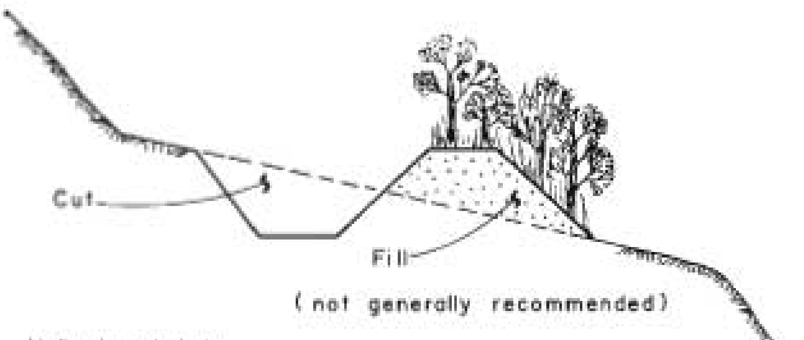


(b) Trapezoidal stone masonry drain (for no traffic surcharge load condition)

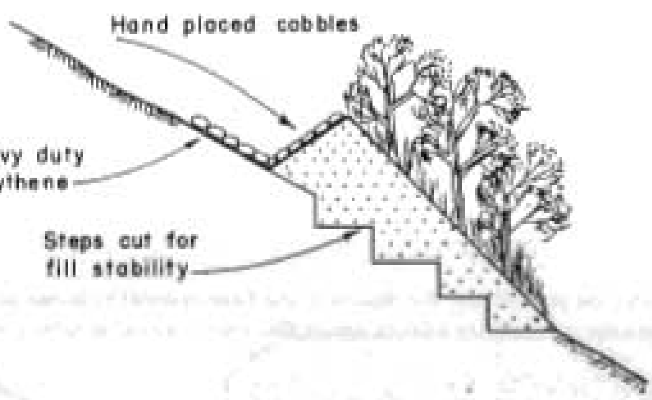


(c) V - shaped drain
The hillside wall could be made vertical if space is limited (shown by broken line)

Fig. 24.10 **Side drains of stone masonry in cement mortar (d = 50-60 cm or as required by pavement drainage conditions)**



(a) Unlined catch drain
(not generally recommended)



(b) Catch drain lined with heavy duty polythene

(c) Catch ditch lined with stone cement masonry in cement mortar

Notes

Catch drains are used along the crown of cut slopes to prevent water flowing from overlying areas over freshly constructed erodible slopes. However, the performance of catch drains has not been encouraging because of local failures and blockages aggravating the problem and difficulties of access and maintenance.

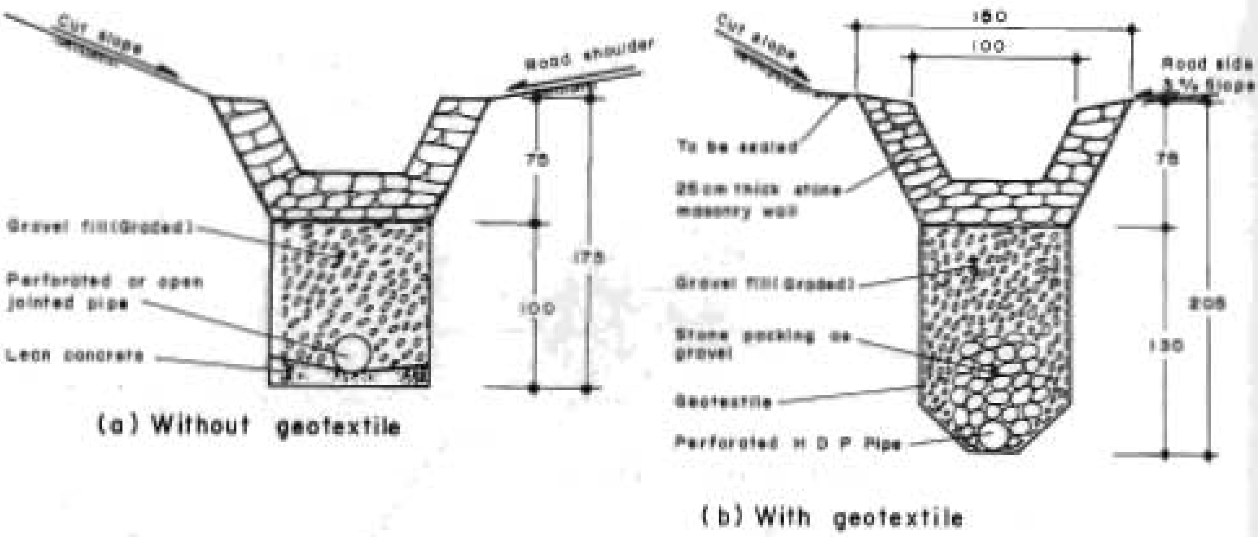


Fig. 24.12 **Sub-surface drains (for drainage of pavement and to lower the water table in wet areas)**

Notes

Filter Material Design

The U.S. corps of engineers have recommended the particle size distribution of the filter material be based on the piping ratio to prevent silt being washed into the drain and the permeability ratio to ensure that filter material is more permeable than the surrounding ground.

$$\textit{Piping ratio} = \left(\frac{d_{15}\ \textit{filter}}{d_{85}\ \textit{subgrade}}\right) \leq 5$$

$$\textit{Permeability ratio} = \left(\frac{d_{15}\ \textit{filter}}{d_{15}\ \textit{subgrade}}\right) > 5$$

d_{15}, d_{85} being the size of the sieve passing 15 per cent or 85 per cent of the material through it.

The d_{85} filter should be greater than twice the size of the opening in a perforated pipe or open-jointed pipes.

If the naturally occurring material is not suitable because of the particle size distribution of the sub-grade soil and the size of opening in the pipes, then two filter materials are used with coarser material surrounding the pipe.

Alternatively, filter material grading, as recommended for various situations by the Indian Road Congress, can be used. However, geoxtextile filter fabrics are available which separate filter and surrounding material and allow only water into the drain.

Grading requirements of filter material
(as recommended by the Indian Road Congress)

Sieve designation	Per cent by weight passing the sieve		
	Class I	Class II	Class III
50mm	--	--	100
40mm	--	--	95-100
25mm	--	100	--
20mm	--	90-100	50-100
10mm	100	40-100	15-55
4.75mm	90-100	25-40	0-25
2.36mm	80-100	18-33	0-5
1.18mm	50-95	--	--
600 micron	30-75	5-15	--
300 micron	10.30	0-7	--
150 micron	1-10	--	--
75 micron	0-3	0-3	0-3

Notes

(i) Where the soil in the trench is of a fine-grained type (e.g., silt, clay or a mixture there of), the backfill material should conform to Class I grading.
(ii) Where the soil in the trench is of coarse silt to medium sand grading.
(iii) Where the soil in the trench is gravelly sand, the backfill material should correspond to Class III grading.
(iv) The thickness of backfill material around the pipe should be at least 150mm all around in all cases.

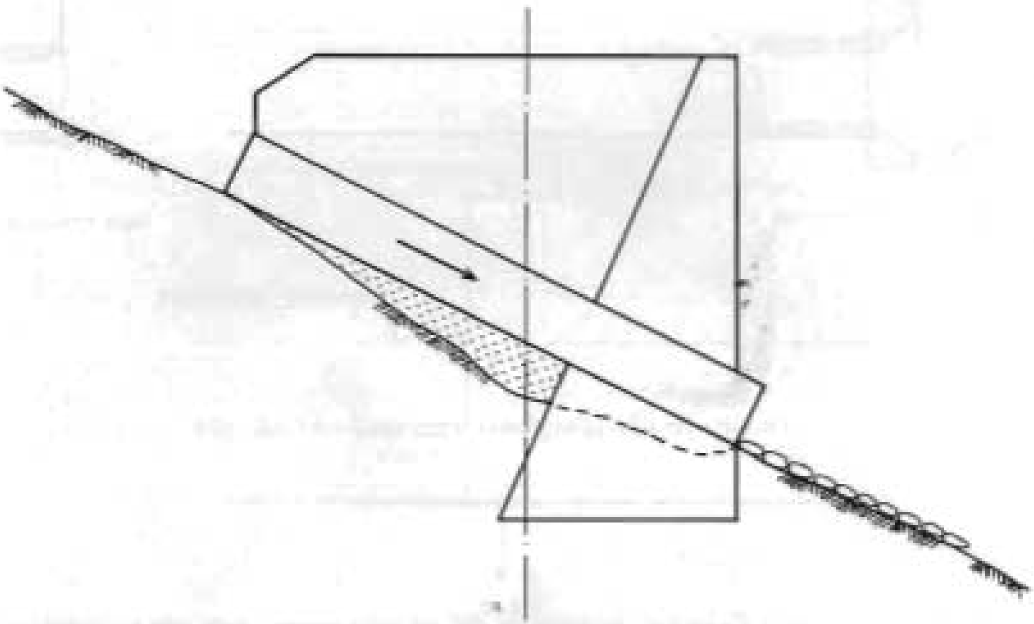


Fig. 24.13(a) Bottom location

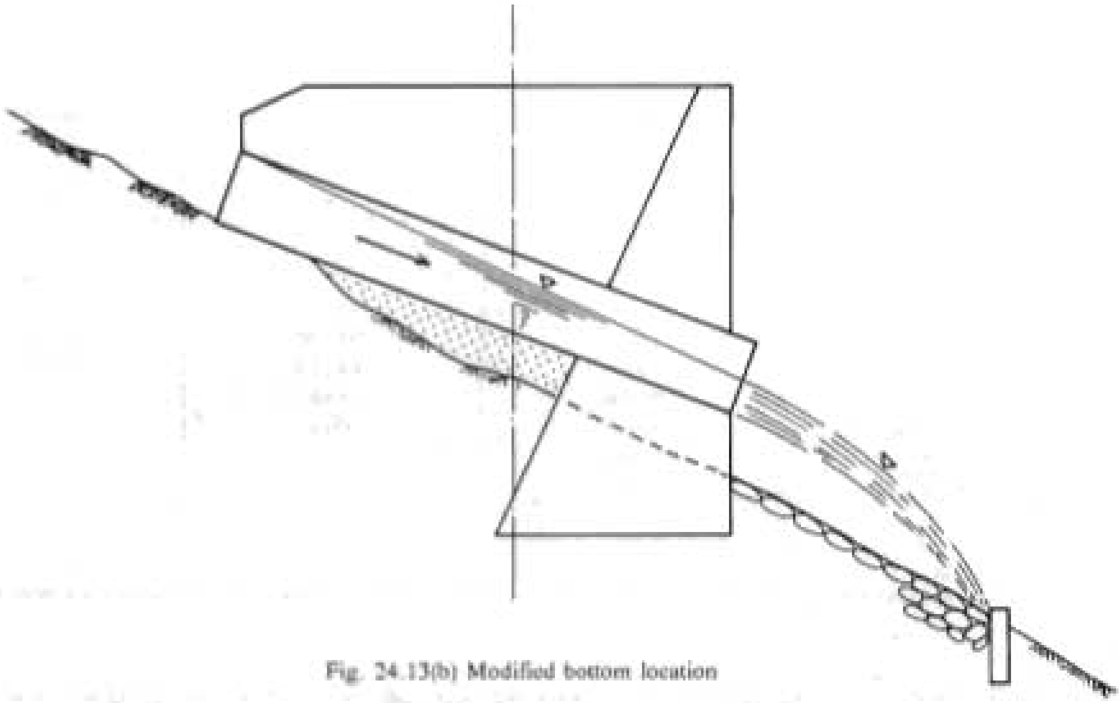


Fig. 24.13(b) Modified bottom location

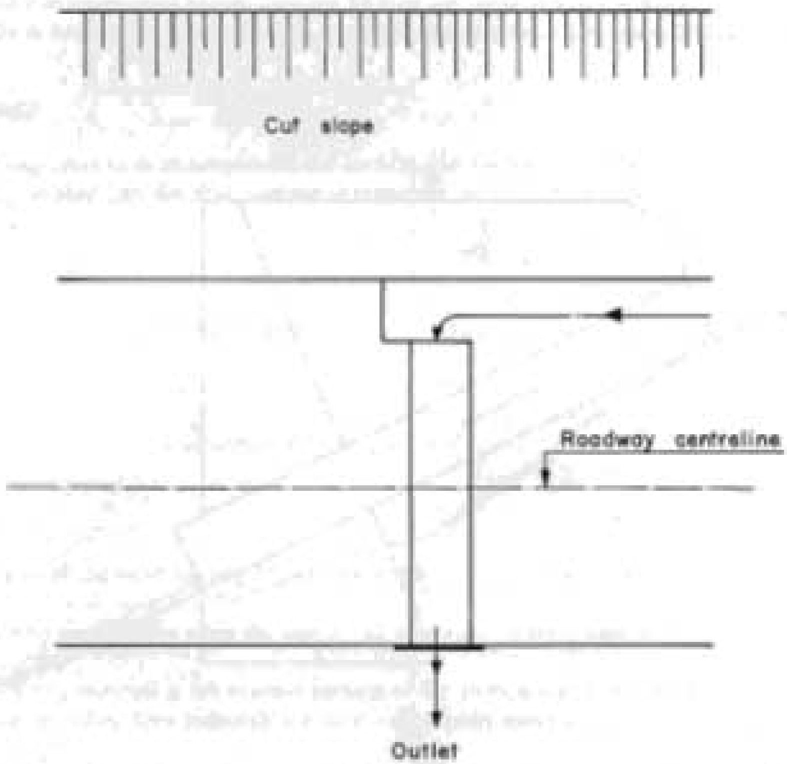
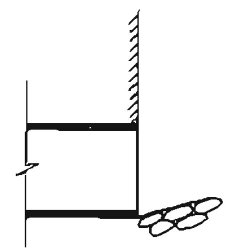
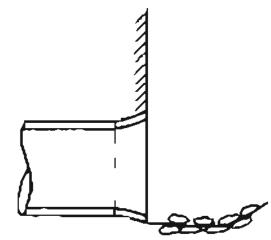


Fig. 24.13(c) Transverse relief location

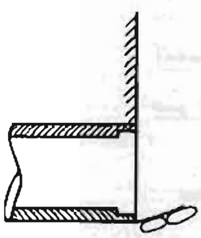
Fig. 24.13 Culvert location



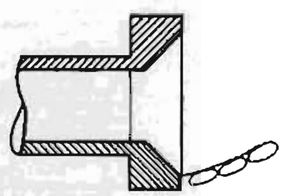
(a) Square end type with headwall



b) Rounded entrance with headwall



c) Endgrooved culvert pipe



d) Bevelled pipe entrance

Fig. 24.14: Entrance configurations of culverts

Notes

It has been established that inlet configurations affect the flow in the culvert significantly in inlet control flow. Streamlining the flow entering the culvert by rounding the entrance or tapering the inlets of rectangular or square cross-sections increases the flow through culverts; i.e., decreases the culvert size for a given design flow and this results in lower costs of culverts.

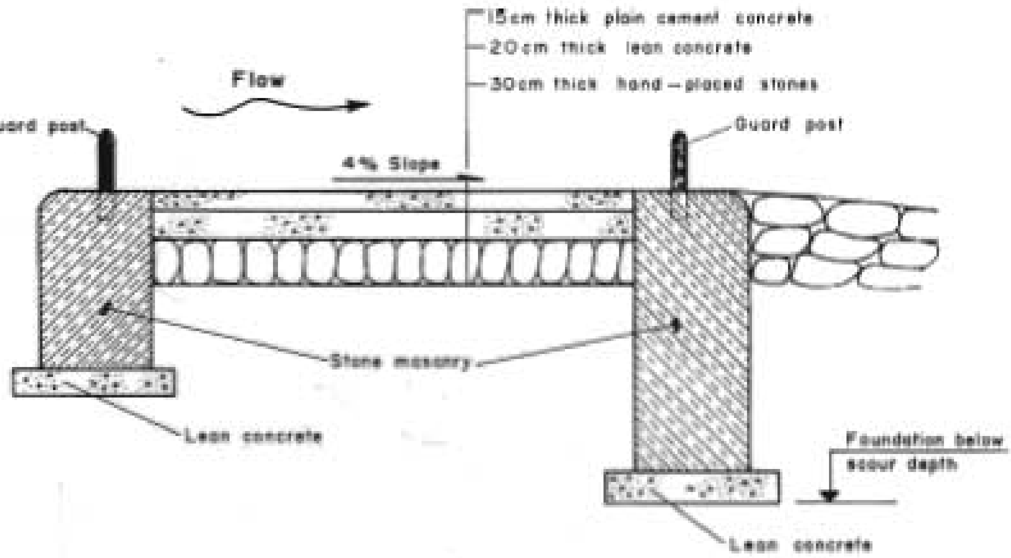


Fig. 24.15a Ford or floodway

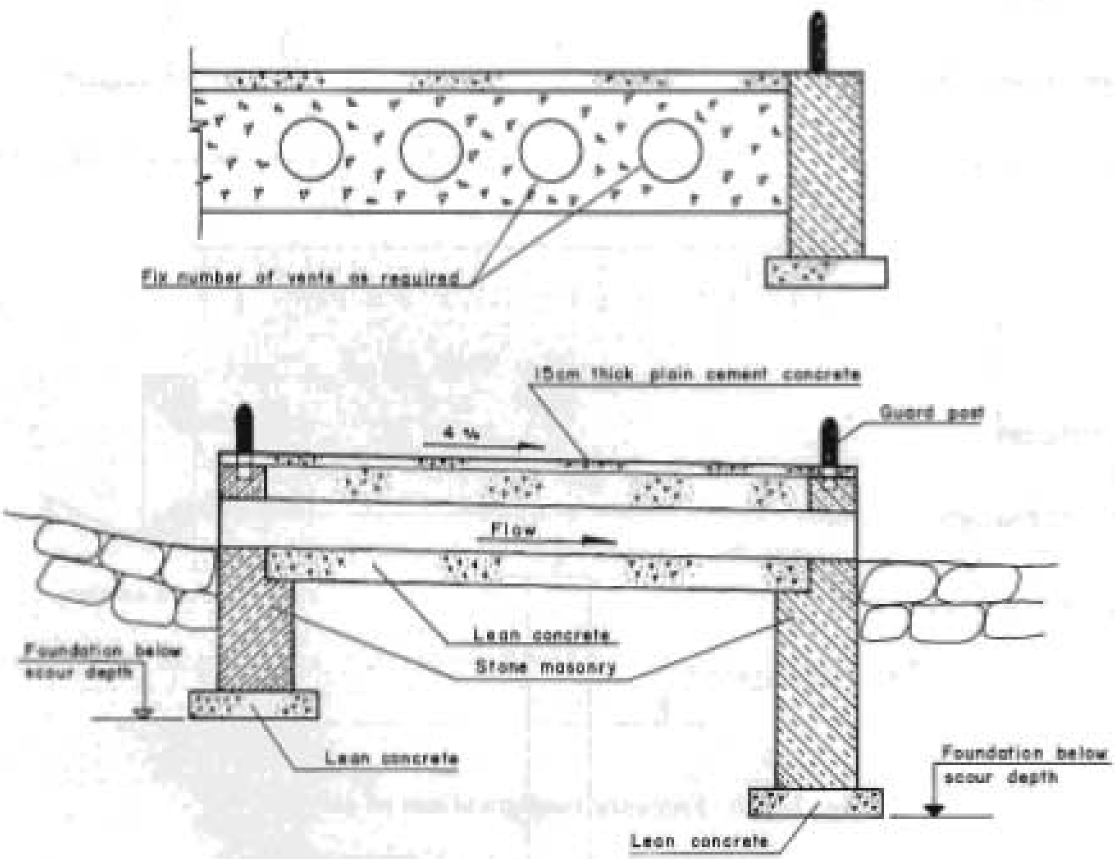


Fig. 24.15b Vented causeway

Notes

- Fords or floodways are built across relatively wide non-perennial streams
- Vented causeways, built across perennial streams, provide vents for passing dry season flows underneath the road carriageway.
- While providing the causeways, attention is needed to provide transition curves long enough so that a smooth ride is possible.
- Small causeways or scuppers are provided extending across the formation width of the road. These causeways, very commonly used in the hills, are unsuitable for road stretches with steep gradients.
- Adequate upstream and downstream protection against scour is vital to the survival of all types of causeways

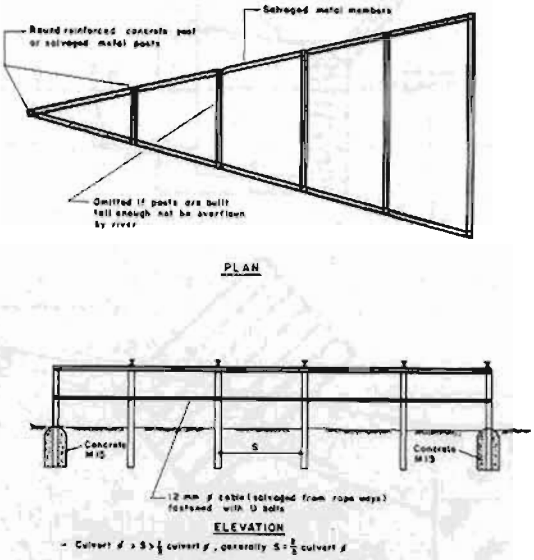
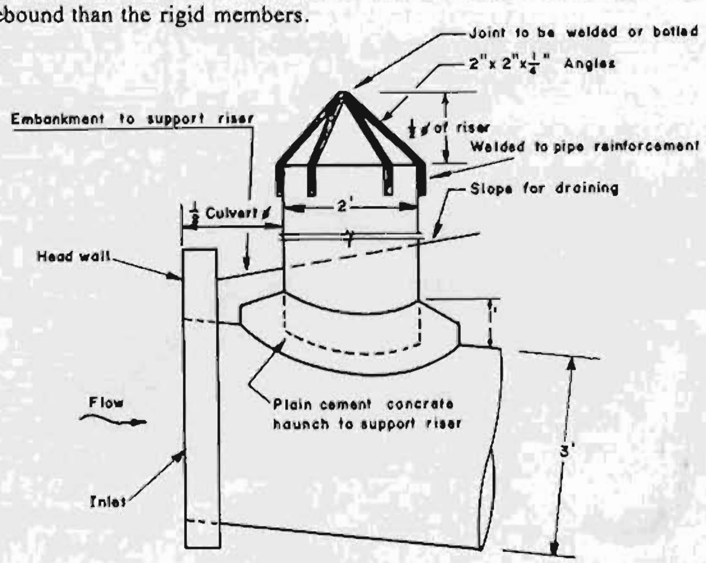


Fig. 24.16 Debris deflector

Notes

Debris deflectors are placed at the culvert inlet to deflect large boulders from the inlet. The cable deflects boulders much more easily with elastic rebound than the rigid members.



DEBRIS RELIEF RISER

Fig. 24.17 Debris relief riser

Notes

The debris relief riser is a closed structure which acts as a relief device in the event of clogging of the culvert entrance by debris. Water can enter through the top of the relief riser when the water level rises becausing of clogging of the culvert entrance.

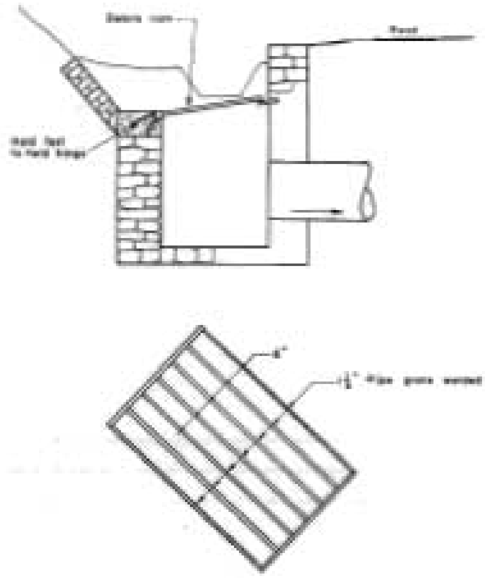
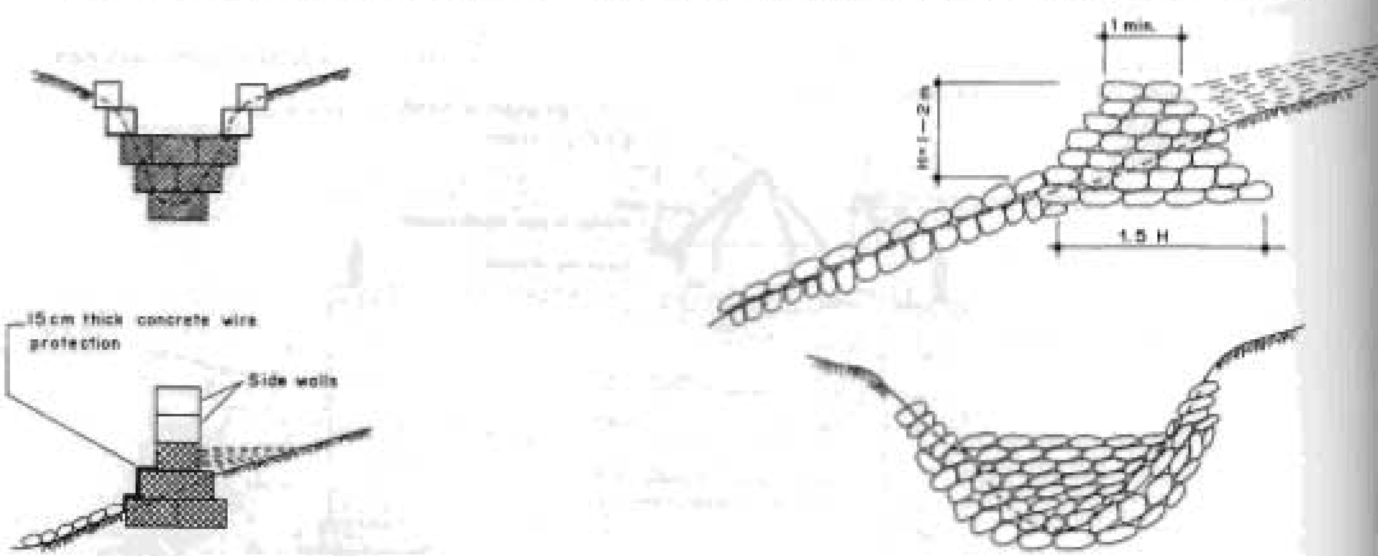


Fig. 24.18 Debris rack

Notes

Debris racks are structures placed above the drop inlets of culverts to prevent clogging of the culvert inlet by debris. They are also used to prevent light and medium floating debris from entering the culvert inlet, in which case they are placed vertically.



(a) Gabion check dam

(b) Loose rock check dam

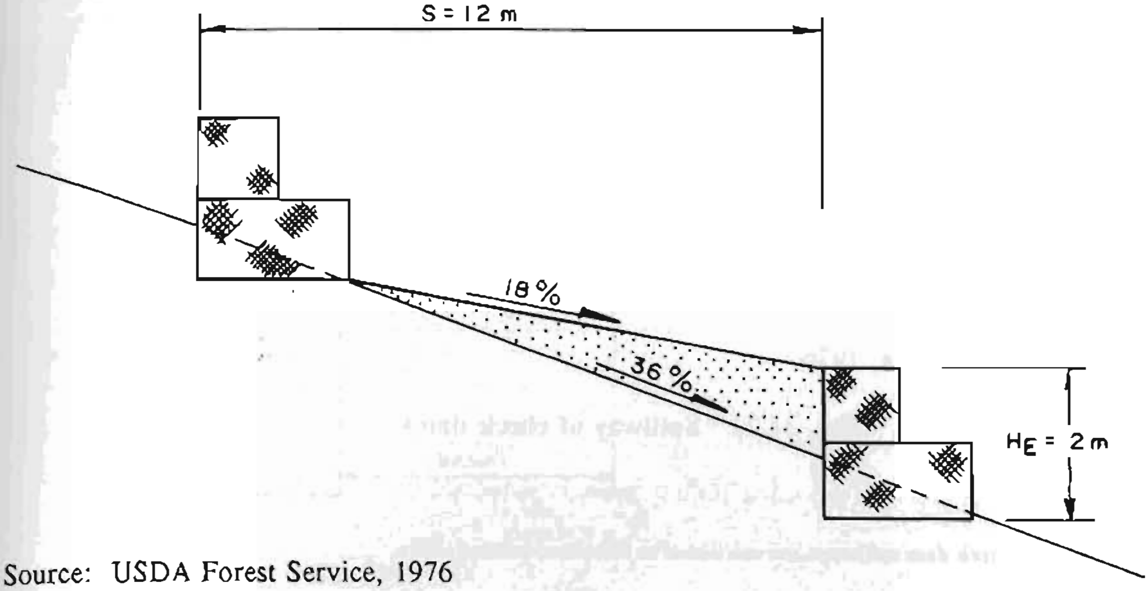
Fig. 24.19 Check dams for gully control

Notes

Gabion check dams are flexible and can take minor settlements quite easily. These are convenient to construct and much stronger than loose rock check dams.

Loose rock check dams are suitable for very small gullies.

Care must be taken not to take out an excess of gully bed materials which otherwise are protecting the gully from bed erosion.



Source: USDA Forest Service, 1976

$K = 0.5$, $G = 0.36$, $H_E = 2m$
slope of accumulated debris $(1-K)G = 0.18$
$S = 11.67m$

Fig. 24.20 Check dam spacing

Notes

Spacing of check dams:

o widely-spaced high check dams are used if a large deposition of sediment behind the structures is desired, and

o closely-spaced relatively low dams are required, if the aim is only to stabilize gully slopes.

Spacing equation, by Heede and Mufich (1973):

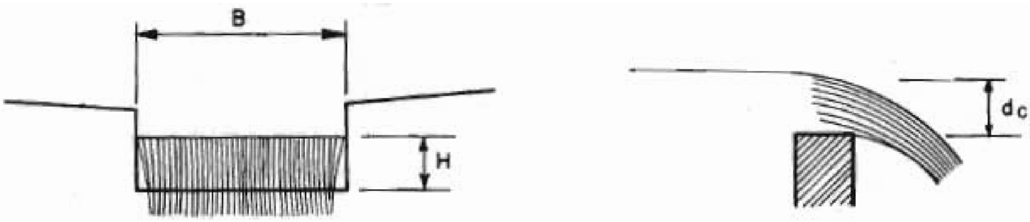
$$S = \frac{H_E}{KG\cos\alpha}$$

where,

S	=	spacing,
H_E	=	effective dam height as measured from bottom to spillway crest,
G	=	$\tan\alpha$, α = angle corresponding to slope of the gully,
K	=	a constant specific to the gully area,
K	=	0.3 when $G < 0.2$, and
K	=	0.5 when $G > 0.2$ have been typically used.

The equation assumes the gradient of sediment deposition as (1-K)G.

If only gully slope stabilization is required, higher values of K (say $K=0.7$) may be assumed for increasing the spacing of check dams.



Source: USDA Forest Service, 1976

Fig. 24.21 Spillway of check dams

Notes

o Discharges over check dam spillways are calculated as for broad-crested weirs:

$$Q = CBH^{3/2}$$

where,

C = coefficient of the weir, generally taken as 1.65,
B = effective length of the weir, and
H = head of flow above the weir crest.

o Critical depth over the spillway, $d_{cr} = 3\sqrt{\frac{q^2}{g}}$

where,

q = Q/B,
Q = estimated peak runoff in the gully

o For trapezoidal shape of gully.
$B = L_B$

where,

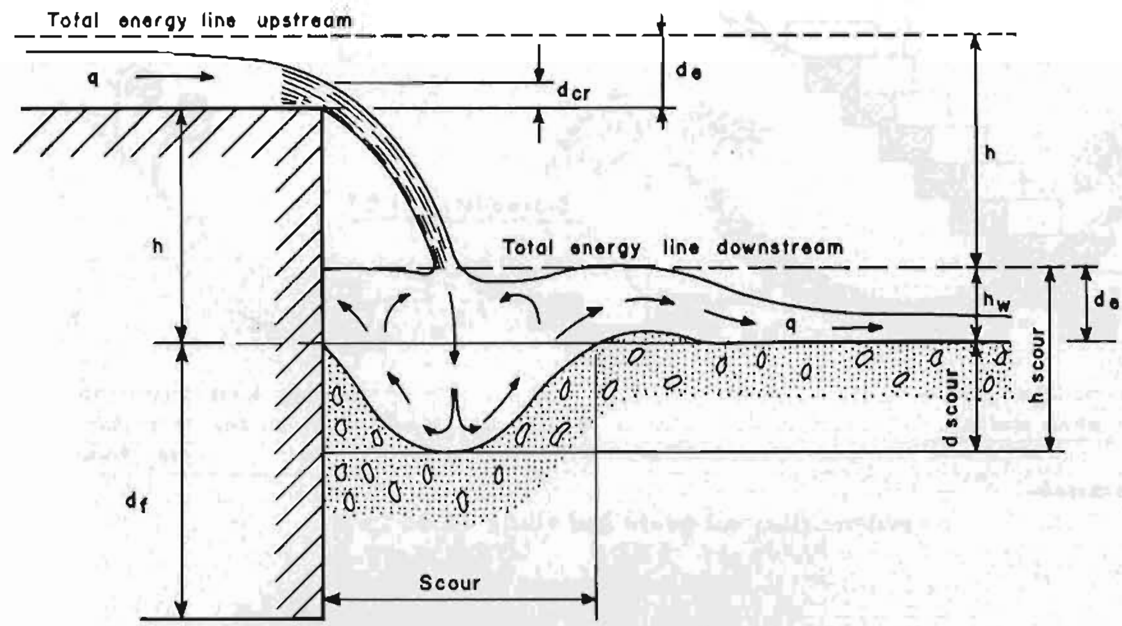
L_B = bottom width of gully

o For rectangular and v-shaped gullies.

$$B = \frac{L_u}{D} H_E - f$$

where,

L_u = bank width of gully,
D = depth of gully,
f = 0.3 for D $\leq$ 1.5m, and
f = 0.6 for D $>$ 1.5m.



Source: adapted from Hiller 1979

Fig. 24.22 Flow through a checkdam

Notes on checkdam apron and foundation depths

$$\textit{Water depth of scour} \quad (h_{scour}) = \frac{0.79\ h^{0.343}\ q^{0.686}}{d_{95}^{\ 0.372}}$$

$$\textit{length of scour} \quad (l_{scour}) = 0.73\ \frac{h^{0.457}\ q^{0.914}}{d_{95}^{\ 0.828}}$$

Breadth of scourhole $(b_{scour}) = 1.5 \times B$

Scour depth $(d_{scour}) = h_{scour} - h_w$

Water cushion height, $h_w = \dfrac{3}{2}\ d_{cr}$

h = total height of the dam = H_E

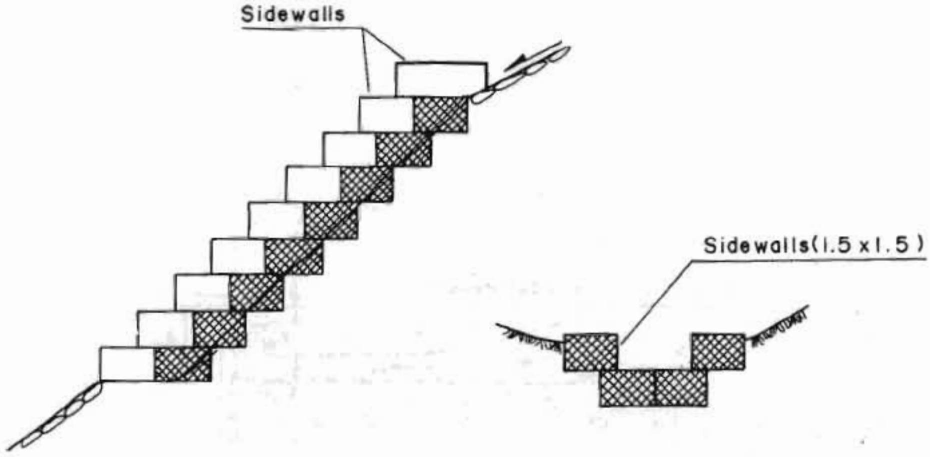
d_{95} = size of the seive which passes 95% of the bed material or loose rock apron.

Adopt

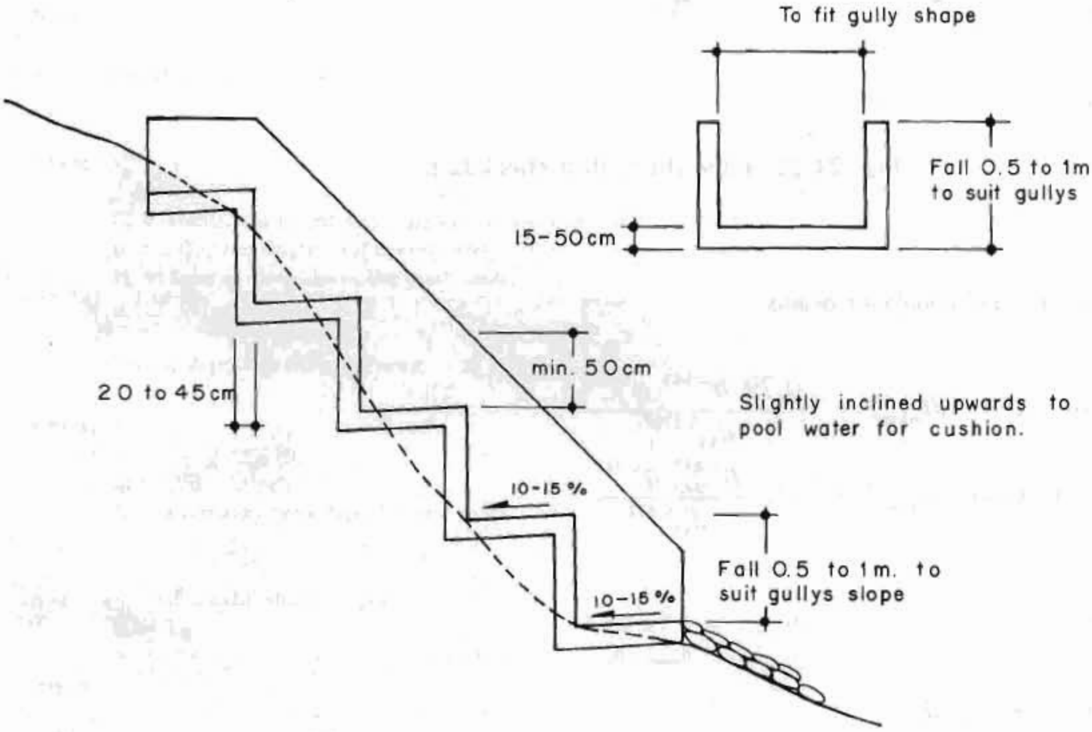
depth of foundations $d_f > d_{scour}$
length of apron $L > ½\ l_{scour}$
breadth of apron $B > b_{scour}$

No downstream check dam should be placed nearer than l_{scour}, i.e., spacing $S >> l_{scour}$

The calculations are based on a four hour peak runoff.



(a) gabion cascade



(b) Stone masonry cascade

Fig. 24.23 Cascades for gully control

Notes

Cascades are used for direct flow over short stretches of very steep erodible gully slopes.

Cascades are used to reduce velocities at culvert entrance.

Gabion cascades are not preferred in gullies with a large bedload, because of damage to gabion wire by boulders.

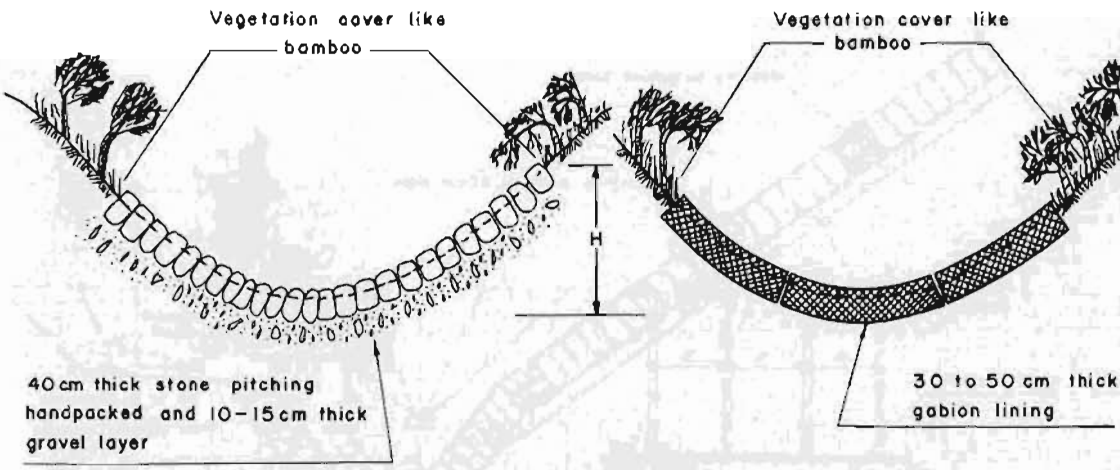


Fig. 24.24 Gully bed lining for gully control

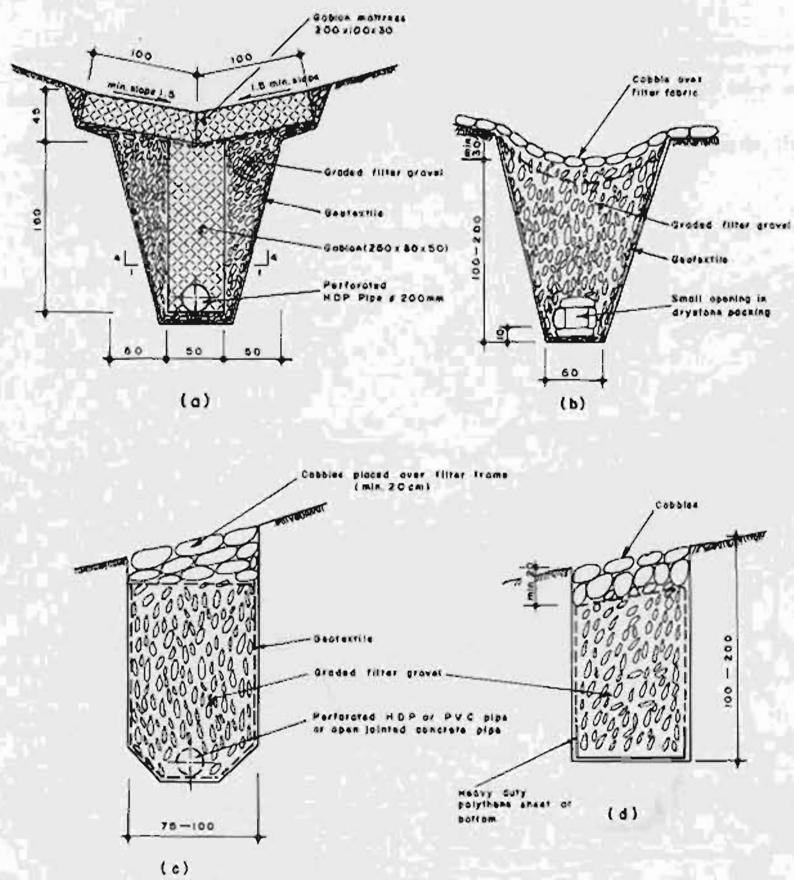


Fig. 24.25 Landslide and cut slope drainage by shallow sub-surface drains

Notes

Types (a), (b), and (c), are suitable for herringbone pattern collectors, of which the section with a pipe is suitable for high flows. Type (d) is the counterfort drain collecting flow from herringbone drains. Filter fabric can be omitted by providing graded filter. For graded filter requirements, see notes below Figure 24.12.

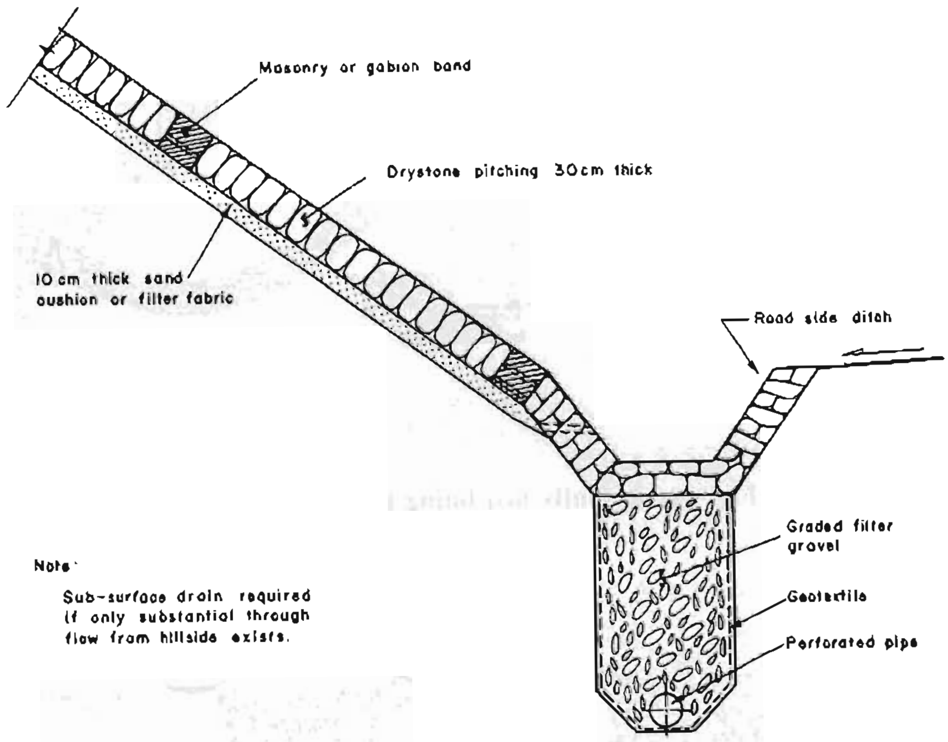


Fig. 24.26 Saturated slope drainage

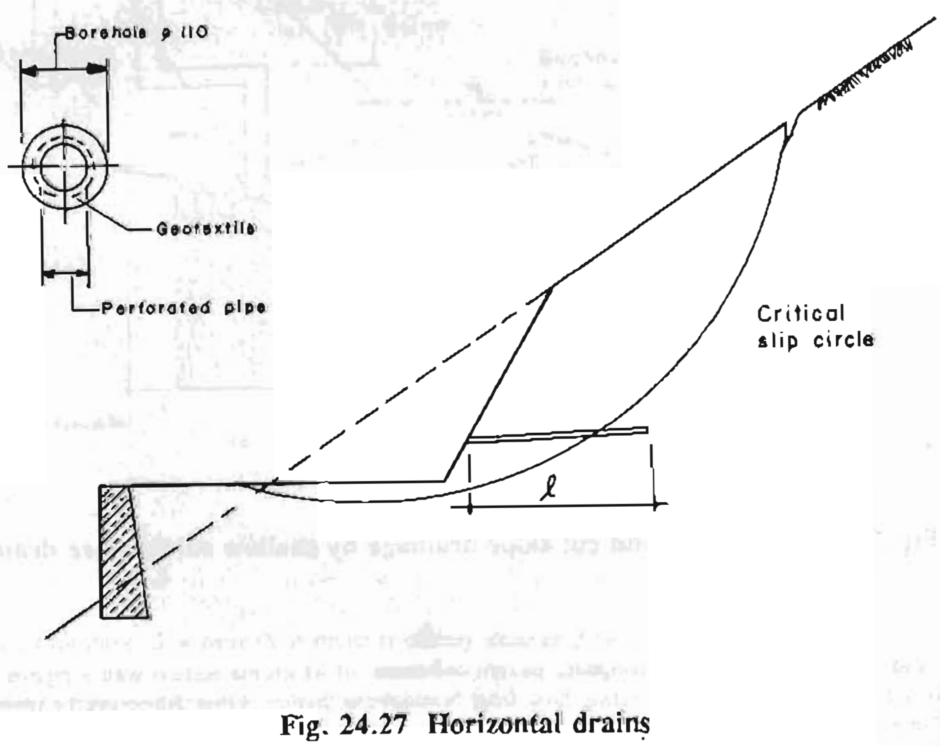
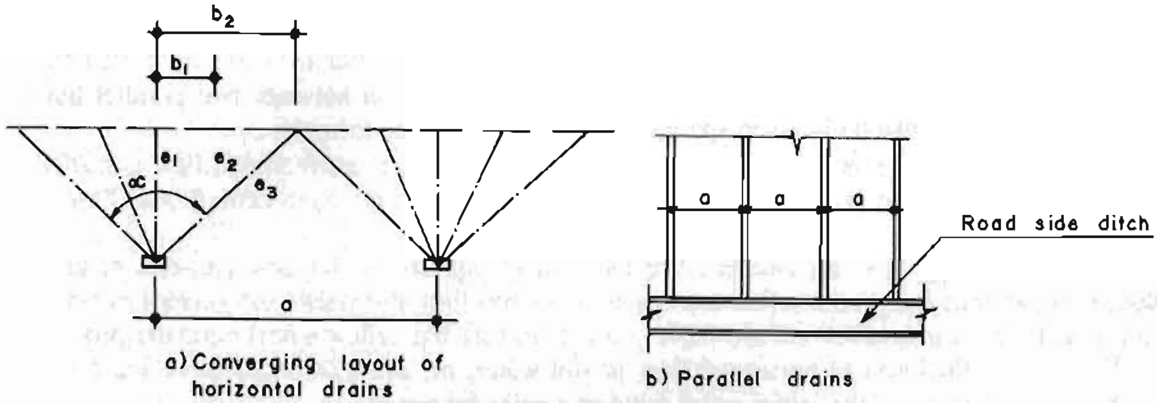
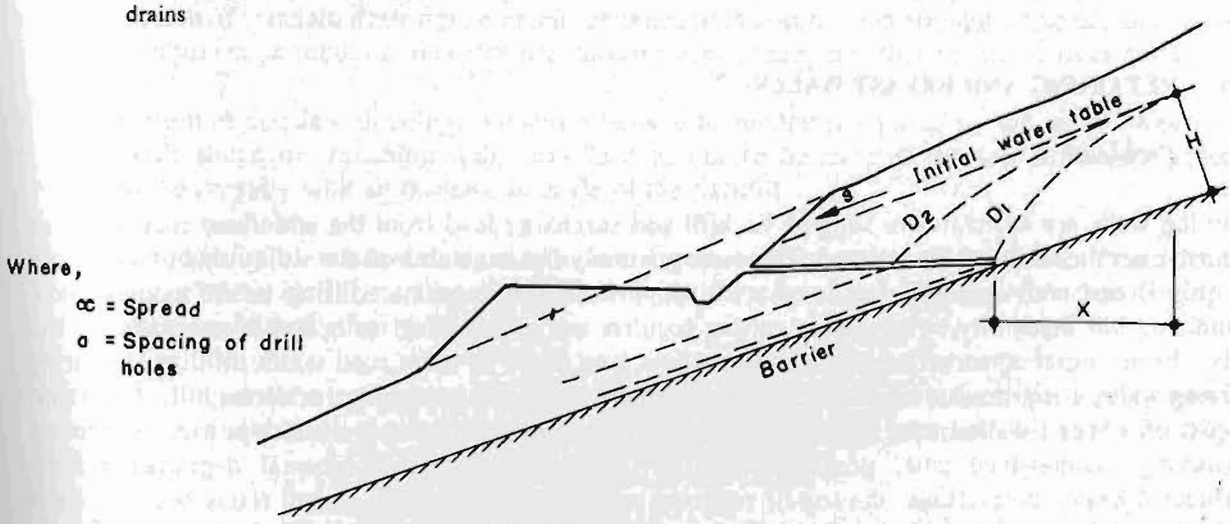


Fig. 24.27 Horizontal drains



(a) Converging layout of horizontal drains

(b) Parallel drains



(c) Drawdown upslope and downslope

Fig. 24.28 Layout and water table drawdowns of horizontal drains

Notes

Installation of horizontal drains consists of drilling horizontally and installing perforated pipes inside the drill hole to reduce groundwater pressures, thus increasing the shear strength of the soil and improving the stability.

The length *l* of the horizontal drain is determined by slope stability analysis. The drain should extend beyond the critical slip circle. Isenhower (1988) has concluded that lengthening of the drain beyond the circle has a limit beyond which the lengthened drain can improve the factor of safety of slope stability only marginally.

The spacing of drains depends on desired drawdown and spread angle. Design of horizontal drains for highway cut slope stabilility is discussed in detail by Pazin, T.C. et al. (1977), Isenhower W.M. (1988), and Prellwitz R.W. (1978).

Phreatic surface D_1 corresponds to the drawdown profile if a blanket drain, extending horizontally over the entire slope, is provided. This is approximated by installation of horizontal drains converging at the outlet (see Fig 24.28a).

Phreatic surface D_2 corresponds to the drawdown profile existing in between two parallel horizontal drains. The drawdown upslope is given approximately by the Glover formula:

$$X = (\frac{4}{3})\ (\frac{H}{S})$$

where,

H = thickness of horizontal flow of soil water, m, and
S = slope of the initial water table as a ratio (m per m).

If the soil strata are the same, the water table below the drain continues parallel to the original water table; except that the water table is now lowered to a level that depends upon the upslope drawdown as shown in Figure 24.28.

24.6 RETAINING AND BREAST WALLS

24.6.1 *Problems*

Retaining walls are structures to support backfill and surcharge load from the additional road width or platform over the wall in hill sections. These are generally the structures on the valley side of a hill road section. Breast walls are structures meant to support cut slopes without filling in the mountainside. Minimizing hill instability by excessive cutting requires use of retaining walls and breast walls for hill roads. From initial construction cost considerations, one metre of extra road width infilling, requiring retaining walls, costs much more than constructing the same width by cutting inside the hill. Similarly, the cost of a breast-walled road section is several times more than a non-walled slope cut. However, considering maintenance cost, progressive slope instability, and environmental degradation from unprotected heavy excavations, the use of retaining walls and breast walls on hill roads becomes most essential.

The effectiveness of these structures depends upon proper design, construction, and maintenance. Proper understanding of design concepts, construction, maintenance problems, and cost implications is very essential in developing appropriate designs.

The following are the problems leading to cost overruns from traditional practices in retaining and breast wall design and constructions.

o Retaining and breast walls are normally not intended to stabilize slope failures. They are mainly meant to support the active or passive earth pressure from the assumed failure wedge above the base of the wall. The stabilization of existing or probable failure planes caused by landslides, flows, and falls requires separate treatment and specific design approaches.

- Use of retaining and breast walls without design concepts leads to inappropriate selections or failures.

- It is generally not possible to design each and every wall along the entire length of a road. Limitations of rule of thumb design, empirical design, and standard design, if not well understood, lead to failures or over-design.

- Selecting wall height without taking into consideration hill slope, effect of front batter on height of wall, and foundation conditions results in underestimation of quantities and cost.

- Cement masonry and gabion masonry walls tend to be almost the only types adopted. Lack of effort in looking for alternative wall types, such as drum walls, reinforced earth walls, reinforced cement, concrete frame walls, and anchored walls, rules out the development of other techniques which are more economical.

- Gabion walls tend to be indiscriminately applied even in areas where foundations are rocky; materials such as cement, sand, and water are easily available; and cement masonry or other types of wall could be more economical and durable.

- Adoption of standard drawings, without reference to the foundation strength of actual site vis-a-vis the minimum strength required for the standard walls, leads to either failure or over-design.

- Adoption of standard drawings without reference to surcharge (caused by the actual slope of the backfill above the retaining wall) may lead to failure because of the fact that earth pressure increases rapidly with an increase in angle of the backfill.

- Blind adoption of standard design leads to unnecessary excavation and sometimes destabilization in areas where bedrock or sound foundations could easily allow stepped or benched cutting for foundations. See Figure 24.29.

- A retaining wall on a thin talus slope may lead to failure of the entire talus slope during monsoon, because of the quick rise of the water table above the relatively impervious bedrock (Figure 24.30).

- Front-battered retaining walls are many times more expensive than back-battered walls in steep hilly areas. Figure 24.31 illustrates this point.

- The practice of undertaking wall construction, invariably after road cutting, results in the problem of disposal of excavated material and loss of topsoil that could otherwise be used for vegetation.

- A series of retaining walls, one above another, on an unstable or marginally stable slope adds more pressure on to the lower walls and destabilization of the slope; contrary to the aim of stabilizing the slope (Figure 24.32).

- Retaining walls alone cannot stabilize an unstable hill slope. Only the fill slope and cut slopes can be stabilized by retaining and breast walls.

o The causes of wall failure commonly noticed as a result of poor construction are:

- differential settlement of dry masonry walls during rainy season,
- poor bonding and wrong bedding of stones in drystone masonry walls (see Figure 24.33),
- roots of trees pushing the wall from behind,
- unstable boulders not cleared from the foundation,
- designed section not fully ensured (poor supervision or negligence),
- poor backfill resulting in high seepage pressure while section design assumes no pore pressure,
- non-functioning weep-holes in cement masonry walls, and
- lack of toe protection for walls (see Figure 24.34).

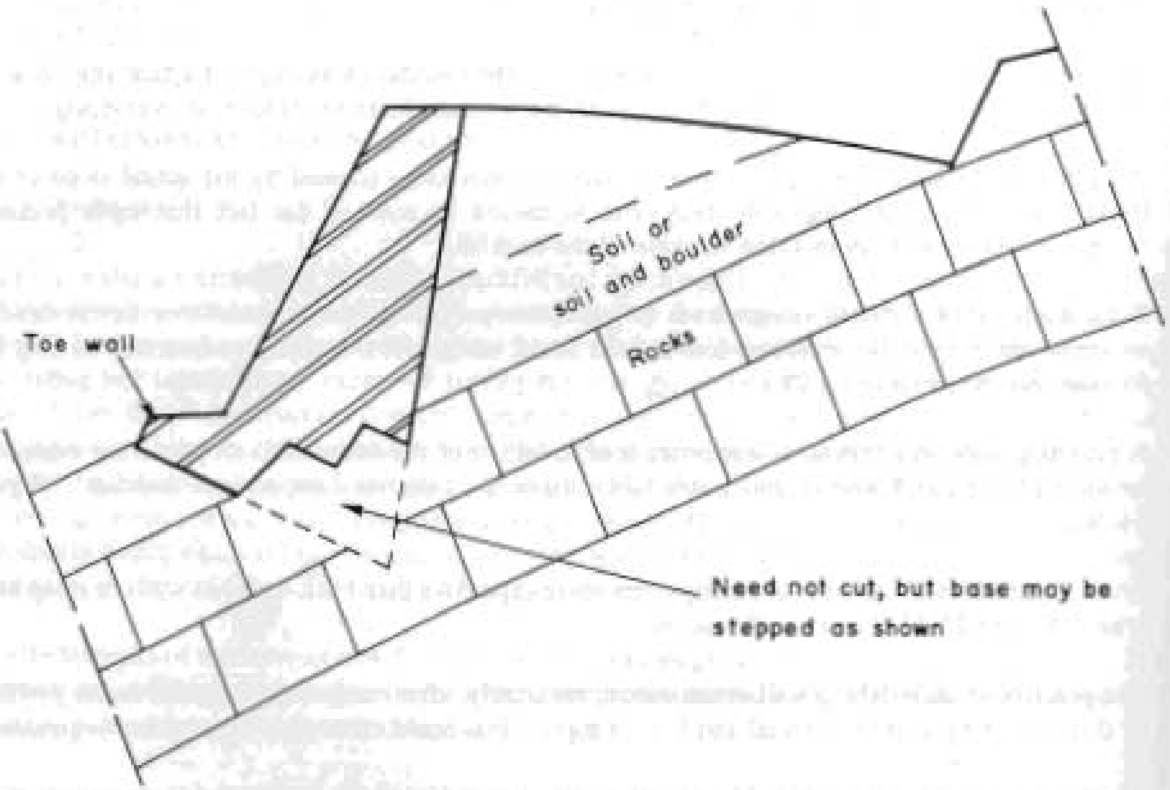
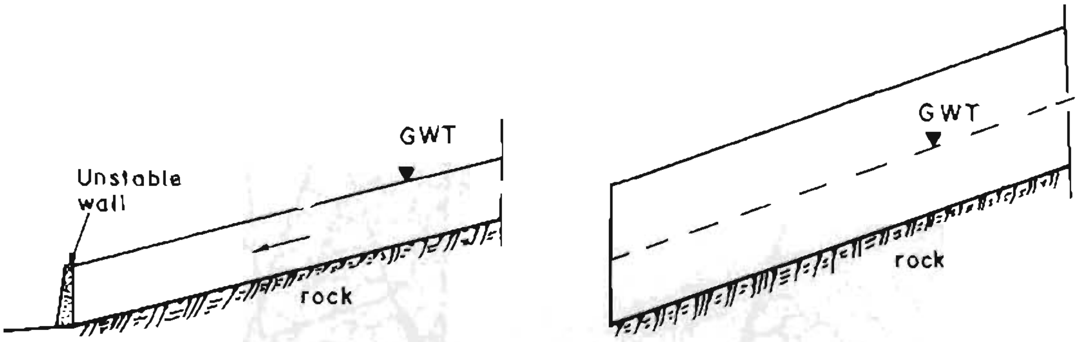


Fig. 24.29 Avoiding unnecessary cut for retaining wall on rock



(i) Unstable talus slope (ii) Stable talus slopes

Fig. 24.30 Thin cover of talus/debris is likely to fail even on gentle slopes

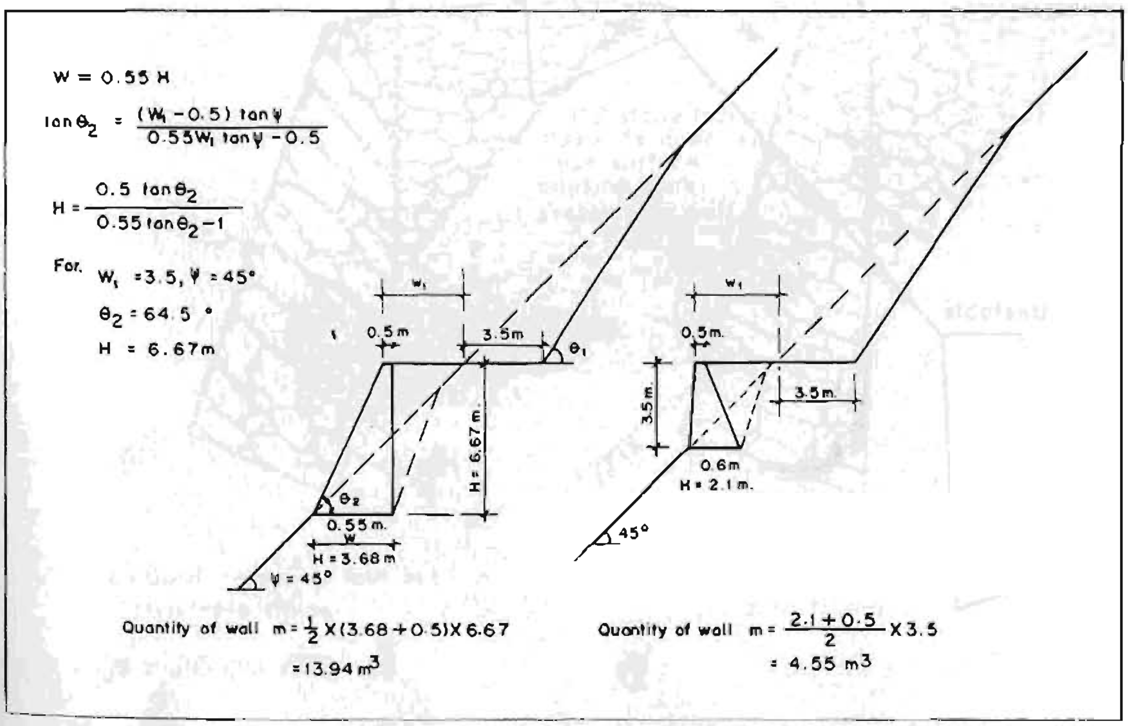


Fig. 24.31 Back-battered versus front-battered retaining wall

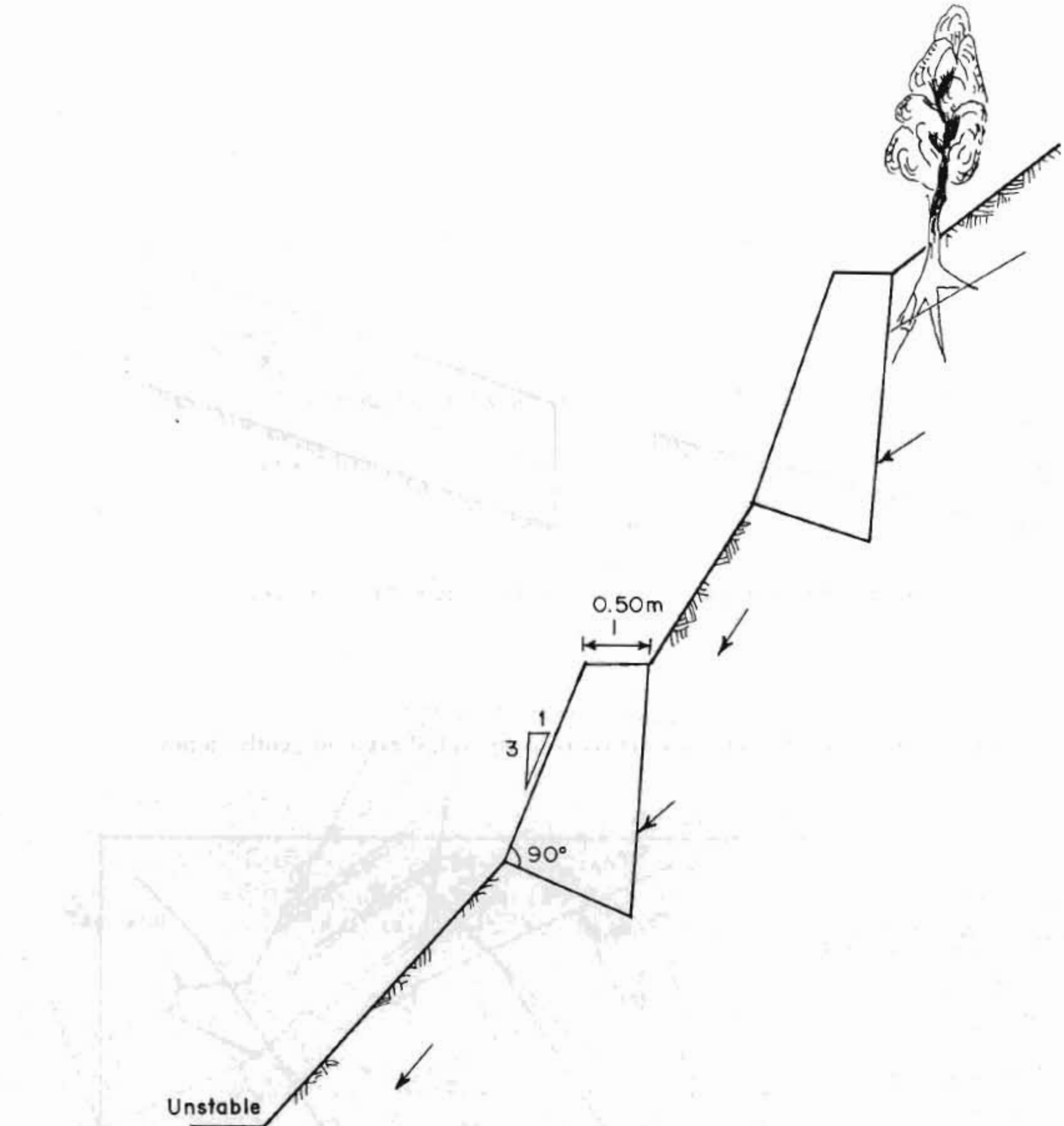
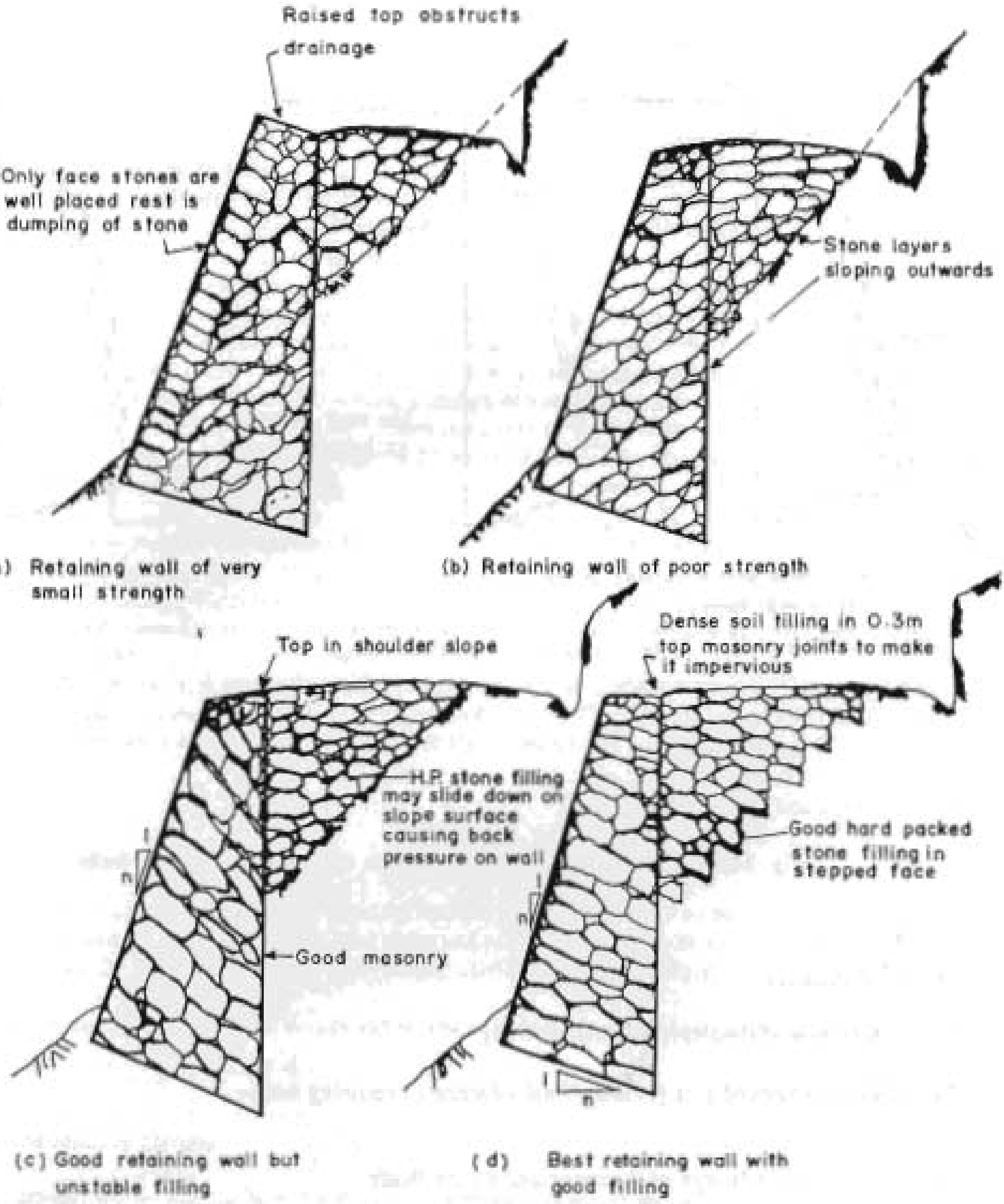


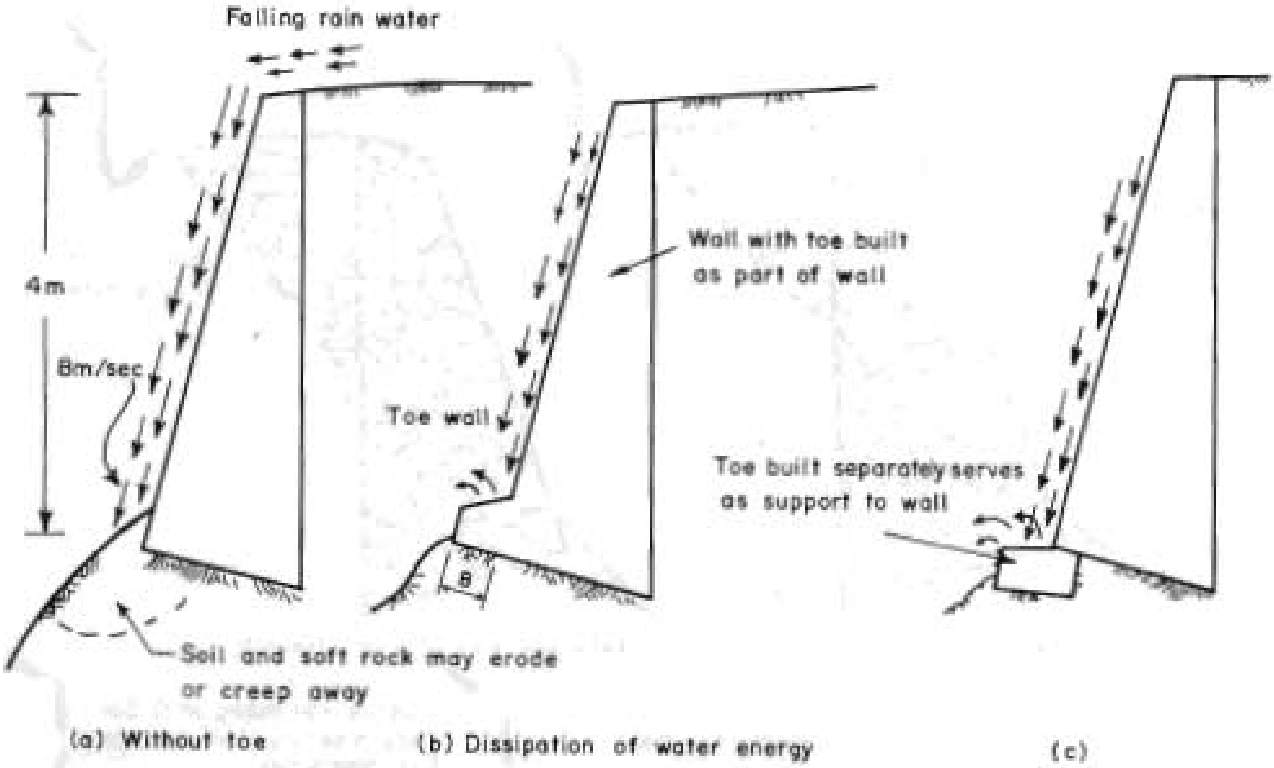
Fig. 24.32 Destabilization of slope by series of retaining walls



Source: Arya and Gupta 1983

Fig 24.33 **Wrong and right methods of construction of drystone masonry wall**

Construction details for drystone masonry wall



Source: Arya and Gupta 1983

Fig. 24.34 Construction of toe projection for toe projection of walls

24.6.2 ***Guidelines***

i) *Selection of Retaining and Breast Walls*

Table 24.6 is suggested as a guide for final selection of retaining walls.

ii) *Approach to Design of Retaining and Breast Walls*

The level of effort required for designing these structures at different stages of a project cycle is different depending upon the resources available at the time. As far as hill roads with innumerable structures are concerned, design is an ongoing process with increasing refinements until actual construction.

Table 24.7 provides guidelines on the selection of a design approach based upon the size of the structure, hazard level of the area, and the project cycle in question.

Retaining walls up to 3m in height may be designed by the following rule of thumb:

Top width = 0.50 to 0.6m
Base width = 0.55 to 0.65 times the height of the wall

Standard designs or drawings may be followed for walls less than 8m in height and 120m^2 in a low hazard area provided the allowable bearing pressure is more than the maximum pressure indicated in the standard drawing. Table 24.8 gives standard designs based on Coloumb's Wedge Analysis using the Computer Programme "RETAIN" Figures 24.35 and 24.36 present standard drawings of gabion walls (see Annex 4). Figures 24.37 and 24.38 present standard drawings of composite (cement + drystone masonry) for walls. Figures 24.39 to 24.41 present drawings of drum walls. Figure 24.42 to 24.45 present drawings of reinforced crib walls. Figures 24.46 to 24.48 show drawings of a typical reinforced concrete frame wall.

Earthquake considerations lead to excessive wall dimensions. High walls (higher than 6m) may, therefore, be avoided by alternative geometric designs or be justified by risk analysis. The dip in the base of the wall increases dynamic stability significantly. For low volume roads, it is suggested that walls may not be designed for earthquake forces. It would be more economical to repair failed walls after earthquakes.

Poor backfill and poor drainage behind the walls involve complicated conditions outside the normal design assumptions and, in any case, lead to excessive wall dimensions. These should, therefore, be avoided except for cases supported and justified by specific analysis and design.

Semi-empirical Designs

Field conditions outside those covered by standard designs or drawings need to be checked. Simplified design methods based on empirical values from observed performance, such as in Tables 24.9 to 24.14 and Figures 24.49, 24.50, and theoretical analyses are useful for manual designs.

Sections 24.6.3 and 24.6.3(a) present details and solved examples by semi-empirical methods of design.

Theoretical Methods of Design

There are various theories and methods for the analyses of earth pressure and these are available in most books on soil mechanics, foundation engineering, or geotechnical engineering. The computer programme RETAIN, is recommended to save time from repetitive calculations relating to several retaining walls, as in the case of a road.

Table 24.6 Design approach for retaining and breast walls at different project cycles

Design Method	Wall Size Height, m	Area, m^2	Type	Hazard level	Project Cycle
Rule of thumb	< 3	< 120	D, CM, G	L	All
	< 3	< 120	G	H	All
	< 3	< 120	D, CM, G	L	PF, F, D, CON.
	< 3	< 120	G	H	PF, F
Standard Drawing	4-8	< 120	D, CM, G	L	All
	4-8	< 120	G	H	PF, F
	4-8	< 120	D, CM, G	L	PF, F
	4-8	< 120	G	H	PF, F
	> 8	< 120	CM, G	L	PF, F
	> 8	< 120	G	H	PF, F
Semi-Empirical	< 3	> 120	D, CM, G	H	CON.
	4-8	< 120	G	H	D, CON
	4-8	> 120	D, CM, G	L	D, CON
	4-8	> 120	G	H	D, CON
	> 8	< 120	CM, G	L	D, CON
	> 8	> 120	G	H	D, CON
	> 8	> 120	CM, G	L	PF, F, D
	> 8	> 120	G	H	PF, F
Theoretical Analysis and Specific Design	> 8	> 120	CM, G	L	CON
	> 8	> 120	G	H	D, CON

TYPE OF WALL

D - Drystone Masonry
CM - Cement Stone Masonry
G - Gabion Masonry

HAZARD LEVEL OF THE AREA

L - Low
H - High

PROJECT CYCLE

PF - Prefeasibility
F - Feasibility
D - Detailed Design

OTHER TYPES OF WALL

Follow proprietary, semi-empirical, or theoretical methods.

Table 24.8 Standard design of cement masonry and drystone masonry retaining walls with horizontal base

BACK FILL TYPE	PARTICULARS	CEMENT MASONRY												DRYSTONE MASONRY											
		HEIGHT 3M			HEIGHT 6M			HEIGHT 8M			HEIGHT 10M			HEIGHT 3M			HEIGHT 6M			HEIGHT 8M			HEIGHT 10M		
GOOD BACK-FILL	TOP WIDTH	0.65	0.7	-	0.75	1.0	1.0	0.8	1.0	1.0	0.9	1.0	-	0.70	-	-	0.75	0.95	1.0	0.85	1.0	1.0	0.90	1.0	-
FULL DRAIN AGE	BASE WIDTH	1.91	2.01	-	3.92	4.78	8.41	5.23	8.1	10.96	6.64	13.57	-	2.01	-	-	3.92	4.32	8.5	5.33	6.89	11.81	6.64	14.58	-
GW, GP, SW, SP	FOUND-ATION PRESS-SURE	14.0	13.0	-	25.0	20.0	13.0	33.0	20.0	17.0	40.0	21.0	-	11.0	-	-	22.0	20.0	17.0	29.0	20.0	13.0	36.0	16.0	-
FAIR BACK-FILL	TOP WIDTH	0.60	0.75	-	0.90	1.0	1.0	0.95	1.0	1.0	1.0	1.0	1.0	0.75	-	-	0.85	1.0	-	1.0	1.0	1.0	1.0	1.0	1.0
LOW PORE WAT-ER PRES.	BASE WIDTH	1.81	2.11	-	4.12	4.47	4.88	5.53	6.59	8.14	6.94	9.9	14.03	2.11	-	-	4.12	4.42	-	5.63	6.49	6.94	6.94	8.5	10.26
GM, SM, SM-SC	FOUND-ATION PRESS-URE	15.0	13.0	-	25.0	22.0	20.0	32.0	25.0	20.0	39.0	25.0	20.0	11.0	-	-	22.0	20.0	-	28.0	22.0	20.0	34.0	25.0	20.0
POOR BACK-FILL	TOP WIDTH	-	-	-	-	-	-	1.0	1.0	1.0	1.0	1.0	-	-	-	-	-	-	-	1.0	1.0	1.0	1.0	1.0	1.0
HIGH PORE WAT-ER PRES SURE	BASE WIDTH	-	-	-	-	-	-	6.49	7.89	8.5	7.79	11.01	-	-	-	-	-	-	-	6.54	8.65	8.70	7.84	10.11	11.97
GC, SC, ML	FOUND-ATION PRESS-URE	-	-	-	-	-	-	22.0	20.0	19.0	29.0	23.0	-	-	-	-	-	-	-	22.0	20.0	16.0	25.0	20.0	18.0

WALL GEOMETRY:
FRONT PACK VERTICAL
BACK FACE INCLINED

BACKFILL TOP:
HORIZONTAL WITH SURCHARGE OF 1.5 T/m^2

Note: Select wall dimensions such that allowable bearing pressure > foundation pressure

GABION WALLS FOR SLOPING SURCHARGE LOADS

WALL TYPE	HEIGHT m	BASE WIDTH m	FOUNDATION MATERIAL	SUPPORTED SOIL TYPE: DESCRIPTION	φ DEG	δ DEG	MOIST UNIT WT KG/m³	θ DEG	F.S O'TURNING	F.S SLIDING	SPEC. KEY	e	TOE PRESSURE KG/m²	HEEL PRESS. KG/m²	REMARKS
A 32	3	2	SOUND ROCK IN PLACE OR	FREE DRAINING GRANULAR	45°	45°	1600	6	6.2	5.6	NO	-0.2	2700	7900	NOTE: e IS THE DISTANCE FROM THE CENTER OF THE WALL BASE
A 53	5	3	BROKEN	SOIL OR BROKEN ROCK GW,	45	45	1600	6	4.8	4.3	NO	-0.3	3852	12171	TO THE POINT OF EXIT OF THE RESULTANT OF FORCES.
A 63	6	3	"	GP, SW, SP.	45	45	1600	6	3.6	3.8	NO	-0.1	8061	13048	POSITIVE VALUES ARE TOWARD TOE, NEGATIVE VALUES ARE
A 63	6	3	"	"	45	45	1600	0	2.6	2.7	NO	0.2	14159	6258	TOWARD THE HEEL OF WALL.
A 64	6	4	"	"	45	45	1600	6	3.3	4.0	NO	-0.5	1790	14837	BASE INCLINATION θ, SHOWN AT 6°, CONSTRUCTION AT 10° SATISFACTORY.
A 32	3	2	FREE DRAINING GRANULAR	GRANULAR SOIL LITTLE	35	35	2080	6	3.6	1.8	NO	0.0	5548	5528	WHEN FOUNDATION SOIL HAS φ LESS THAN 30° SLIDING KEY
A 53	5	3	SOIL OR BROKEN ROCK	SILT OR CLAY, COHESION	35	35	2080	6	2.9	1.5	NO	0.1	9946	7500	OR IMBEDMENT OF WALL AT LEAST 1.5m BELOW GRADE IS
A 63	6	3	W/SOME FINES GW, GP, SW,	NEGLIGIBLE.	35	35	2080	6	2.1	3.1	YES	0.3	19000	4165	REQUIRED FOR SAFETY AGAINST SLIDING.
A 64	6	4	SP	"	35	35	2080	6	3.3	3.2	YES	-0.1	7561	10615	
A 32	3	2	ROCK OR FIRM LOW	SILTY SAND OR GRAVEL	28	28	2080	6	3.9	0.K	YES	0.1	7515	3348	
A 43	4	3	PLASTICITY SOIL GC,	LOW PLASTICITY SILT OR	28	28	2080	6	3.3	3.3	YES	0.0	5920	6012	
A 53	5	3	GM, SC, SM, SHALE.	CLAY SM, GM, CL, ML.	28	28	2080	6	2.5	2.3	YES	0.3	13870	3192	
A 64	6	4	"	"	28	28	2080	6	2.6	1.5	YES	0.2	11415	6334	NOTE: TYPE C WALL MAY BE SUBSTITUTED FOR TYPE A WALL IN THIS SOIL, SPECIAL KEY REQUIRED FOR ALL CONFIGURATIONS; NOT RECOMMENDED. WIRING FOR SP. KEY IS MARGINAL.
B 32	3	2	BROKEN OR IN PLACE	FREE DRAINING GRANULAR	45	45	1600	6	4.9	4.1	NO	-0.1	6154	9215	
B 53	5	3	SOUND ROCK	SOIL OR BROKEN ROCK	45	45	1600	6	4.0	3.7	NO	0.0	14942	14486	
B 63	6	3	"	GW, GP, SW, SP.	45	45	1600	6	3.0	3.3	NO	0.2	22705	11916	
B 32	3	2	"		45	45	1600	0	3.9	2.9	NO	0.0	9393	8919	
B 53	5	3	"		45	45	1600	0	3.3	2.8	NO	0.1	22724	14255	
B 63	6	3	"		45	45	1600	0	2.5	2.5	NO	0.3	34741	7692	
B 64	6	4	"		45	45	1600	0	4.1	3.1	NO	0.0	26373	23235	
B 32	3	2	FREE DRAINING GRANULAR	GRANULAR SOIL, LITTLE	35	35	2080	6	3.4	1.8	NO	0.1	9657	6989	
B 53	5	3	SOIL OR BROKEN ROCK	SILT OR CLAY, NEGLIGIBLE	35	35	2080	6	2.8	1.6	NO	0.2	23005	8618	
B 63	6	3	W/SOME FINES, GW, GP, SW,	COHESION.	35	35	2080	6	2.1	1.5	NO	0.5	36292	1732	UNBALANCED FOUND. PRESSURE e > B/6
B 64	6	4	SP.		35	35	2080	6	3.5	1.8	NO	0.2	25747	14509	
B 32	3	2	ROCK OR FIRM LOW	SILTY SAND OR GRAVEL	28	28	2080	6	3.9	0.K	YES	0.1	11360	4389	NOTE: TYPE C MAY BE USED INPLACE OF TYPE B IN THIS SOIL. SPECIAL KEY REQUIRED FOR ALL CONFIGURATIONS.
B 53	5	3	PLASTICITY SOILS SC, GC,	LOW PLASTICITY SILT OR	28	28	2080	6	2.3	2.7	YES	0.4	26228	2833	
B 64	6	4	SM, GM, SHALE.	CLAY SM, GM, CL, ML.	28	28	2080	6	2.7	2.0	YES	0.4	28908	7029	
C 54	5	4	"	"	28	28	2080	6	3.6	2.1	YES	0.0	10868	9680	
C 65	6	5	"	"	28	28	2080	6	3.8	1.5	YES	0.0	11615	11598	
GABION WALLS FOR NO SURCHARGE LOADS															
A 32	3	2	ROCK OR FIRM LOW	SILTY SAND OR GRAVEL	28	0	2080	6	6.1	3.2	NO	-0.1	3493	5581	
A 53	5	3	PLASTICITY SOILS GM, SM,	LOW PLASTICITY SILT OR	28	0	2080	6	4.6	2.3	NO	-0.1	3221	9471	
A 64	6	4	GC, SC.	CLAY SM, GM, CL, ML.	28	0	2080	6	5.3	2.2	NO	-0.3	3345	10775	
B 32	3	2	"	"	28	0	2080	6	3.1	2.8	NO	-0.1	6245	4407	
B 53	5	3	"	"	28	0	2080	6	3.7	2.2	NO	0.2	13121	4434	
B 64	6	4	"	"	28	0	2080	6	4.0	2.2	NO	0.4	15241	4438	
G 22	2	2	"	"	28	0	2080	6	15.9	9.8	NO	-0.1	2670	4810	
G 32	3	2	"	"	28	0	2080	6	6.7	4.2	NO	0.0	4845	8295	

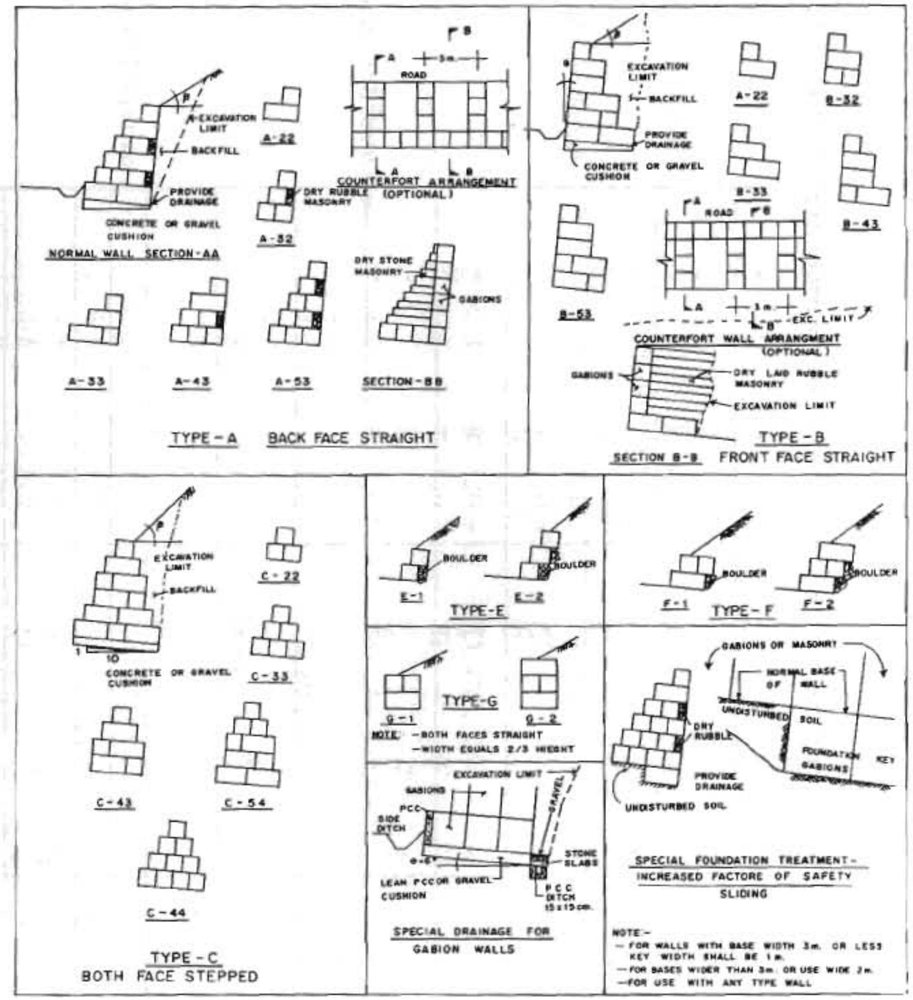
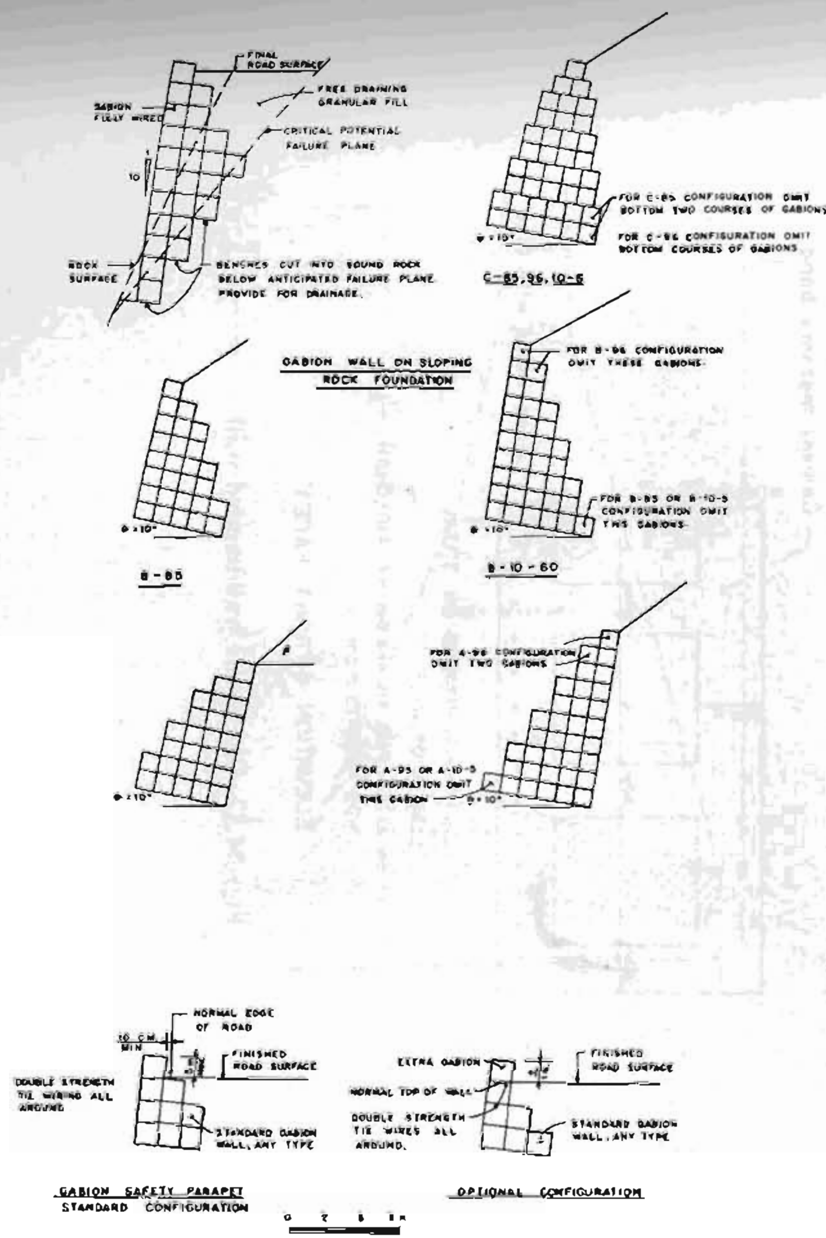


Fig. 24.35 Standard gabion walls



EXTRA HEIGHT GABION WALLS FOR SLOPING SURCHARGE

WALL TYPE	HEIGHT (M)	BASE WIDTH (M)	FOUNDATION MATERIAL	SUPPORTED SOIL TYPE: DESCRIPTION	φ DEG	δ DEG	MOIST UNIT WT KG/m³	α DEG	F.S. TURNING	F.S. SLIDING	SPEC. KEY	e	TOE PRESSURE KG/m²	HEEL PRESS KG/m²	REMARKS
A 63	6	3	SOUND ROCK IN PLACE OR	FREE DRAINING GRANULAR	45	45	1600	10	4.8	6.3	NO	-0.3	3304	4079	
A 64	6	4	BROKEN	SOIL OR BROKEN ROCK GW,	45	45	1600	10	7.8	7.0	NO	-0.7	4441	15947	NOT RECOMMENDED, NEG TOE PRESSURE, OK AT α ≤ 6°
A 85	8	5	"	GP, SW, SP.	45	45	1600	10	6.9	7.4	NO	-0.7	1189	21064	PREFER α < 10°
A 10-6	10	6	"	"	45	45	1600	8	5.9	3.3	NO	-1.0	333	24869	NOT RECOMMENDED FOR α > 5° DUE TO PRESSURE DISTRIBUTION
B 63	6	3	"	"	45	45	1600	10	3.3	4.3	NO	-0.5	10644	16587	
B 64	6	4	"	"	45	45	1600	10	3.8	3.5	NO	-0.2	14587	24633	
B 85	8	5	"	"	45	45	1600	10	3.9	7.3	NO	-0.2	31216	45230	
B 10-6	10	6	"	"	45	45	1600	10	3.5	6.9	NO	-0.04	43448	50045	
C 54	5	4	"	"	45	45	1600	10	8.4	3.5	NO	-0.3	3208	17103	
C 65	6	5	"	"	45	45	1600	10	8.5	3.2	NO	0.6	2858	20638	
C 85	8	5	"	"	45	45	1600	10	3.3	4.7	NO	-0.4	11739	30101	
C 10-6	10	6	"	"	45	45	1600	10	4.8	4.3	NO	-0.4	16732	36090	
A 63	6	3	GRANULAR SOIL, FREE	GRANULAR SOIL, LITTLE	35	35	2080	10	2.8	1.7	NO	0.1	13557	8371	
A 64	6	4	DRAINING, BROKEN ROCK	SILT OR CLAY, NEGLIGIBLE	35	35	2080	10	4.2	1.8	NO	-0.3	4411	12626	
A 85	8	5	W/FINES	COHESION	35	35	2080	10	3.8	1.8	NO	-0.2	9028	15804	
A 10-6	10	6	"	"	35	35	2080	10	3.1	1.5	NO	-0.2	11098	17321	
B 63	6	3	"	"	35	35	2080	10	2.5	1.8	NO	0.3	31389	7614	
B 64	6	4	"	"	35	35	2080	10	4.1	2.2	NO	0.1	22346	18721	
B 85	8	5	"	"	35	35	2080	10	4.1	2.4	NO	0.2	43068	30417	
B 10-6	10	6	"	"	35	35	2080	10	3.6	2.5	NO	0.4	64575	30059	
C 54	5	4	"	"	35	35	2080	10	3.8	2.1	NO	-0.3	6570	15786	
C 65	6	5	"	"	35	35	2080	10	6.0	2.0	NO	-0.4	6735	16580	
C 85	8	5	"	"	35	35	2080	10	3.4	1.7	NO	0.3	17799	16440	
C 10-6	10	6	"	"	35	35	2080	10	3.0	1.6	NO	0.2	23189	16407	
A 63	6	3	SOFT ROCK OR FIRM LOW	SILTY TO CLAYEY SAND &	28	28	2080	10	2.0	1.4	YES	0.4	20204	5694	SLIDING F.S. MARGINAL
A 64	6	4	PLASTICITY SOIL SC, CL,	GRAVEL LOW PLASTICITY	28	28	2080	10	3.3	1.5	YES	0.0	8258	8804	
A 85	8	5	SM, GM	SILT & CLAY SHALE FRAGMENTS	28	28	2080	10	2.9	1.5	YES	0.2	14338	9918	
A 10-6	10	6	"	"	28	28	2080	10	2.2	1.1	YES	0.3	18397	9490	NOT SATISFACTORY FOR THIS FOUND SOIL (α = 20°)
B 63	6	3	"	"	28	28	2080	10	2.0	2.2	YES	0.5	33774	4761	NOT RECOMMENDED - PRESSURE DIST
B 64	6	4	"	"	28	28	2080	10	4.2	2.8	YES	0.3	23544	9387	
B 85	8	5	"	"	28	28	2080	10	3.0	2.0	YES	0.4	34985	11798	
B 10-6	10	6	"	"	28	28	2080	10	2.6	1.6	YES	0.7	49182	7933	
C 54	5	4	"	"	28	28	2080	10	4.7	2.9	YES	-0.1	8304	10812	
C 65	6	5	"	"	28	28	2080	10	4.9	2.1	YES	-0.1	9091	12708	
C 85	8	5	"	"	28	28	2080	10	2.8	1.7	YES	0.3	22379	10104	
C 10-6	10	6	"	"	28	28	2080	10	2.8	1.4	YES	0.5	31475	9924	SLIDING F.S. MARGINAL

GENERAL NOTES

1. BASE INCLINATION α SHOWN AT 10°, CONSTRUCTION AT 6° GENERALLY SATISFACTORY CHECK SLIDING F.S.
2. FOUNDATION PRESSURES ARE GENERALLY HIGH, SITE SHOULD BE VERIFIED AS ABLE TO SUPPORT THE LOAD BEFORE CONSTRUCTING WALLS.
3. WHEN FOUNDATION SOIL HAS φ LESS THAN 30°, SLIDING KEY OR INSECONSET OF WALL AT LEAST 1M BELOW GRADE ARE REQUIRED FOR SAFETY AGAINST SLIDING.

EXTRA HEIGHT GABION WALLS FOR NO SURCHARGE LOADS

WALL TYPE	HEIGHT (M)	BASE WIDTH (M)	FOUNDATION MATERIAL	SUPPORTED SOIL TYPE: DESCRIPTION	φ DEG	δ DEG	MOIST UNIT WT KG/m³	α DEG	F.S. TURNING	F.S. SLIDING	SPEC. KEY	e	TOE PRESSURE KG/m²	HEEL PRESS KG/m²	REMARKS
A 85	8	5	ANY FOUNDATION LISTED	ANY BACKFILL LISTED	28	0	2080	10	6.3	3.5	NO	-0.5	3499	15363	
A 10-6	10	6	ABOVE	ABOVE	28	0	2080	10	5.3	2.5	NO	-0.7	5293	18754	
B 85	8	5	"	"	28	0	2080	10	4.7	3.0	NO	0.2	17206	9910	
B 10-6	10	6	"	"	28	0	2080	10	3.8	2.4	NO	0.5	23946	7433	
C 85	8	5	"	"	28	0	2080	10	3.2	2.9	NO	-0.1	9938	12921	
C 10-6	10	6	"	"	28	0	2080	10	4.4	2.6	NO	-0.1	15260	14836	

Fig. 24.36 Extra height gabion wall

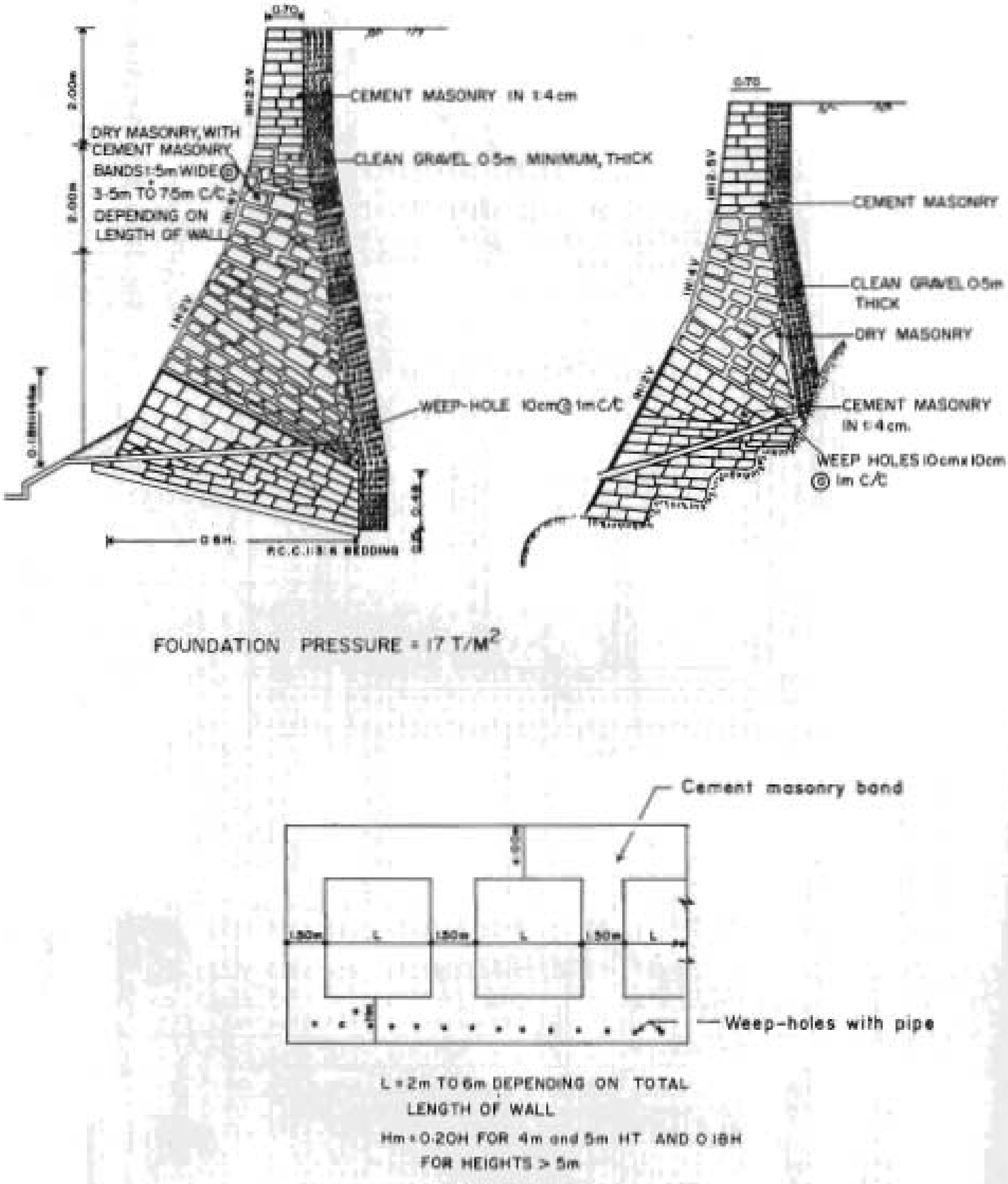


Fig. 24.37 Composite wall/banded wall

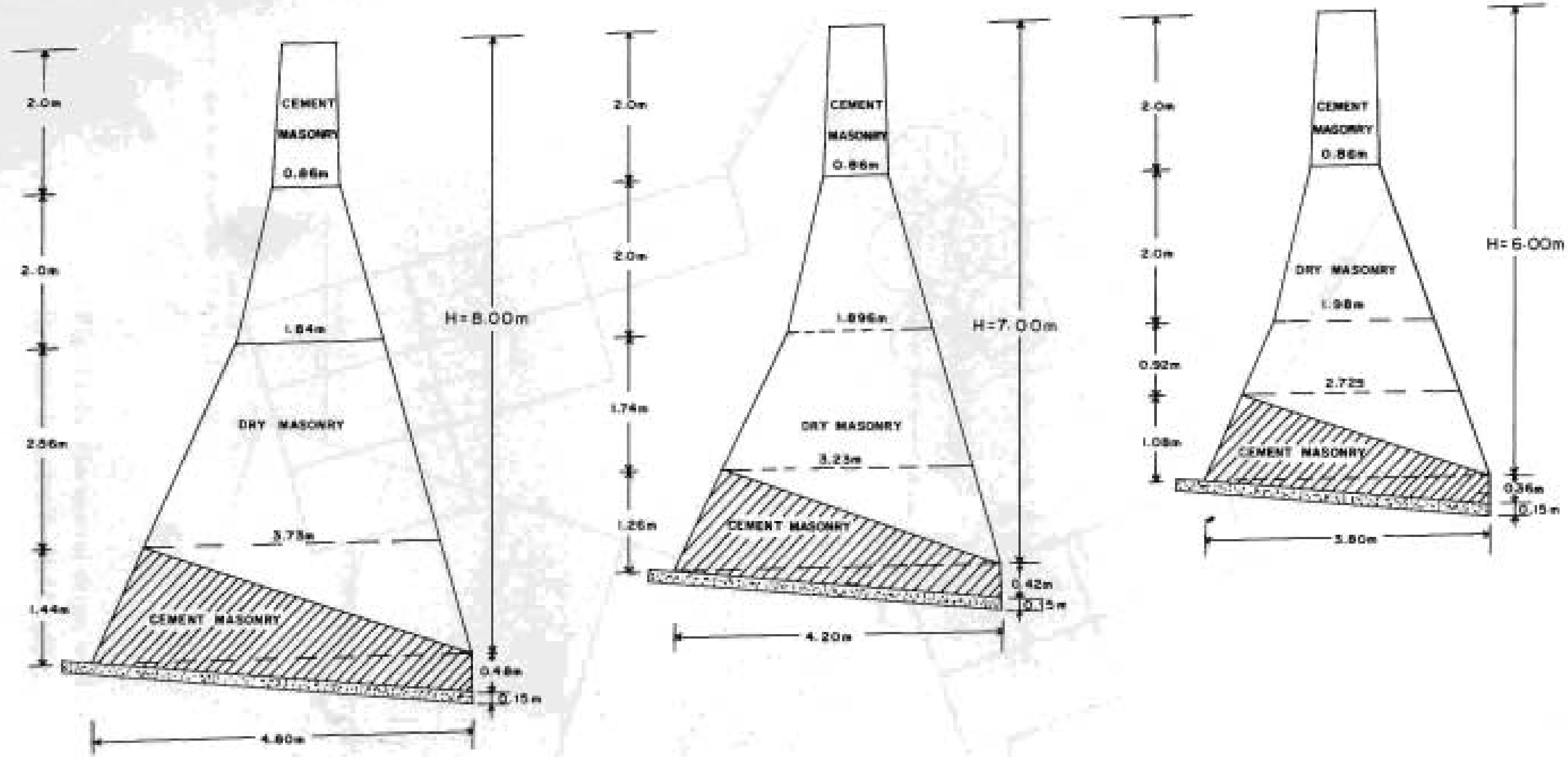


Fig. 24.38 Details of composite walls

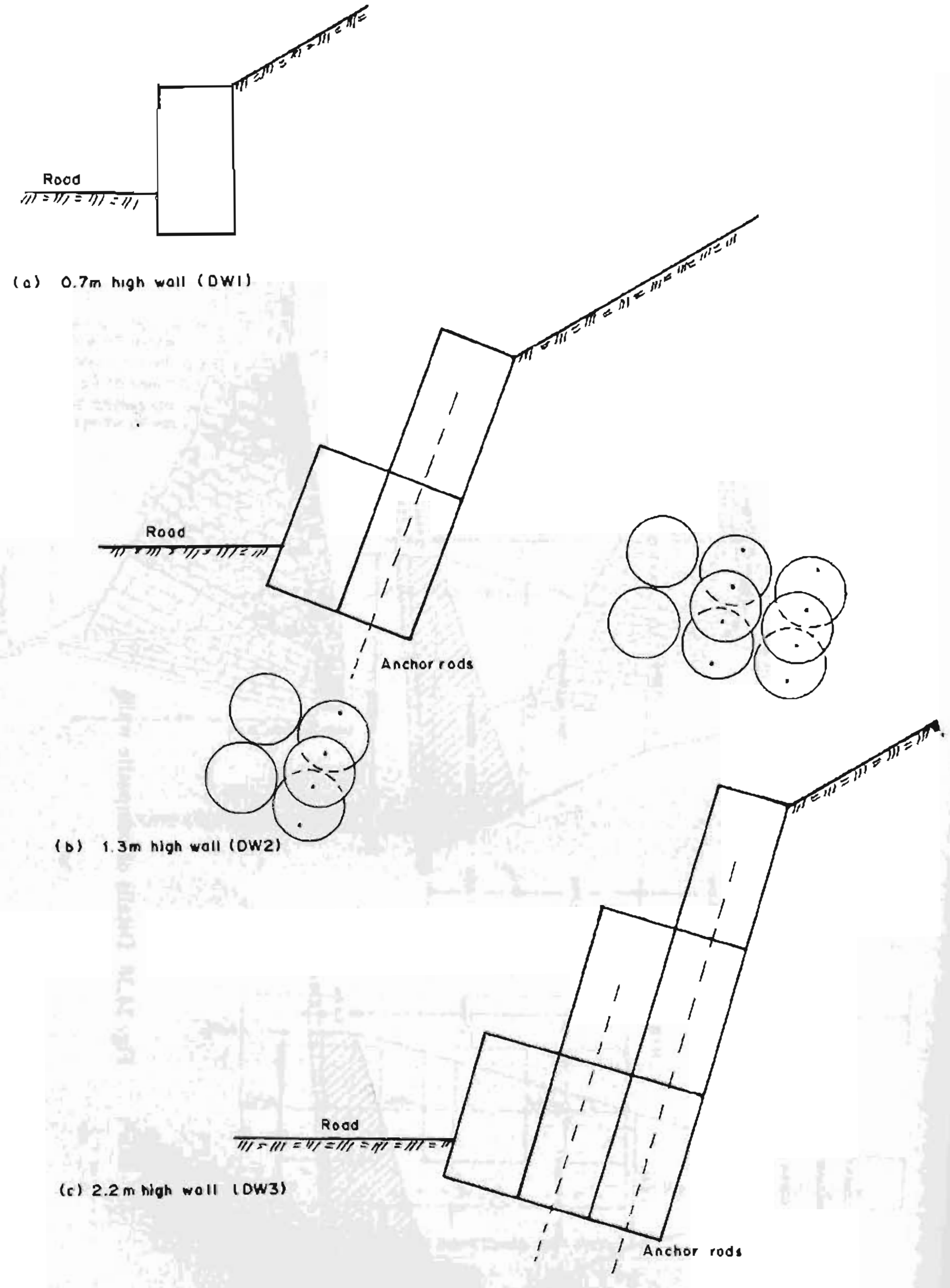


Fig. 24.39 Vertical drum wall

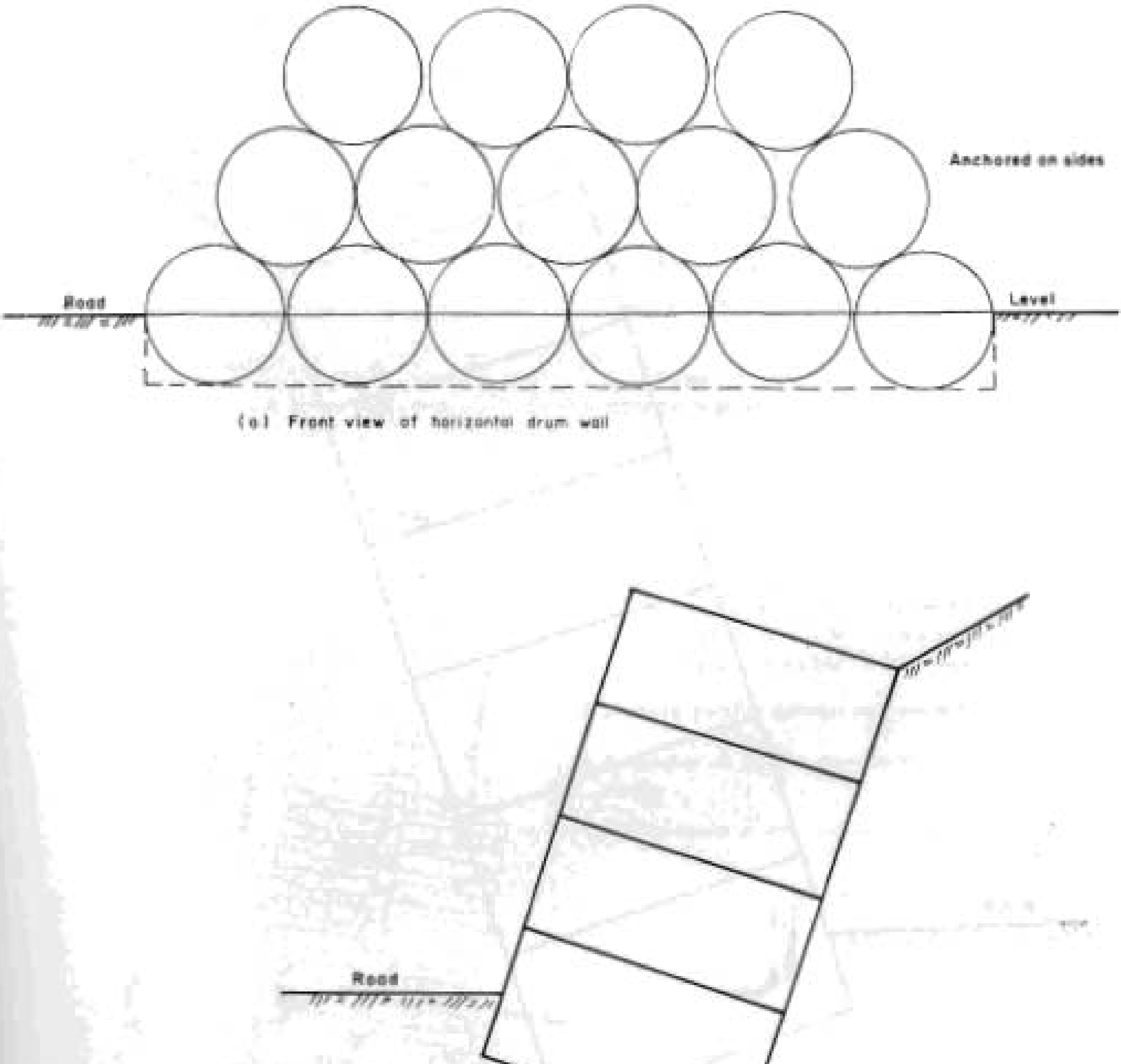


Fig. 24.40 Horizontal drum wall

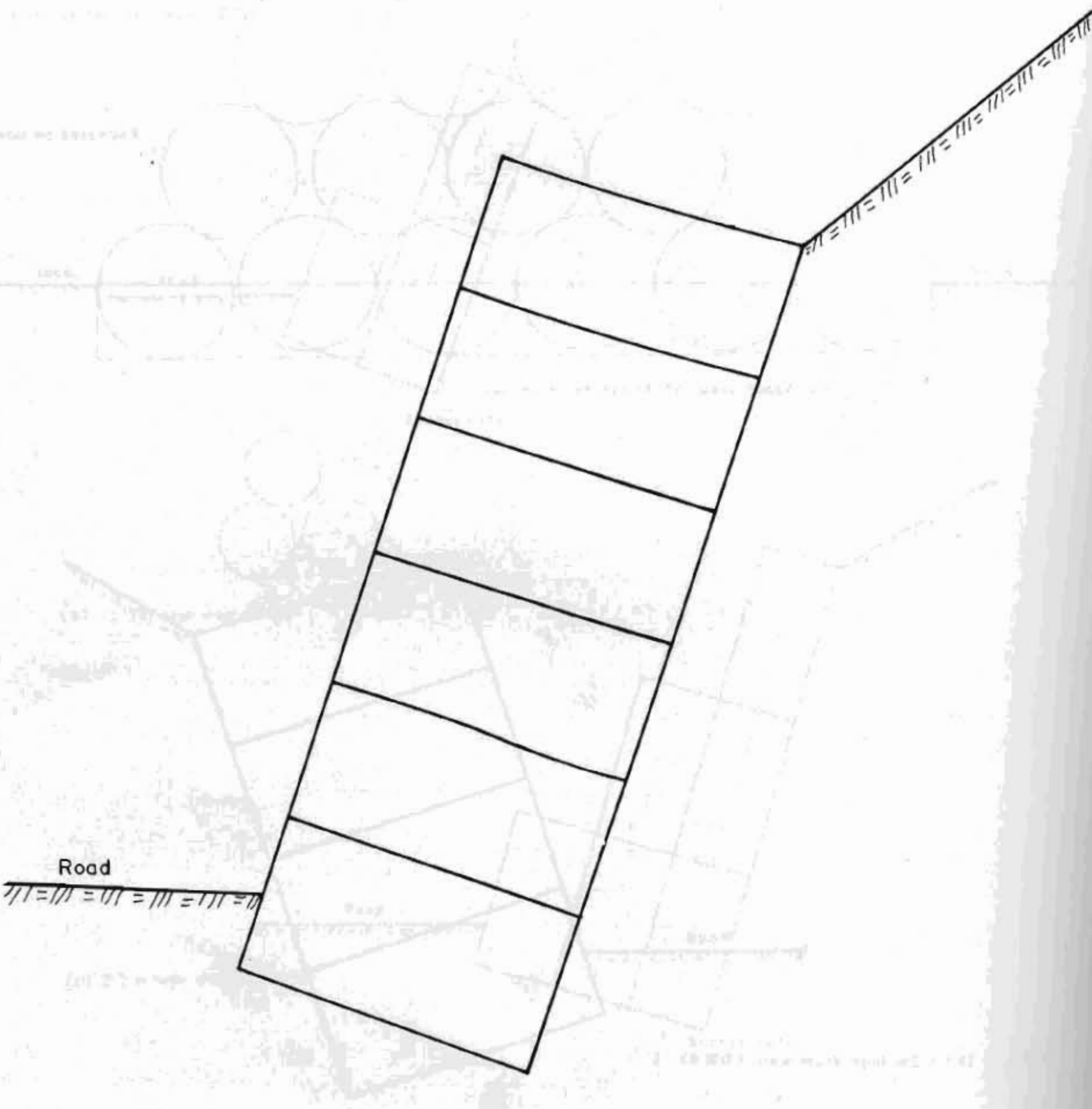
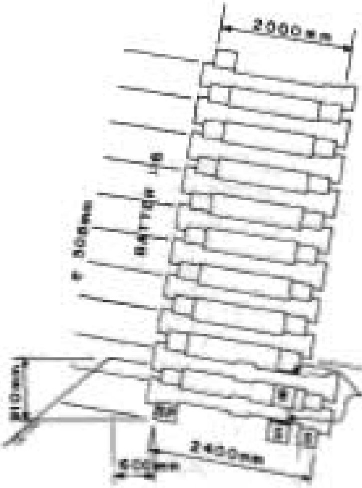
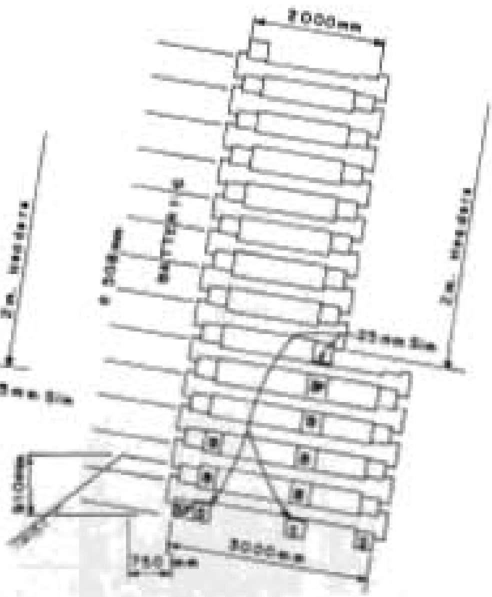


Fig. 24.41 Horizontal drum wall of 2.2 m height



TYPE I
(4 to 5m High wall)



TYPE II
(5 to 6m. High wall)

TYPE III

NOTES :-

1. Allowable bearing capacity
   Type I = $\nless$ 33 T/M^2
   Type II = $\nless$ 33 T/M^2
   Type III = $\nless$ 43 T/M^2
   } Double the bearing Capacity for walls With sloping backfill

2. Structure backfill material only should be used

3. Compaction of backfill should be 95% of standard proctor

4. All dimensions in mm unless otherwise stated

Fig. 24.42 (A) RCC crib wall

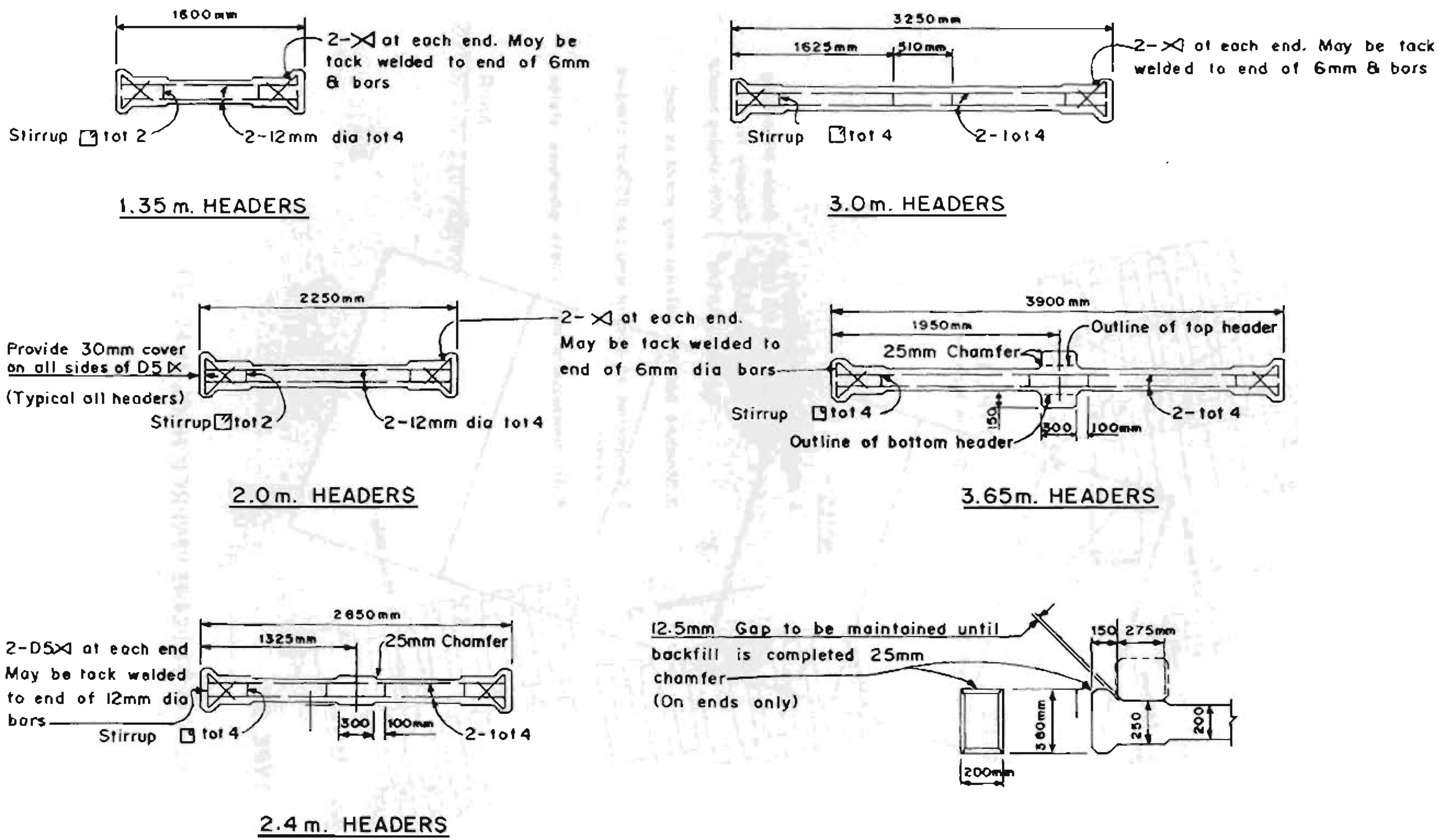
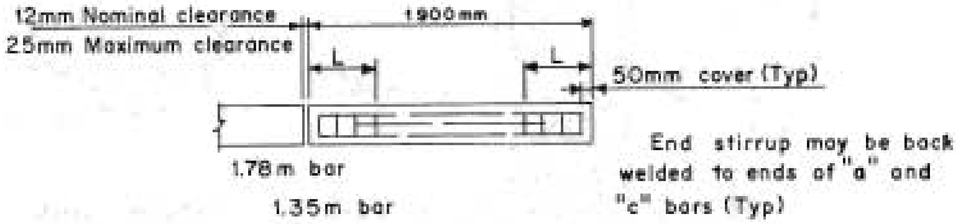
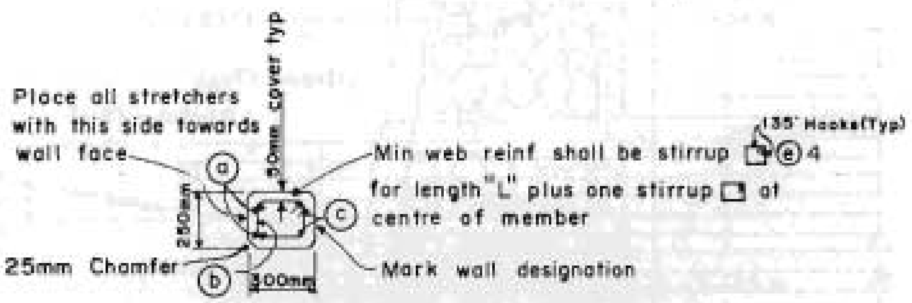


Fig. 24.43 (B) Header details for RCC crib wall



1.9 m. STRETCHER

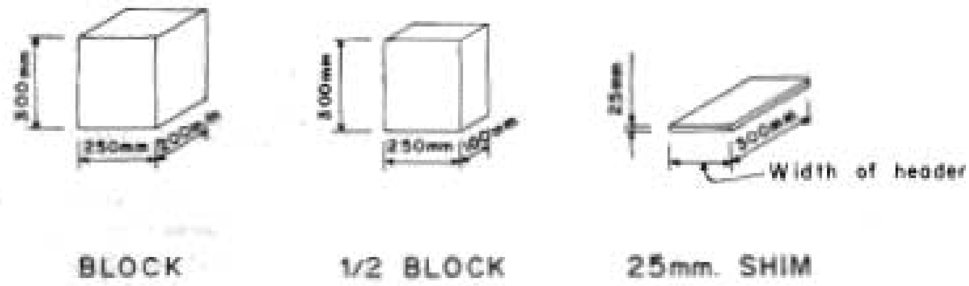
3.80m. STRETCHER



Section

STRETCHER BAR SIZE DATA DIA. IN mm.									
Wall Type	Design f'c psi	Open Face						L	
		1900 mm.			3800 mm.			1900	3800
		a	b	c	a	b	c		
I-II	3250	12	12	12	19		16	250 mm.	250 mm.
III	4000	12	12	12	19	19	19	355 mm.	660 mm.

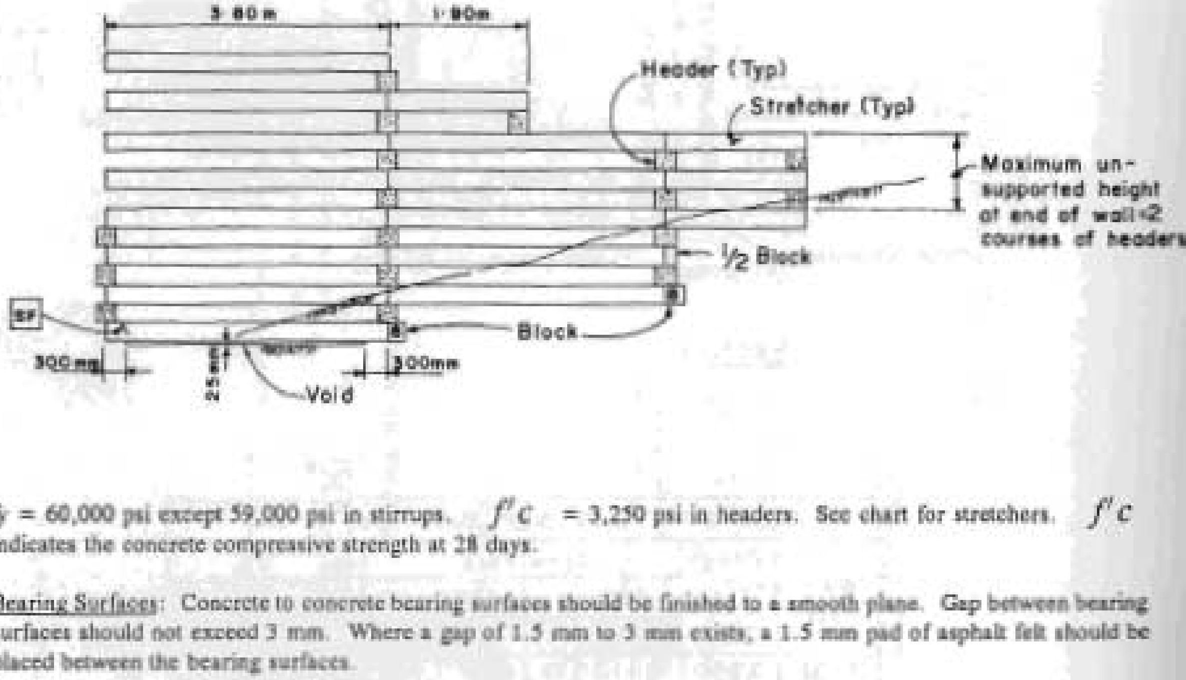
Fig. 24.44 (C) Stretcher details for RCC crib wall



S — Foundation stretchers placed thus (Mark, 250mm, 300) on a 100mm layer of loose structure backfill material. Details and reinforcement are the same as other strechers.

B — Blocks - a block is required immediately behind the front stretchers on each front header for a minimum of two courses above the foundation stretcher for Type II & III

SF — Front face foundation stretcher, place as shown below.



NOTES

1. fy = 60,000 psi except 59,000 psi in stirrups. $f'c$ = 3,250 psi in headers. See chart for stretchers. $f'c$ indicates the concrete compressive strength at 28 days.

2. Bearing Surfaces: Concrete to concrete bearing surfaces should be finished to a smooth plane. Gap between bearing surfaces should not exceed 3 mm. Where a gap of 1.5 mm to 3 mm exists, a 1.5 mm pad of asphalt felt should be placed between the bearing surfaces.

3. Special Members at Utility Openings: Where an opening is specified in the face of the wall for pipes or other utilities, special length stretchers and additional headers will be required.

Fig. 24.45 Block and stretcher placement details for RCC crib wall

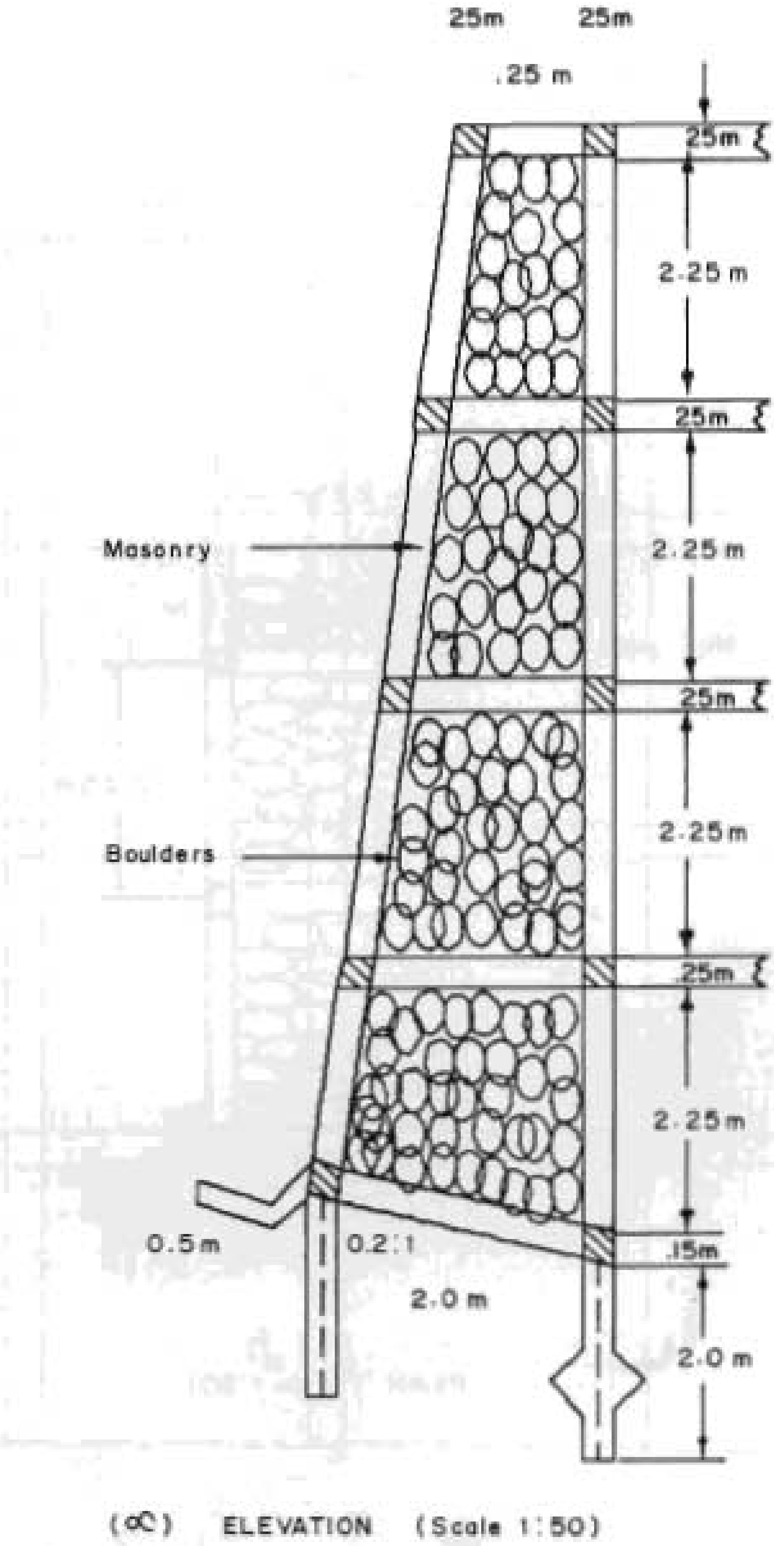


Fig. 24.46 RCC frame retaining wall

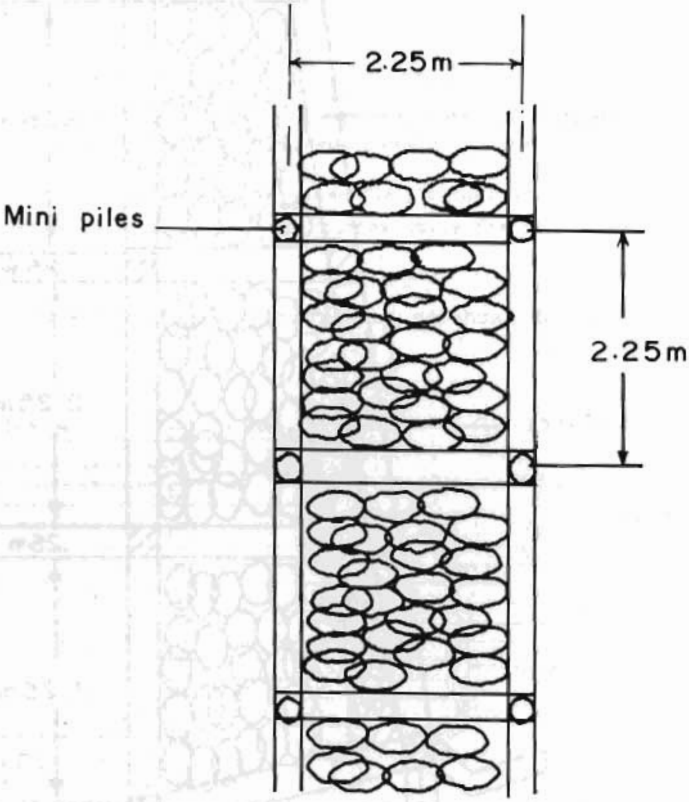


Fig. 24.47 RCC frame retaining wall

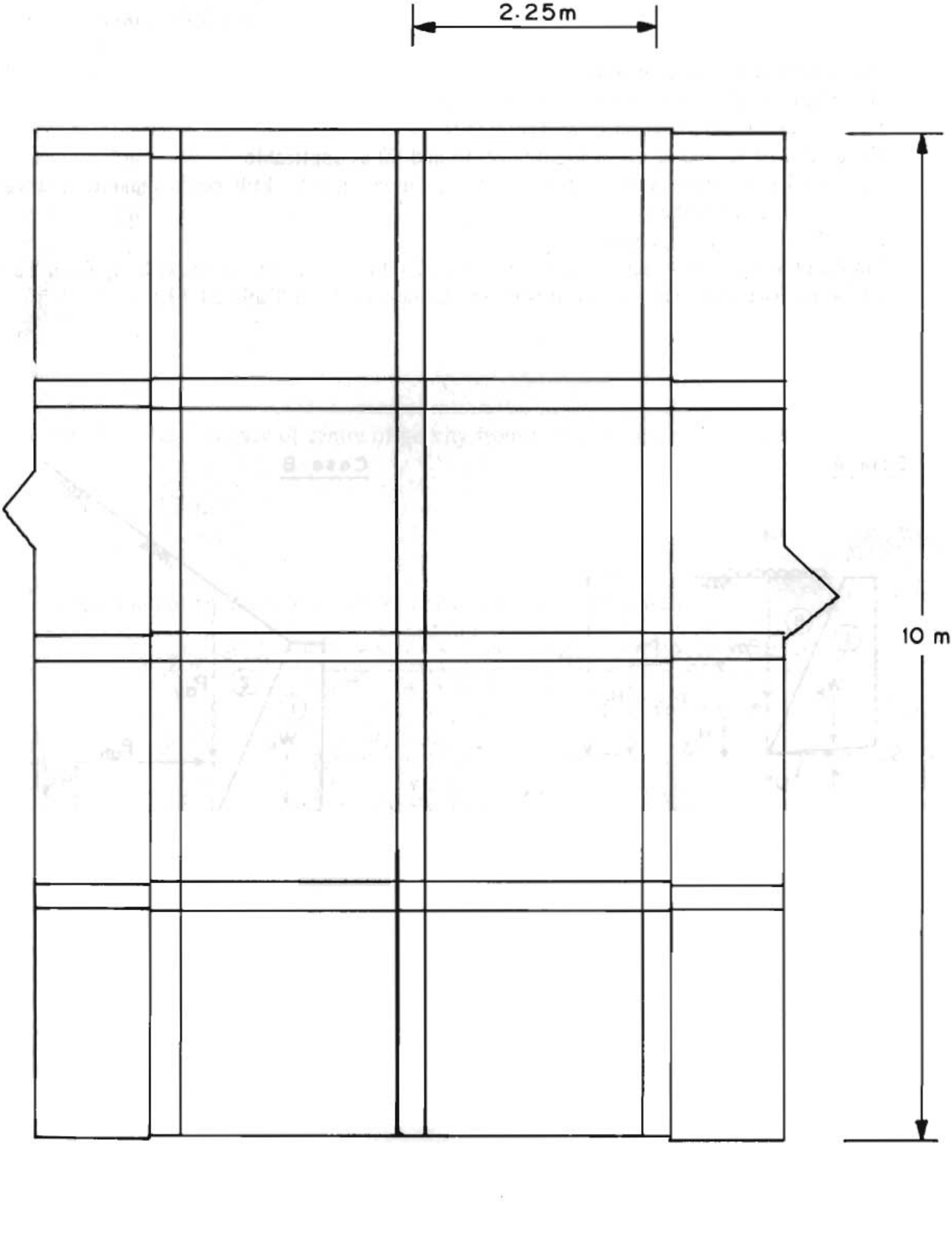
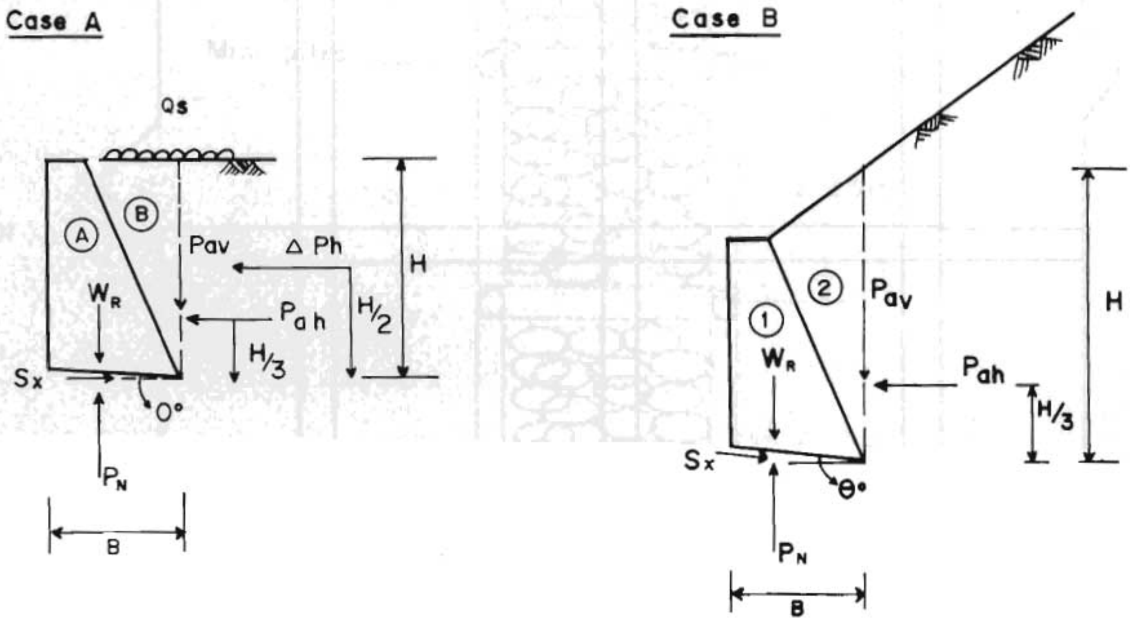


Fig. 24.48 Frame retaining wall

24.6.3 *Semi-Empirical Design of Retaining Walls (Dry Backfill and No Earthquakes)*

- o Select trial dimensions of walls.
- o Ascertain backfill geometry from Table 24.10
- o Select backfill classification from Table 24.9
- o Find value of K_r and K_v from Figures 24.49 and 50 as applicable.
- o Find surcharge intensity Q_s if any. Surcharge for inclined backfill needs separate treatment and is not described here.
- o Find Θ from selected geometry.
- o Find unit weight of structure r_m from Tables 24.14(b) unit wt. of backfill, r_s from Table 24.14(a), frictional factor between wall and foundation from Table 24.13.



Calculate P_{ah} ΔP_h:

$$P_{ah} = \frac{1}{2} K_h H^2$$

$$\Delta P_h = C \times Q_s \quad .$$ (C from Table 24.11)

Calculate P_{av}:

$$P_{av} = \frac{1}{2} K_v H^2 \quad . \qquad (P_{av} = 0 \text{ for horizontal backfill})$$

Calculate weight of structure and backfill:

w_1 , w_2 , w_3 .... w_n

$$W_R = W_1 \ W_2 + W_3 + ...W_n + P_{av} \quad .$$

Calculate lever arm (distance of centre of gravity from toe), for each block of weight:

l_1 , l_2 ,.......... l_n .

Calculate position of resultant of all forces at base from the toe of wall:

$$\bar{x} = \frac{[(w_1 \ l_1 + w_2 \ l_2 + w_n \ l_n) + P_{av} \times B] - [P_{ah} \times H/3 + \Delta P_h \times H/2]}{(w_1 + w_2 + ... w_n + P_{av})}$$

$$= \frac{M_R - M_o}{P_N}$$

$$P_N = (W_R + P_{av}) \cos\theta + (P_{ah} + \Delta P_h) \sin\theta$$

= *normal force on base* .

Calculate eccentricity along base of wall:

$$e = (\frac{B}{2} - \bar{x}) \sec\theta$$

$$|e| < \frac{B \sec\theta}{6} \quad (\textit{absolute value}) \quad .$$

Calculate sliding and resisting forces at base of wall:

Sliding force:

$$S_f = (P_{ah} + \Delta P_h)\cos\theta - (W_R + P_{av})\sin\theta \ .$$

Resisting frictional force:

$$S_r = friction\ factor \times P_N$$

$$= fP_N \ .$$

Calculate maximum pressure at base:

$$P_{\max,\ \min} = \frac{P_N}{B\ \sec\theta}\left(1 \pm \frac{6e}{B\ \sec\theta}\right) + Q_s \ .$$

Check for factor of safety:

$$Overturning = \frac{Resisting\ Moment}{Overturning Moment} = \frac{M_R}{M_o}\ (\geq 2.0)$$

$$Sliding = \frac{S_r}{S_f}\ (\geq 1.5) \ .$$

Base pressure < allowable bearing capacity:

$$\sim \frac{Ultimate\ bearing\ Capacity}{Maximum\ pressure\ at\ base} \geq 2 \ .$$

Table 24.9 Type of backfill for retaining walls

1.	Coarse-grained soil without admixture of fine soil particles, very permeable (clean sand or gravel).
2.	Coarse-grained soil of low permeability due to admixture of particles of silt size.
3	Residual soil with stones, fine silty sand, or granular materials with conspicuous clay content.
4.	Very soft or soft clay, or organic salts, or silty clays.
5.	Medium or stiff clay, deposited in chunks and protected in such a way that a negligible amount of water enters the spaces between the chunks during floods or heavy rains. If this condition cannot be satisfied, the clay should not be used as backfill material. With increasing stiffness of the clay, danger to the wall due to infiltration of water increases rapidly.

Source: Terzaghi and Peck 1967

Table 24.10 Classification: backfill geometry and surcharge conditions

1.	The surface of the backfill is plane, i.e., horizontal or sloped upwards from the crest of the wall.
2.	The surface of the backfill is sloped upwards from the crest of the wall but becomes level at some elevation above the crest.
3.	The backfill is surcharged.

Source: Terzaghi and Peck 1967

Table 24.11 Values of C, coefficient for horizontal load due to uniform surcharge

Type of Soil **	C
1	0.27
2	0.30
3	0.39
4	1.00
5	1.00

Source: Terzaghi and Peck 1967

* From Table 24.9

Table 24.12: Allowable bearing capacities for different types of soil

Type of Bearing Material	Consistency in Place	Recommended Value of Allowable Bearing Capacity Ton T/m^2
Well-graded mixture of fine and coarse-grained soil; glacial till, hardpan, boulder clay (GW-GC-, GC, SC)	Very compact	100
Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW, GP, SW, SP)	Very compact	80
	Medium to compact	60
	Loose	40
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact	40
	Medium to compact	30
	Loose	30
Fine to medium san, silty or clayey medium to coarse sand, (SW, SM, SC)	Very compact	30
	Medium to compact	25
	Loose	15
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very compact	30
	Medium to compact	20
	Loose	15
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard	40
	Medium to stiff	20
	Soft	5
Inorganic silt, sandy or clayey silt, varied silt-clay-fine sand (ML, MH)	Very stiff to hard	30
	Medium to stiff	15
	Soft	5

Table 24.13 Friction factor for dissimilar materials

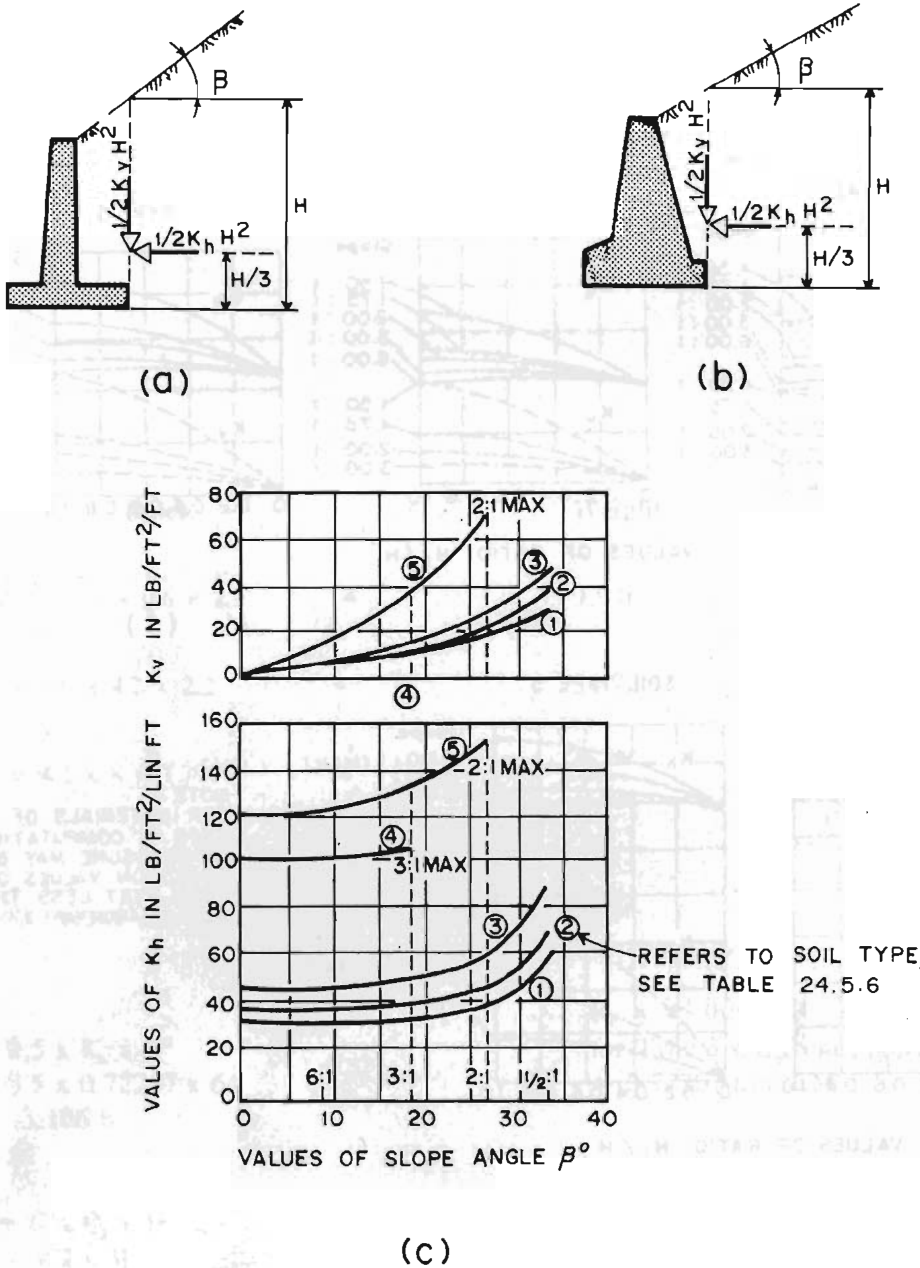
Interface Materials	Friction Factor $\tan\phi$ or f_f	Friction angle ϕ Degrees	Adhesion CA, PSF
Masonry on the Following Materials			
Clean sound rock	0.70	35	
Clean gravel, gravel-sand mixtures coarse sand	0.55 to 0.60	29 to 31	
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29	
Clean fine sand, silty or clayey fine to medium sand................	0.35 to 0.45	19 to 24	
Fine sandy silt, non-plastic silt	0.30 to 0.35	17 to 19	
very stiff and hard residual or preconsolidated clay............	0.40 to 0.50	22 to 26	
Medium stiff and stiff clay and silty clay.....................	0.30 to 0.35	17 to 19	
Masonry on Masonry, Igneous and Metamorphic Rock			
Dressed soft rock on dressed soft rock..	0.70	35	
Dressed hard rock on dressed soft rock..	0.65	33	
Dressed hard rock on dressed hard rock..	0.55	29	
Masonry on wood (cross grain)...........	0.50	26	
Steel on steel at sheet pile interlocks.	0.30	17	

Table 24.14(a) Friction factor and unit weight of compacted material

Group Symbol	Soil Type	Range of Max. Dry Unit Weight T/m^3	Typical Strength Characterisitics
			ϕ Effective angle of friction (degrees)
GW	Well-graded clean gravels, gravel-sand mix.	2.00 - 2.15	> 38
GP	Poorly-graded clean gravels, gravel-sand mix	1.85 - 2.00	> 37
GM	Silty gravels, poorly-graded gravels-sand silt	1.90 - 2.15	> 34
GC	Clayey gravels, poorly-graded gravel-sand clay	1.75 - 2.10	> 31
SW	Well-graded clean sands, gravelly sand	1.60 - 1.90	38
SP	Poorly-graded clean sands, sand-gravel mix	1.60 - 1.90	37
SM	Silt sands, poorly graded sand-silt mix	1.75 - 2.00	34

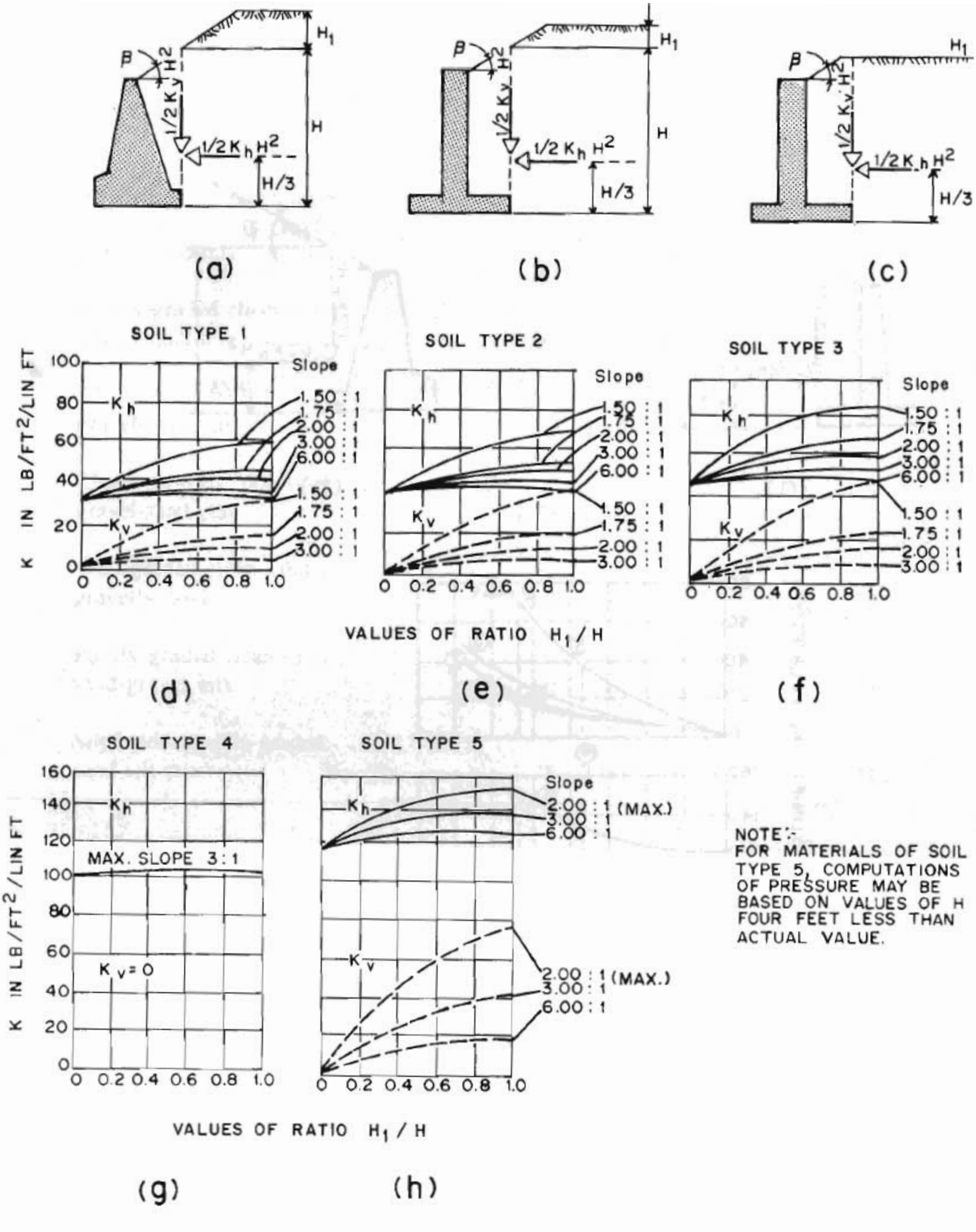
Table 24.14(b) Unit weight of walls

Type of Wall	Unit Weight of Masonry for Various Rock Types, T/m^3			
	Brick	Granite	Limestone	Sandstone
Dry Masonry	1.38	1.88	1.74	1.53
Gabion Masonry	1.28	1.88	1.74	1.53
Cement Masonry	1.79	2.44	2.66	1.99



Source: Terzaghi and Peck 1967

Fig. 24.49 Backfill coeffients - plane backslope



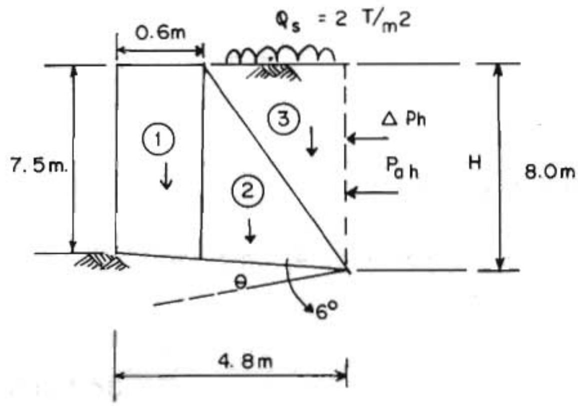
Source: Terzaghi and Peck 1967

Figure 24.50 Backfill geometries and coefficients - transitioned backfill

24.6.3(a) *Example of Retaining Wall Design Using Semi-empirical Method*

Case I
Backfill

- Type 3 soil
- Horizontal
- Unit Wt. = 1.77 t/m³



Block		Weight	Moment
1. $\frac{7.5 + 7.56}{2} \times 0.6 \times 2.2$	=	9.94t	2.98tm
2. $\frac{1}{2} \times 7.56 \times 4.2 \times 2.2$	=	34.93t	69.86tm
3. $\frac{1}{2} \times 4.2 \times 8 \times 1.77$	=	29.74t	101.10tm
		74.61t	173.94tm

Resisting moment, $\sum M_R = (2.98 + 69.86 + 101.10) = 173.94$

P_{ah}
$= 0.5 \times K_h \times H^2$
$= 0.5 \times 0.72207 \times 64$
$= 23.106$ t

$K_h = 45$ lb/ft²/LF
for Type 3 soil from Figure. 24.49
= 0.72207 t/m²/LM.

$\Delta P_h = C \times Q_s \times H$
$= C \times 2 \times H$
$= 0.39 \times 2 \times 8$
$= 6.24$ t

C = 0.39
for Type 3 soil from Table 24.11.

Overturning moment about toe of the wall:

$$\sum M_o = 23.106\ (\frac{8}{3} - 4.8\ \tan\theta) + 6.24 \times (\frac{8}{2} - 4.8\ \tan\theta)$$

$$= 71.77\ tm\ .$$

Normal force at base of wall:

PN = 74.61 cosΘ + (23.106 + 62.4) sinΘ = 77.27t.

Eccentricity:

$$\bar{x} = \frac{(173.93\ -\ 71.77)}{77.27} = 1.32m$$

$$e = (2.4 - \frac{103.16}{77.27})\ \sec\theta = (2.4 - 1.32)\ \sec\theta = 1.09 > 0.8M\ .$$

Maximum toe pressure, P_{max} or P_{min}:

$$= \frac{N}{B\ \sec\theta}\ (1 \pm \frac{6e}{B\ \sec\theta}) + surcharge$$

$$= \frac{77.27}{4.8 \times 1.005}\ (1 + \frac{6 \times 1.09}{4.8 \times 1.005}) + 2$$

$$= 16.02 \pm 21.72 + 2\ t/m^2$$

$$P_{max} = 39.74\ t/m^2$$

$$P_{min} = -\ 3.7\ t/m^2\ (may\ be\ neglected\ at\ heel)$$

Frictional resistance, S_R:

= 0.40 P_N
= 77.27 x 0.4
= 30.91.

Sliding force, S_f:

$$= (\Delta P_h + P_{ah}) \cos\theta - W_R \, Sin\theta$$

$$= (6.24 + 23.106) \times 0.9945 - 74.61 \times 0.10452$$

$$= 21.39 \ t \ .$$

Factor of safety:

overturning = 173.94/71.77 = 2.42 (> 2.0)

sliding = 30.91/21.39 = 1.445 (say 1.5).

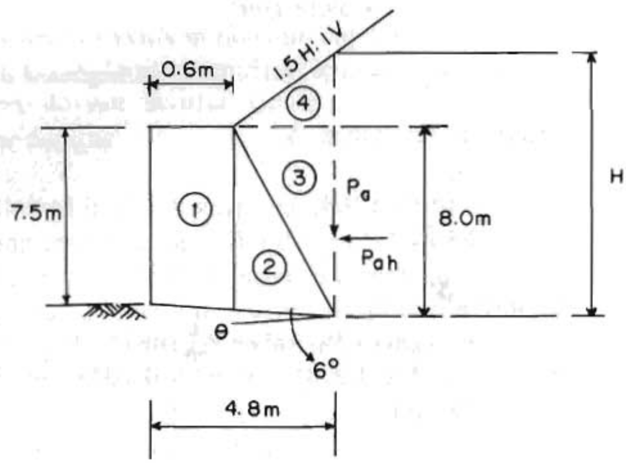
Toe pressure = Allowable Bearing Capacity/39.73 > 1 (Possible here only if foundation is in rock).

Note: The front face of the wall is considered vertical for simplicity although in practice it is inclined at Θ° to vertical that is at 90° to base which is inclined at 0.

Case II

Backfill:

- Type 3 soil
- Sloping at 1.5H:IV
- Unit wt. = 1.77 t/m^3



1)	9.94 t	2.98 tm
2)	24.93 t	69.86 tm
3)	29.74 t	101.10 tm
4)	10.41 t	35.39 tm
	85.02 (W)	209.33 tm

$$P_{ah} = 0.5 \times 0.72207 \times (e + \frac{4.2}{1.5})$$

$$= 0.5 \times 0.72207 \times 10.8^2$$

$$= 42.11 \ t \ .$$

P_{av} K^{av}

$= \quad 1/2 \times K_{av} \times 10.8^2$ $= 50$ lb/ft²/lft

$= \quad 0.5 \times 0.8023 \times 10.8^2$ $= 0.8023$ t/m/lm.

$= \quad 46.79$ t.

Resisting moment $= 209.33 + 46.79 \times 4.8 = 433.92.$

Overturning moment $= 42.11 \times (8/3 - 4.8 \tan\Theta) = 91.05$ tm.

Eccentricity:

$$e = [2.4 - \frac{433.92}{85.02 + 46.79}] \sec\theta = (2.4 - 3.29) \sec\theta$$

$$= -0.89\ m > 0.8\ m \quad . \qquad (absolute\ value)$$

Normal force at base of the wall:

$P_N \quad = (W + P_{av}) \cos\Theta + P_{ah} \sin\Theta$

$\quad = (85.02 + 46.79) \cos6° = 42.11 \sin6°$

$\quad = 135.49$ t.

Maximum pressure at heel, P_{max} or P_{min}:

$$= \frac{N}{B \sec\theta} (1 \pm \frac{6e}{B \sec\theta})$$

$$= \frac{135.49}{4.8 \sec\theta} (1 \pm \frac{6 \times 0.89}{4.8 \sec\theta})$$

$$= 28.09 \pm 31.09$$

$$P_{max} = 59.18\ t/m^2$$

$$P_{min} = -3\ t/m^2 \quad (may\ be\ neglected\ at\ toe)\ .$$

Frictional resistance, S_r:

$= \quad N_f$

$= \quad 135.49 \times 0.4$

$= \quad 54.20$ t.

Sliding force, S_f:

$= \quad P_{ah} \cos\Theta - W \sin\Theta - P_{av} \sin\Theta$

$= \quad 42.11 \times 0.9945 - 85.02 \times 0.1045 - 46.79 \times 0.1045$

$= \quad 28.10$ t.

Factor of safety:

Overturning	= 433.92/91.05	= 4.77	(> 2.0).
Sliding	= 54.20/28.10	= 1.93	(> 1.5).

Maximum foundation pressure (heel), P_{max}:

= Allowable bearing capacity 59.18. (> 2), possible only if foundation is in rock

24.7 PAVEMENT DESIGN

24.7.1 *Problems*

- o Pavement is the most visible and directly impacting element of a road for the users.

- o Lack of a properly paved and sealed surface on a hill road not only causes inconvenience to users but also aggravates the problem of erosion, soil loss, and landslides.

- o The intensive maintenance requirement of unpaved hill roads is contrary to the availability of materials, equipment, trained manpower, adequate annual budget, and overall maintenance management systems in the developing countries.

- o Planned but not properly implemented staging is a general situation arising from the low initial cost considerations generally followed in developing countries. Serious maintenance efforts are warranted only after the 'limit' state of the pavement has been reached, when the original structures (pavement layers) are completely damaged and reconstruction is the only option left. This will not only defeat the original concept of low cost and staged planning but also add a considerable burden of capital investment for reconstruction.

- o Pavement design is not yet a well-developed science. Most highway agencies use empirical methods of design developed either by their own or by other agencies. The highway agencies of developing countries rarely have methods specifically developed for them. Reliability of design solutions, therefore, requires a thorough understanding of the conditions and assumptions concerning the empirical methods of other agencies.

- o Need for low cost considerations and the requirement of skills in maximizing the effectiveness of limited resources present a conflicting situation in developing countries. However, reasonable pavement design techniques for civil engineers do not involve significant cost and time compared to the losses to pavement from *ad hoc* designs.

- o Low-cost and low-volume pavement designs followed in developing countries mostly consider thin surfacing and are essentially based on rutting criteria alone. The design life of bases and sub-bases are generally much in excess of the normal 3 to 5 years' lifespan of the thin surfacing. The base and sub-base, if not properly sealed, after 3 to 5 years become increasingly damaged by the water seeping down and weakening the base, sub-base, and the sub-grade.

- o The traffic on the roads in developing countries consists mostly of trucks and buses which are normally overloaded. The damaging power of any load with reference to a standard load increment increases approximately by a power of 4. Thus, if the damaging power of an 8 ton single axle truck is 1, the damaging power of a 16 ton single axle load increases to 16.

- o Because of low cost considerations, many hill roads are single carriageway and single-lane roads where the traffic loads are channellized and pavement damage is much higher compared to multi-lane roads.

- The poor performance of pavements, resulting from low-cost principles, insufficient budgets, and inadequate maintenance leads to a situation of loss of credibility for the agencies concerned rather than an understanding of the constraints of design principle and priorities of budget appropriation.

- Sub-grade conditions on hill roads vary widely at different sections. Rocky sub-grades require less pavement thickness compared to soil sub-grades. Pavement design, therefore, must be carried out separately for different sections.

24.7.2 *Guidelines*

- Paved and sealed surfacing should be planned for all hill roads, at least in the critical sections such as those having poor sub-grades, steep gradients, surface erosion, gullying conditions, and landslide potential because of uncontrolled runoff.

- Pavement standards in the initial design should be based on realistic considerations of maintenance budget, interruptions to traffic caused by too frequent maintenance operations or upgrading work, and manpower and equipment resources available for the desired level of maintenance.

- Periodic training on new pavement and overlay design methods, for civil engineers engaged in design, construction, and maintenance, should be imparted.

- Research activities, leading to development of pavement design methods specific to the region or agency, should be promoted.

- Axle load-based pavement design methods should be followed for all important roads.

- Engineers should provide an adequate choice of alternative designs with sufficient information on initial costs, life, reliability, time factors, maintenance costs, and weighting of different alternatives to decision-makers, rather than trying to impose decisions by providing a single design. Refer to Chapter 26.

- Pavement designs should be based on a thorough understanding of the principles and assumptions of the methods considered. The following should be considered for every pavement design :

    - traffic count,
    - traffic growth rate,
    - axle load,
    - equivalency factors,
    - traffic expressed in ESA or EAL,
    - drainage conditions,
    - seasonal variations,
    - material properties of different layers of pavement,

- serviceability,
- ageing of bitumen,
- construction quality,
- minimum thickness,
- fatigue consideration for asphalt concrete or dense bituminous macadam surfaces, and
- periodic resealing of thin surfacing.

o Chapter 18, Section 18.3, provides the background for calculations of traffic load for pavement design.

o Chapter 18, Section 18.4, provides the background for new pavement designs by various methods.

o Chapter 18, Section 18.5, presents examples of new pavement designs by various methods. The design charts, tables, and nomographs for various methods included under this section will serve as ready reference material for design. However, the source material should be consulted for further details on their applications.

The traffic volume for pavement design should normally be based on Annual Average Daily Traffic (AADT) from 7 day, 24 hour, classified traffic counts. A 3 day count is used in exceptional cases. In the case of new roads, traffic estimates can be made on the basis of potential land use and traffic on existing routes in the area. Attention should be given to anticipated traffic, possible changes in road network, land use in the area served, and probable growth of traffic.

Traffic growth should be obtained by studying the past trends. An average value of 7.5 per cent annual growth may be adopted for rural roads where adequate data are not available.

o Sub-surface drains below the lined surface drain should be provided in wet areas in soils so that the pavement layers and the sub-grade are kept relatively dry. See Figure 24.51.

o For insloped pavements, an outlet should be provided in the side wall of the drain to allow drainage of under layers. The side drain has to be sufficiently deep so that the water from the drain does not go into the pavement layers. See Figures 24.52 and 24.53.

o The width of the sub-base course should be extended to the edge of the pavement towards the valley side for ensuring drainage of the pavement layers and protection of the sub-grade. See Figure 24.54

o The shoulders of single-line, paved roads should be treated with single course asphalt seal to protect the edges of the road and prevent damages to the pavement from the rutting and erosion of the shoulders.

o Outsloped sealed pavements should be considered for low design speed (less than 30 km/hr) roads and for sections at vertical gradients of less than 5 per cent.

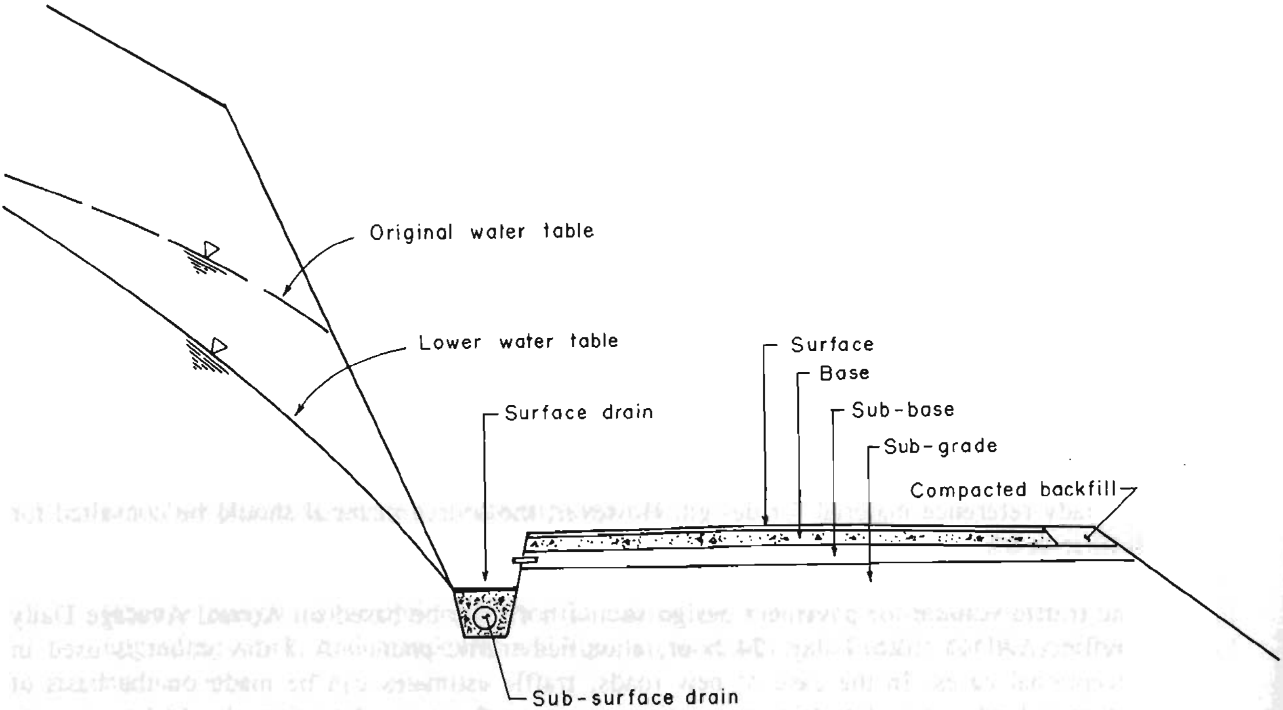


Fig. 24.51 Sub-surface drain to protect pavement in wet areas in soils

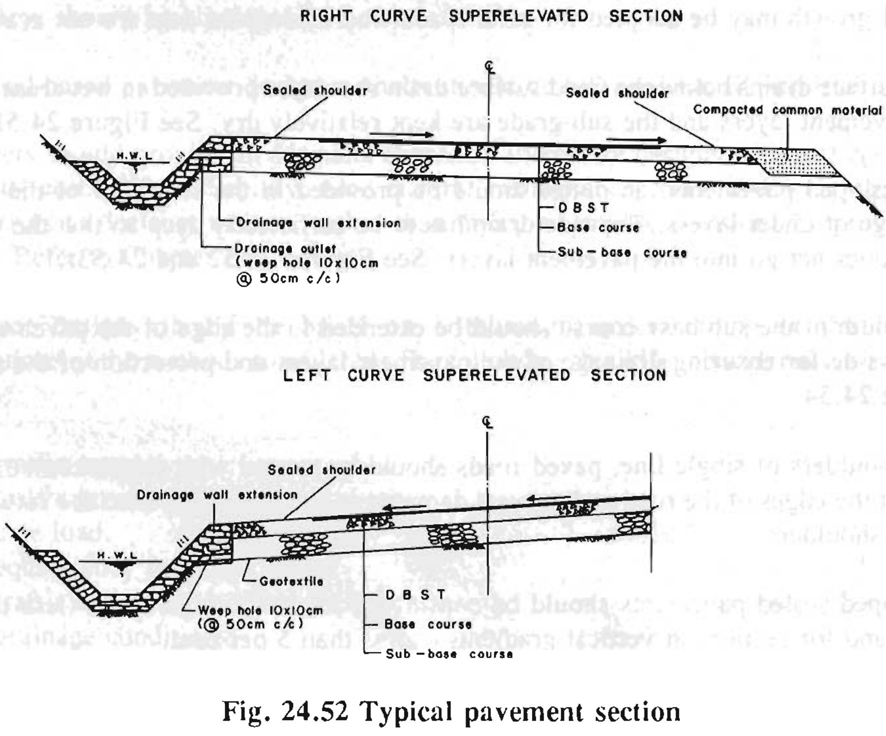


Fig. 24.52 Typical pavement section

NORMAL SECTION

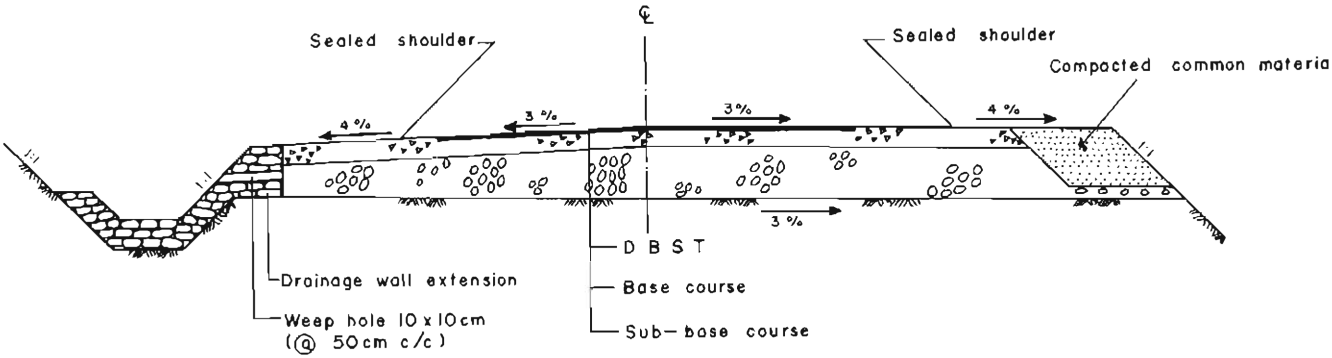


Fig. 24.53 Typical pavement section, normal section

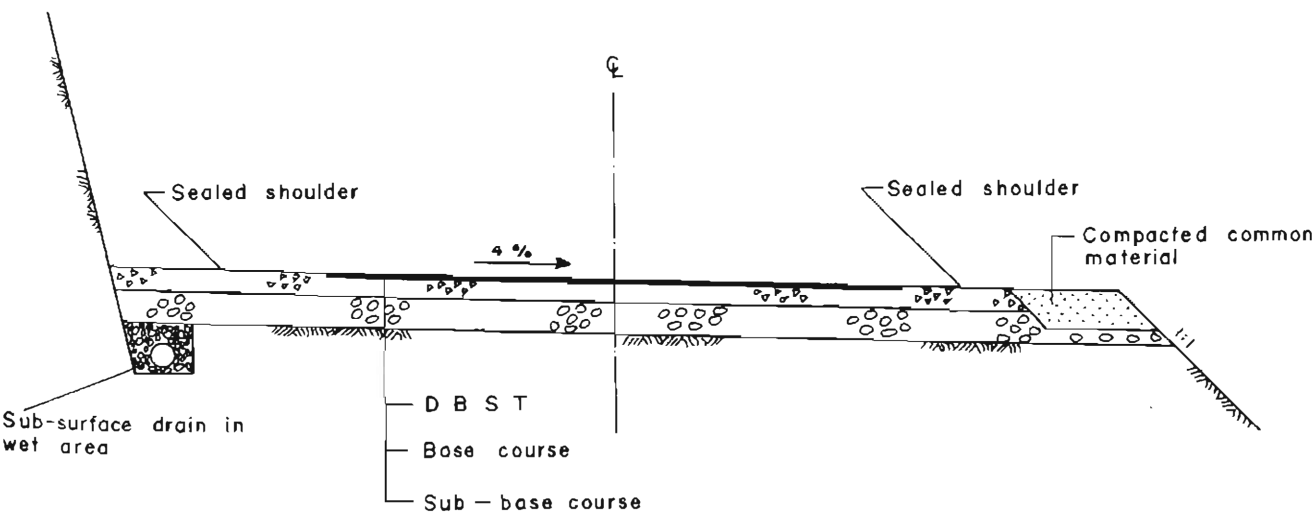


Fig. 24.54 Typical outsloped pavement section

24.8 BLASTING DESIGN

The excavation of rock slope usually involves blasting, which refers to the instantaneous release of shock energy that is propagated into the surrounding rock mass as a violent shock wave producing rapid, alternating compression and tension into the rock causing it to break. When an explosive is detonated, it is converted into a high temperature gas in a few thousandths of a second, producing high pressure exceeding 100,000 atmospheres. This pressure, exerted in the form of shock waves, is sufficient to shatter the rocks and form a crushed zone around the hole. The outgoing waves cause a radial crack zone around the crushed zone due to higher tensile stress component than the radial compressive component. When the shock waves reach a free face, the rock is able to expand and slabs of rock break from the face (Fig. 24.55). The shock waves are reflected back from the face and this accelerates rock breakage.

The blasting for road excavations differs much from open-cast mining in the sense that it is a small-scale operation and the depth of excavation rarely exceeds 5m. The blastholes are drilled by jack hammers and the depth of blasting is a cycle of operation of less than 3m, although it is generally between 1 to 2m. From the practical convenience of operation, it is preferable to have cut slope benches of 1.0 to 1.5m width and 1.5 to 2.0m height, depending upon the design cut slope angle. Based on the existing slope geometry and geological conditions, the cut slope angles are chosen and cross-sections are prepared to show the cut slopes (Fig 24.56). The excavation starts from the highest level and progresses down to the road level.

24.8.1 *Explosives*

There are basically two types of explosives available: i) low strength explosives - e.g., black powder, a mixture of sulphur, charcoal, and ammonium nitrate and Ammonium Nitrate-Fuel Oil (ANFO) slurry explosives; ii) high strength explosives - e.g., dynamite, available in 100 per cent strength or a range of strength from 20 per cent to 100 per cent. High strength explosives are generally used for road excavations. However, ANFO explosives are preferable in highly-jointed rocks and softer rocks, since the explosion is associated with the production of excessive gases which causes backbreaks and better fragmentation.

24.8.2 *Blasting Techniques*

When the road passes through populated areas, it is essential to adopt controlled blasting so that overbreaks (backbreaks) are not caused to damage the structures. Moreover the excavated slope should have a more or less smooth face without overhangs. To achieve these objectives, a pre-split or pre-shear blasting technique is often preferred. The pre-split blasting technique for road excavations has been shown in Figure 24.56. Let us consider the shaded portion of the rock to be excavated at the top bench level. If this portion of rock is to be blasted out, say by five rows of drill holes R1, R2, R3,R4, and R5 in sequence, the first row of blast hole (R1) to be fired is located at the extreme hillside end of the bench, called the **neat excavation** line. These holes are called **pre-split blastholes**. Line drilling which consists of closely-spaced drill holes (Figure 24.57) is carried out in this row of holes. The spacing of holes is kept at 20 to 30cms.

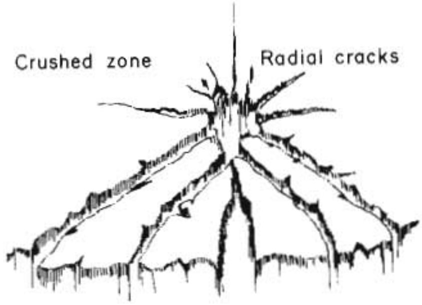
The theory of pre-splitting is that when the charges are shot simultaneously in adjoining holes, the collision of shock waves between the holes places the web in tension and causes the cracking that forms a sheared zone between the holes. With proper spacing and charge, the fracture zone will be a narrow sheared area. Subsequently, when primary blasting of rows 2 to 5 is carried out, the generated shock waves will be prevented from being transmitted into the finished wall thereby minimising shattering and overbreak.

The pre-split holes are loaded with light, well-distributed charges and completely stemmed as shown in Figure 24.58. For pre-split blasting the amount of explosive may vary from 0.08 to 0.25 lb/ft of the hole. Of the total amount, 50 per cent may be loaded at the bottom of the hole, while 25 per cent may be packed at three fourths (3/4) of the depth and another 25 per cent at half the depth of the hole. If the depth of the hole (D) is 8', then the total amount of charge may be 8 x 0.15 = 1.20 lb, which may be distributed as 0.30 lb at C, 0.30 lb at C_2 and 0.60 lb at C_3. Alternatively, especially in shallow holes, 70 per cent of the total charge may be kept at the bottom of the hole, while the rest may be packed at just below half the depth of the hole. The holes may be fired simultaneously, using a **Prima** trunk line. However, if manual blasting is undertaken, the length of the fuse at the bottom charge should be kept sufficiently long, i.e., it should extend at least about 1 to 1.5m outside the hole. The length of the fuse for other charges should be almost the same so that all the charges are blasted at the same time. The sequence of firing should start from the top charge and progress towards the bottom charge.

For primary or main blasting, as the blastholes rarely exceed 2m, the amount of explosive may be kept at twice the weight as that for the pre-split holes. The blast hole pattern is another important factor for a successful blasting. A **staggered** blasthole pattern is preferred for road excavations. The distance between the rows of blastholes is called effective burden (e) and the distance between individual holes in a row is the effective spacing (s). In general an effective burden of 1.0m may give good results. An effective spacing of 1.25 times the effective burden is preferred for good fragmentation. The optimum amount of explosives per hole, as well as the spacing and the burden of holes, may vary for locations. Field trials using different combinations of these parameters will help the field engineer to decide upon an effective combination.

24.8.3 *Chemical Blasting*

Chemical blasting is another method of controlled blasting. Chemical blasting is expensive and is not practicable for large-scale operations. However, chemical blasting may be more useful for small- scale blasting near existing structures and in areas susceptible to risks from flying rock fragments. ***Calmmite*** is a patented breaking compound developed jointly by Nikon Cement Co. Ltd., Nippon Oil and Fats Co. Ltd., and Nichiya Giken Kogyo Co. Ltd. of Japan. Calmmite is supplied in capsule form. **Bristar** is another breaking agent patented by Onoda Cement Co. Ltd. of Japan. **Bristar** is supplied in powder form. There are several such components manufactured and marketed in different parts of the world.



Source: Hoek and Bray 1981

Fig. 24.55 Mechanism of rock fracture by explosives

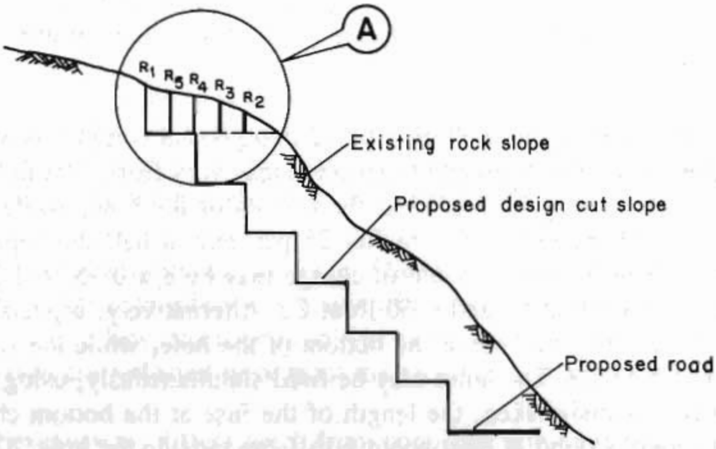


Fig. 24.56 Illustrative diagram for pre-split blasting

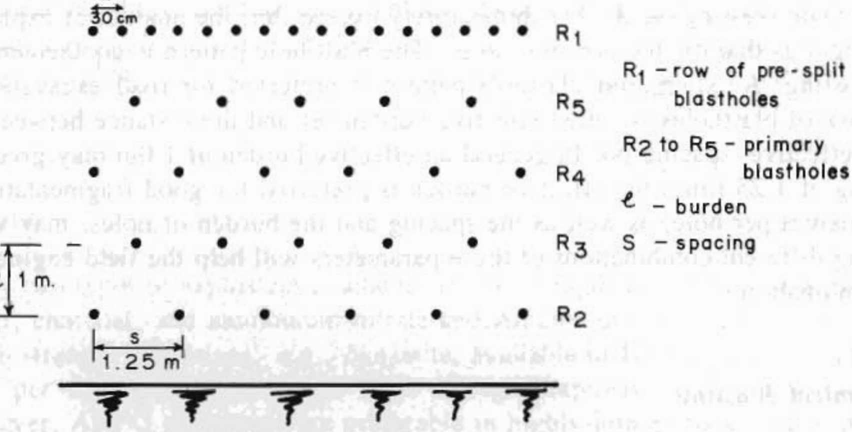


Fig. 24.57 Plan view of A - staggered pattern of blasting

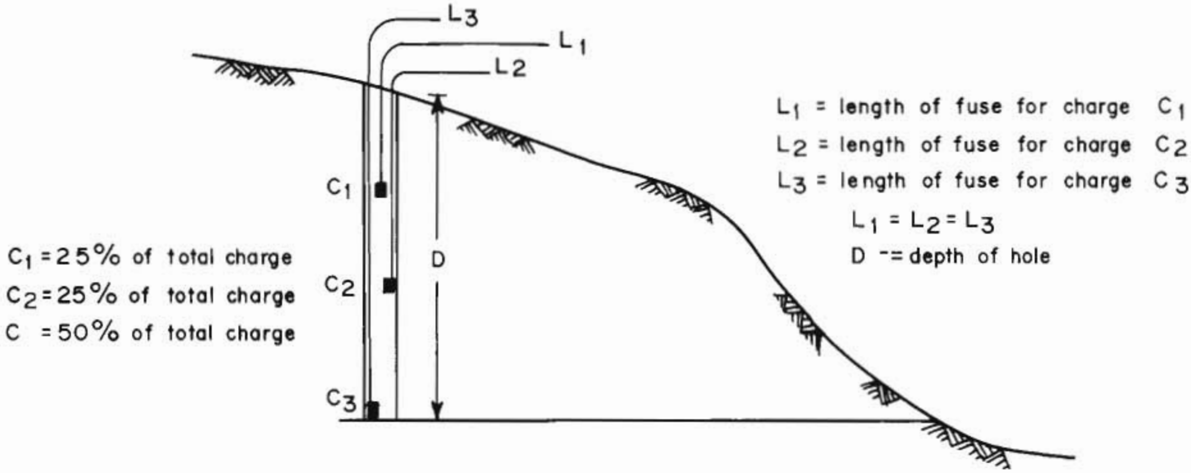


Fig. 24.58 Details of charge placement of pre-split holes of row R

24.9 LANDSLIDE STABILISATION

24.9.1 *General*

- Landslides that are deep-seated, massive, and influenced by tectonic movements defy analysis and control.

- Medium and minor landslides can be stabilized by drainage (see Table 24.15 for design details), anchors, toe support, removal of load on the top, and protection from erosion by vegetation and plants. However, normally the slide areas extend far beyond the edge of cut slope and the right of way and any effort to stabilize the landslide would require the acquisition of the privately-owned land. Agricultural activities in the landslide areas if continued, and mostly where irrigation is involved, aggravate the instability conditions and lead to the failure of naturally or artificially stabilized landslides. The ponds for irrigation cause seepage into the slope mass below them.

- Landslides once started may be i) one-off, ii) diminishing, iii) cyclic (intermittent, diminishing), and iv) continued. Except for small one-off slides that can be simply cleared, stabilization becomes necessary from environmental as well as road operation considerations.

  Most of the landslides give enough warning before failure, e.g., tension cracks, increase in seepage from cut slopes, release cracks in rock slopes, and tilt of trees on soil slope.

- Long-term stability requires that slope enrichment by plantation and vegetation should be considered in the entire watershed influencing the particular landslide. The humus generated by vegetation should be able to reduce the rate of sheet and rill erosion and eventual gullying.

- Landslides can be stabilized by several methods, namely, engineering structures alone, combination of engineering structures and biotechnical measures, and biotechnical measures alone. However, compatibility in the requirements of plantation and engineering structures should be properly managed (Gray and Leiser 1982).

- Shallow slides and **surficial slides** may be tackled without rigorous analysis, but deep-seated slides must be thoroughly investigated and analyzed before recommending solutions. Table 24.16 presents a summary of methods for prevention and correction of landslides. Table 24.17 presents empirical relations among various factors in the use of restraining devices to control active landslides. Figures 24.59 to 24.79 present some examples of various measures of stabilization.

- Slopes involving heavy cutting and major landslides must be investigated and analyzed before stabilization measures are recommended.

- Landslide analysis may be done by manual methods, chart methods, or by computer applications. Manual methods are cumbersome and time-consuming. Package programmes are available for analysis using personal computers. Use of appropriate package programmes enable rapid analysis of several landslides. Chapter 13 describes some chart methods and a package of computer programmes. Solutions to landslide analysis and control are the task of a geotechnical specialist. However, engineers dealing with mountain roads must be familiar with the principles and existing methods of landslide analysis and control. The principles of landslide analysis are discussed in Chapter 13.

Analysing landslides and identification of mode of slope failure involve firstly data collection, secondly back analysis, and lastly analysis to determine the factor of safety of the slide mass with proposed corrective measures.

The following computer programmes, developed by the University of Roorkee, India, may be used for landslide analysis and design of remedial measures according to the dominant mode of slope failure.

Soils

BAST	-	Back analysis of slopes with talus deposit.
BASC	-	Back analysis of slopes with circular mode of failure
SARC	-	Stability analysis of reservoir and submerged slopes in circular failure.
SAST	-	Stability analysis of slopes in talus deposit.
ASC	-	Cut slope angle of slope with circular wedge failure

Rocks

BASP	-	Back analysis of slope with planar failure in rocks.
ASP	-	Stability analysis of slope failure in rocks.
ASW	-	Analysis for optimum angle of cut slope with planar wedge failure.
SASW	-	Stability analysis for slope wedge failures.
ASP	-	Cut slope angle of slope with planar failure in rocks.

24.9.2 *Specifications for Landslide Drainage*

Drainage System for Stabilizing Debris Slides

Observations indicate that debris slides with heavy seepage generally occur on gentle slopes with a thin cover of debris/talus. It is also supported by theory (Chapter 13.2.2). So the ideal solution for such landslides would be to provide sub-surface drainage by trench drains.

<u>Landslide stabilization consists of the following works</u>:

(i) Construction of lined, catch drain at least 15m from the landslide area so that it does not act as a water-filled tension crack. The size of the drain should be according to the catchment area. It is joined with the neighboring gullies so that rainwater can flow out easily without entering into the landslide area. (Alternatively sub-surface pipe drains may be constructed above the landslide area if the water table is near ground level.)

(ii) Construction of counterfort trench drains is also essential in the case of debris flow and associated seepage problems. The design of counterfort drains should be based on Chapter 19. Their spacing is generally between 5 to 10 metres. Their width is usually 1 metre. Their depth may be up to rock bed\clay seam or clay layer. It would be better to use the computer programme SAST for designing a system of trench drains. The length of the drains should be up to the lip of the landslide. The top 1m depth of the trench drain is filled with clayey soil which should be well compacted; otherwise rainwater will quickly enter inside the landslide area and ruin everything.

Sub-surface pipe drains (French drains) are also successful and have branches on both sides to drown a large area of shallow debris slide.

(iii) Construction of gabion toe wall. A trench drain is also constructed below the heel of the wall which will collect all seepage water from the counterfort drains.

(iv) Finally a few cross drains are built below the road to drain off the seepage water from the heel drain and keep the sub-grade of pavement dry.

(v) Planting of fast-growing grass and bushes should always be done wherever possible. Filter material for all types of drains should meet the following requirement of gradation.

Percentage smaller than 100mm	=	100 per cent.
Percentage smaller than 10 mm	=	5 to 30 per cent.
Percentage smaller than 5 mm	=	5 to 10 per cent.
Percentage of clay passing ASTM Sieve no 200	=	less than 2 per cent.

It may be recalled that the efficiency of drains drops down to about 50 per cent when there is lack of maintenance and partial choking (Hutchinson 1977). The life of even a properly maintained drainage system is no more than 10 to 15 years.

Trench drains are also used as a preventive method for stabilizing slopes in distress.

Horizontal Drains for Deep Sub-surface Drainage

There are various types of thick materials that are unstable because of groundwater conditions. Among them, there is one that typically shows an abundant silty matrix and which is frequently deposited in the lower sections of valleys of the northern Himalayan hills (See Annex 1, Section 1.3.2 and Figure 1.3 [Ann.]). These soils show a more or less deep water table and in addition perched water tables formed on more clayey horizons. The water table or perched water table levels rise during the rainy season, and meanwhile a several metres thick (for under tension) and loose upper layer of material is soaked by the rainwater. A wide landslide may be the result of such conditions.

Trench drains are insufficient for controlling these types of slide and, in such cases, deep, horizontal sub-surface drainage together with trench drains are necessary.

Drainage of a deep sub-surface, perched water table, or groundwater table and aquifer stabilizes a slope in distress for the following reasons.

(i) High pore-water pressure within the soil/rock mass is reduced considerably.
(ii) Shear strength along the slip surface is increased.
(iii) Seepage forces are reduced. Seepage erosion is checked as groundwater is lowered down below the slope face. Thus fines are not washed out.
(iv) The unit weight of soil is reduced slightly.

Applicability

(i) Geological and geophysical investigations should be carried out in the field to find out whether the landslide has really been caused by heavy seepage problems. It is also necessary to explore the depth and extent of the water table and permeable layers of sand/sandstone and aquifers and to observe all springs.

(ii) The slope material should not be impervious or have a lot of fines or clay so that the groundwater can easily drain into the pipe.

(iii) It should be possible to drill holes horizontally. In silt or slushy soil slopes, drill holes collapse quickly. In highly-fractured, hard rock, it is difficult to drill and there is considerable loss of circulating fluid during drilling.

(iv) Landslide areas should not be active, otherwise pipes will be ruptured quickly.

(v) A heavy boring machine is available on site to drill horizontal drill-holes. This needs a road or wide **berm** to move and drill. These berms should also be made in a landslide area if possible.

Horizontal drains are now quite popular among highway engineers working in the Himalayan Region of India. They have also been successfully used in Nepal for the first time, quite recently. Experience has shown that these are effective under a wide variety of soils and geologic, topographic, climatic, and groundwater conditions. They should be used with other corrective measures, e.g., drystone breast wall, anchored surface drains, and bioengineering methods. The horizontal drains are also used as preventive methods for stabilizing slopes in distress.

Installation of Drains

Basically, horizontal drains are drill holes that are drilled into a cut face above the landslide area and close to its edge, and cased with perforated pipes. Figure 24.77 shows one typical example. Figures 24.75 and 24.76 give typical locations of horizontal drains.

The following guidelines are offered for the planning of drains (Natrajan et al. 1985).

(i) The spacing of drains varies from 3m to 30m, depending upon the seepage problem, soil\rock permeability, and size of landslide. Several rows of drains may be installed if the terrain and groundwater table permit. The inner end of the drain should be placed just below or above the tip of the landslide.

(ii) The length of drains varies from 8m to 100m, depending upon the depth of the water-bearing strata. It is better to install shorter drains in large number rather than very long drains at wide intervals. The pipe should never extend to more than 3-5m beyond the slip surface. Greater length may bring more water to the failure zone. Table 24.15 gives design details.

(iii) The grade of drain is between 2-20 per cent according to dip and thickness of permeable layers. A grade of 10 per cent is easily adapted (Figure 24.75).

(iv) All outlets of the horizontal drains are connected by pipes to carry discharge away from the landslide area. Alternatively, the outlet should discharge water into a roadside drain or metal pipes of 15 to 20cm diameter.

(v) The pipes are generally rigid PVC pipes of 50mm in diameter and perforated with 3mm diameter holes at 10-15 mm distance in zig-zag fashion. The filter around the drain is formed by porous concrete or resin-bonded sand or geotextile fabric.

Alternatively slots (2 m width x 55 m length) are made in pipes at spacings of 50 mm. The slots are more effective than holes. The pipes should preferably be surrounded by geotextile fabric to reduce the chances of the clogging of the holes/slots in the case of fine soils. These pipes are joined with solvent and coupling to obtain the desired length.

(vi) The inside end of the pipe is fitted with chuck (for mechanical anchorage). The outer end is not perforated for up to one-third of the length to prevent choking by roots or debris. The invert of the pipe is made impermeable by a layer or cement grout underneath the pipe in the case of fully perforated/slotted pipes (see Figure 24.77).

(vii) Water flow through each drain should be monitored during the rainy and dry seasons and recorded.

One should not assume that drains with little flow are least effective. They may drain off more water later during the rainy season. In silty soils, discharge is low but pore-water pressures may be substantially reduced.

Table 24.15 : Length and spacing of horizontal drains for soil slopes

S.No.	Slope Condition	Length of pipe / Height of slope	Spacing of pipes / Height of slope
A	Slip surface above bedrock which is far below toe of slope	3	2
B	Bedrock is horizontal and passes through toe of slope	1.5	1.5

Source: Natrajan et al. 1985

Maintenance of Horizontal Drains

(i) Positions of all outlets of drains should be recorded.

(ii) The toe of the slope should be kept clean of sliding debris so that outlets are not covered up.

(iii) Growth of the plants/weeds/grassroots is often encountered around outlets and in the first 3-6m section of horizontal drains. This not only conceals the drain but also affects its performance. Selected herbicides may be used to prevent the growth of plants.

(iv) PVC pipes are easily damaged by rockfalls, so galvanized iron pipe sleeve with a length of 3m may be inserted at the outlet.

(v) The drain should be cleaned after 3 to 5 years to improve its efficiency.

Recent Developments

In Europe, the HYDRO-GEO Company of France has developed an automatic syphon drain system for vertical drains. Figure 24.78 shows the drainage system which is based on the syphon principle. Vertical drains of about 10cm in diameter are made along the road in the distressed reach where the perennial source of groundwater is causing the landslide. Plastic tubes of 5mm in diameter are inserted into the vertical drains. These are filled with water to drain off air from the pipes. The other ends of the pipes are then lowered down below the bottom of the borehole level. In this way, water collected in the boreholes is automatically drained off by the syphon arrangement. More pipes are inserted if discharge is more. This system may give a lot of maintenance problems.

24.10 RIVER TRAINING

24.10.1 *Background*

Normally, the upper reaches of a natural water course discharge a young stream, marked by relatively high velocity on a steep gradient.

The stream tends to scour its bed and erode its banks by corrosion. If hard particles (sand, gravel, cobble, or boulder) are transported along the bed, scour is more rapid than erosion and the stream develops a gorge. Young streams are usually V-shaped, with the stream at flood covering all or most of the valley floor. Young streams rapidly lower the bed and cause instability of the banks by toe removal and steepening of the side slopes.

Once the limit of degradation is reached by young streams, a stable gradient is reached and the stream is mature. Mature streams usually develop U - shaped valleys with broad terraces above the floodplain. Banks are still vulnerable to erosion and the stream widens its channel. Irregularities in resistance lead to erratic alignment. Inertia of the stream at bends accelerates erosion on the outside of bends, developing a meander. Progressive meander is outwards and downwards on the bends, working and reworking the entire floodplain without achieving horizontal stability unless restrained artificially. The hydraulic action of streams results in various types of erosion as illustrated in Figures 24.80 to 24.85.

Table. 24.16 Summary of methods for prevention and correction of landslides

Effect on Stability of Landslides	Method of Treatment	General Use		Frequency of Successful Use[1]			Position of Treatment on Landslides[2]	Best Applications and Limitations
		Prevention	Correction	Fall	Slide	Flow		
Not affected	I. Avoidance methods:							
	A. Relocation	x	x	2	2	2	Outside slide limits	Most positive method if alternate locations economical
	B. Bridging	x	x	3	3	3	Outside slide limits	Primary highway applications for steep, hillside locations affecting short sections (parallel to C/L)
Reduce shearing stresses	II. Excavation:[3]							
	A. Removal of head	x	x	N	1	W	Top and head	Deep masses of cohesive material
	B. Flattening of slopes	x	x	1	1	1	Above road or structure	Bedrock: also extensive masses of cohesive material where little material is removed at toe
	C. Benching of slopes	x	x	1	1	1	Above road or structure	
	D. Removal of unstable material	x	x	2	2	2	Entire slide	Relatively small shallow masses of moving material
Reduces shearing stresses and increases shear resistance	III. Drainage:							
	A. Surface							
	1. Surface ditches	x	x	1	1	1	Above crown	Essential for all types
	2. Slope treatment	x	x	3	3	3	Surface of moving mass	Rock facing or previous blanket to control seepage
	3. Regrading surface	x	x	1	1	1	Surface of moving mass	Beneficial for all types
	4. Sealing cracks	x	x	2	2	2	Entire, crown to toe	Beneficial for all types
	5. Sealing joint planes and fissures	x	x	3	3	N	Entire, crown to toe	Applicable to rock formation
	B. Sub-drainage:							
	1. Horizontal drains	x	x	N	2	2	Located to intercept and remove sub-surface water	Deep extensive soil mass where groundwater exists
	2. Drainage trenches	x	x	N	1	1		Relatively shallow soil mass with groundwater present
	3. Tunnels	x	x	N	3	3		Deep extensive soil mass with some permeability
	4. Vertical drain wells	x	x	N	3	3		Deep slide mass, groundwater in various strata
	5. Continuous siphon	x	x	N	2	2		Used principally as outlet for trenches or drain walls
Increases shearing resistance	IV. Restraining structures:							
	A. Buttresses at foot:							
	1. Rockfill	x	x	N	1	1	Toe and foot	Bedrock or fire soil at reasonable depth
	2. Earthfill	x	x	N	1	1	Toe and foot	Counterweight at toe provides additional resistance
	B. Cribs or retaining walls	x	x	3	3	3	Foot	Relatively small moving mass or where removal of support is negligible
	C. Piling:							
	1. Fixed at slip surface		x	N	3	N	Foot	Shearing resistance at slip surface increased by force required to shear or bend piles
	2. Not fixed at slip surface		x	N	3	N	Foot	
	D. Dowels in rock	x	x	3	3	N	Above road or structure	Rock layers fixed together with dowels
	E. Tie-rodding slopes/soil nailing	x	x	3	3	N	Above road or structure	Weak slope retained by barrier which in turn is anchored to solid formation
Primarily increases shearing resistance	V. Miscellaneous Methods:							
	A. Hardening of slide mass:							
	1. Cementation or chemical treatment							
	a. At foot		x	3	3	3	Toe and foot	Non-cohesive soils
	b. Entire slide mass	x	x	N	3	N	Entire slide mass	Non-cohesive soils
	2. Freezing			N	3	3	Entire	To prevent movement temporarily in relatively large moving mass
	3. Electro-osmosis	x		N	3	3	Entire	Effects hardening of soil by reducing moisture content
	B. Blasting		x	N	3	N	Lower half of landslide	Relatively shallow cohesive mass underlain by bedrock Slip surface disrupted; blasting may also permit water to drain out of slide mass
	C. Partial removal of slide at toe	-	-	N	N	N	Foot and toe	Temporary expedient only; usually decreases stability of slide

1. 1 = frequently; 2 = occasionally; 3 = rarely; N = not considered applicable.
2. Relative to moving or potentially moving mass
3. Exclusive of drainage methods

Table 24.17 Empirical relations between various factors in the use of restraining devices to control active slides*

Type of Treatment	Effect of Quantity of Moving Mass, % by Vol.	Effect of Foundation Conditions	Relative Stability	Type of Movement
1. Buttress at foot a. Rockfill	Buttress should be 1/4 to 1/3 of the volume of total moving mass to be retained.	Should extend at least 2 to 3m below slip-plane unless stable bedrock is encountered	With the exception of rock buttress, retaining structures are not recommended for controlling very unstable masses at the toe. Near the top of landslide, piling, cribs, retaining walls, and tie rodding/soil nailing of slopes can be used successfully.	In general, restraining structures are not recommended for falls or flows except as underpinning. If drainage is also provided, a restraining device may be helpful if area is permitted to drain before retainer is built.
b. Earthfill	Recompacted fill should be 1/3 to 1/2 that of total moving mass to be related.			
2. Crib or retaining wall	Volume of crib should be 1/6 to 1/10 that of total moving mass to be retained.	Stable bedrock preferred. Otherwise extend 1 to 2m below slope-plane.		
3. Piling a. Fixed at slip-surface	One pile per 100 cu. m. of moving mass; maximum depth of moving mass of 4 to 5m.	Anchor 1/4 to 1/3 total length of piles in stable bedrock; 1/3 in stable soil.		
b. Not fixed surface	One pile per 50 m^3 of moving mass; maximum depth of moving mass of 3 to 4m.	Necessary only where no stable bedrock is beneath slip-plane.		
4. Dowels in rock	-	Stable bedrock required.		
5. Tie-rodding of slopes/soil nailing	-	Stable material needed for anchorage.		

Source: HRB Special Report 19, 1958

* Subject to evaluation and experience in a given locality.

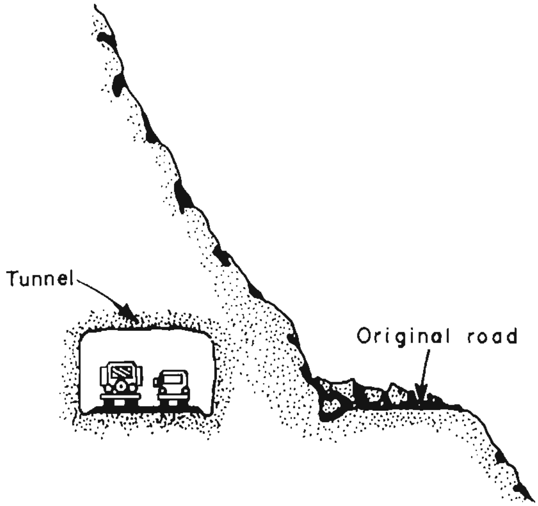


Fig. 24.59 Rock sheds and tunnels

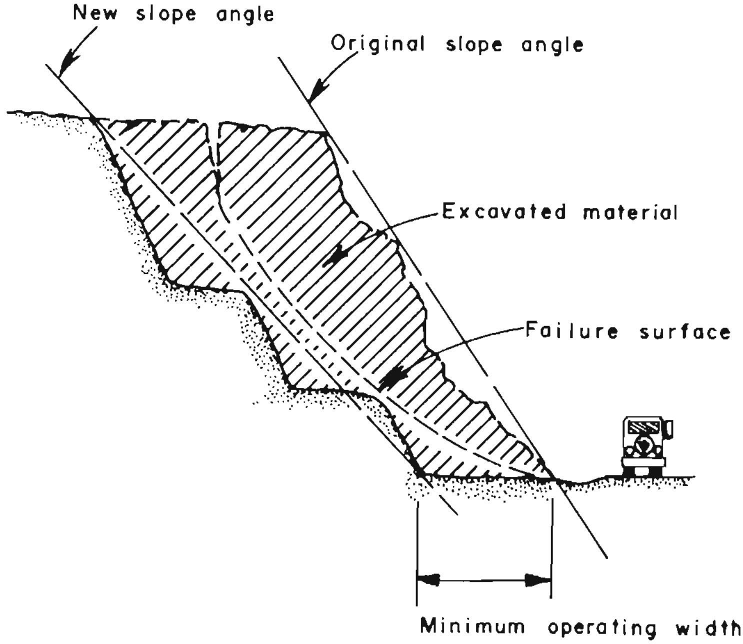
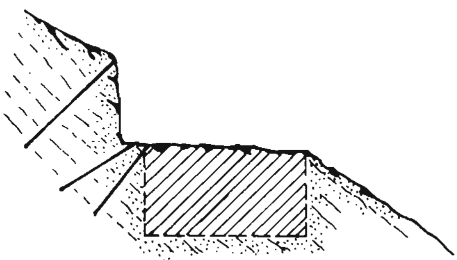
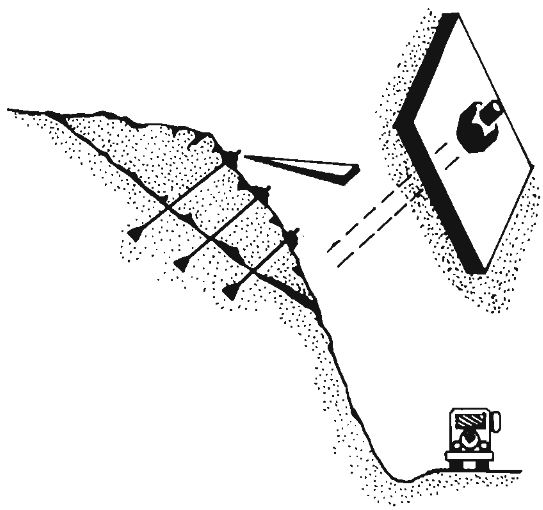


Fig. 24.60 Resloping



a) Untensioned bolts



b) Tensioned anchors installed

Fig. 24.61 Rock bolt installations

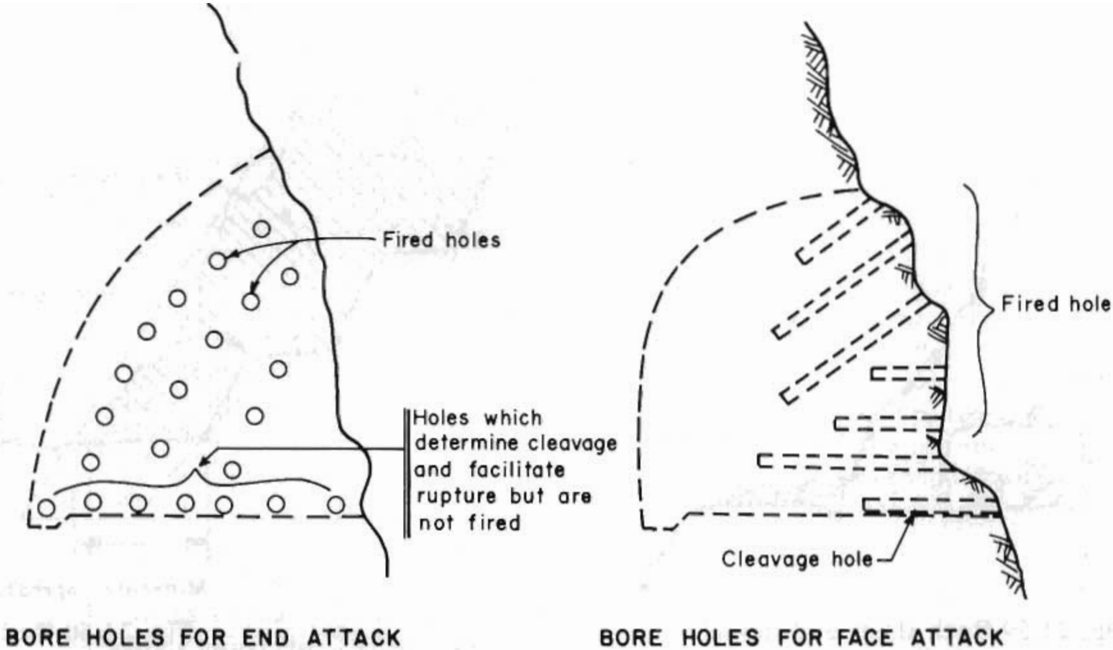
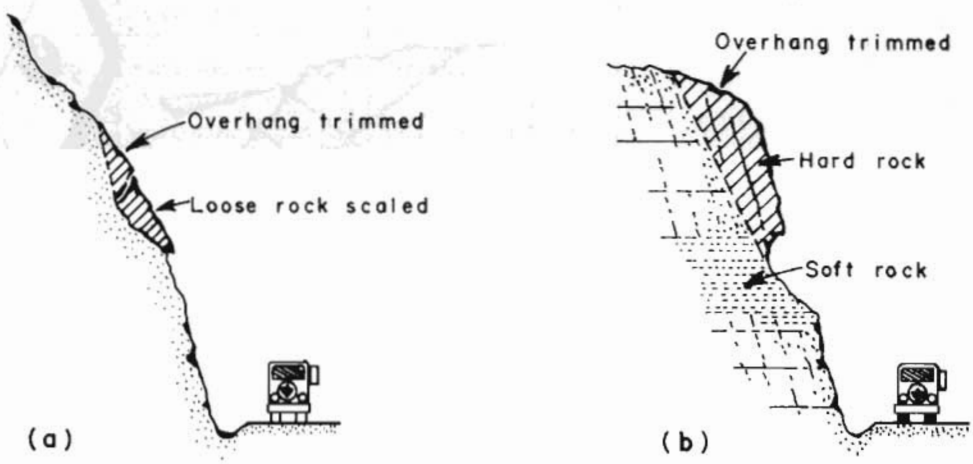
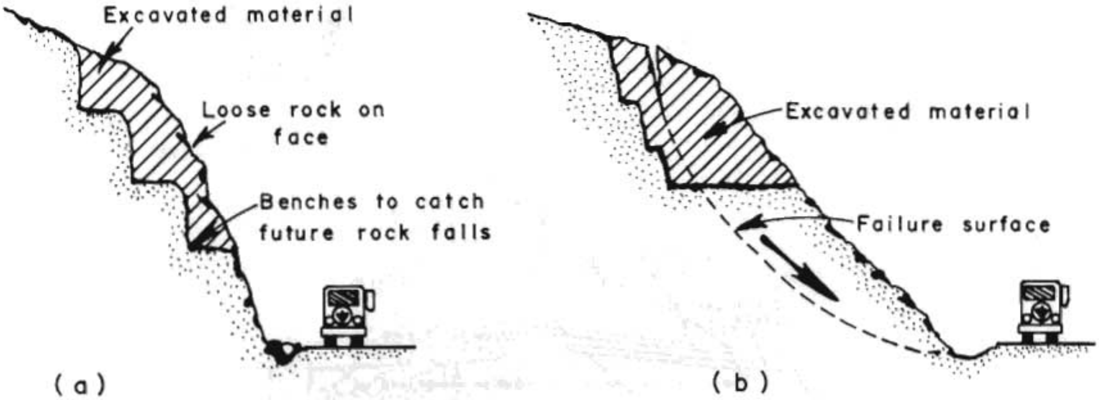


Fig. 24.62 Half-tunnels in hard rocks with joints dipping into very steep hills



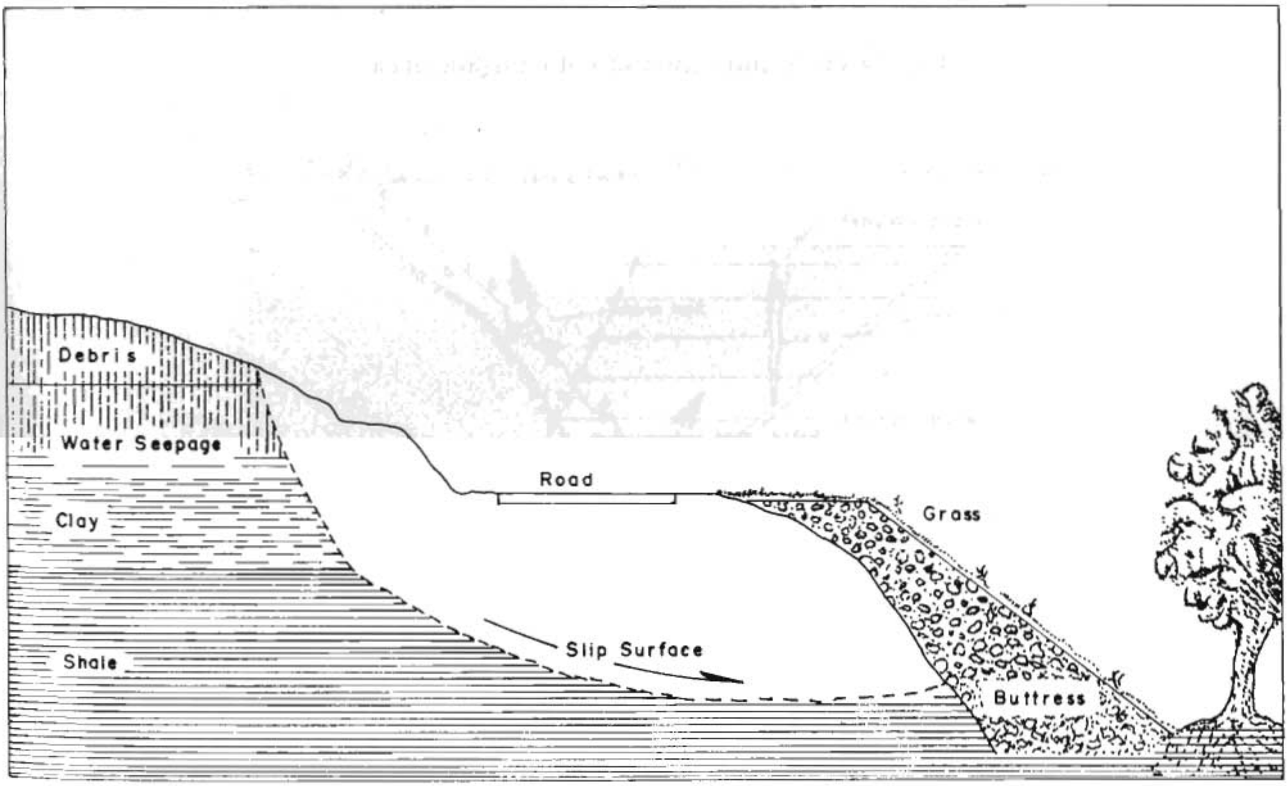
Source : Rock Slopes 1981

Fig. 24.63 Scaling and trimming



Source: Rock Slopes 1981

Fig. 24.64 Benching and unloading



Source: Schuster and Krizek 1978

Fig. 24.65 Rock-fill buttress acting as toe support

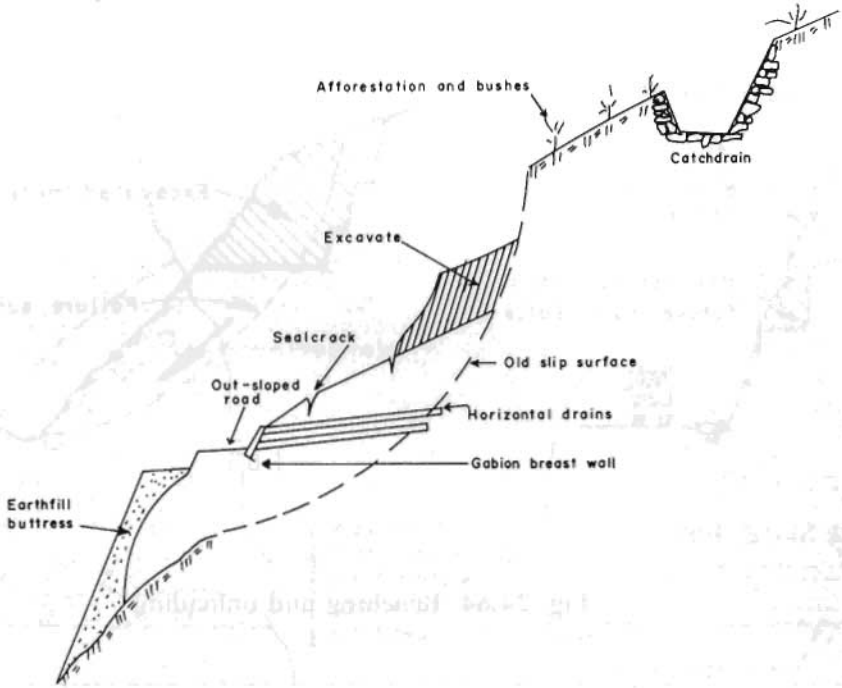


Fig. 24.66 Stabilization of old landslide area

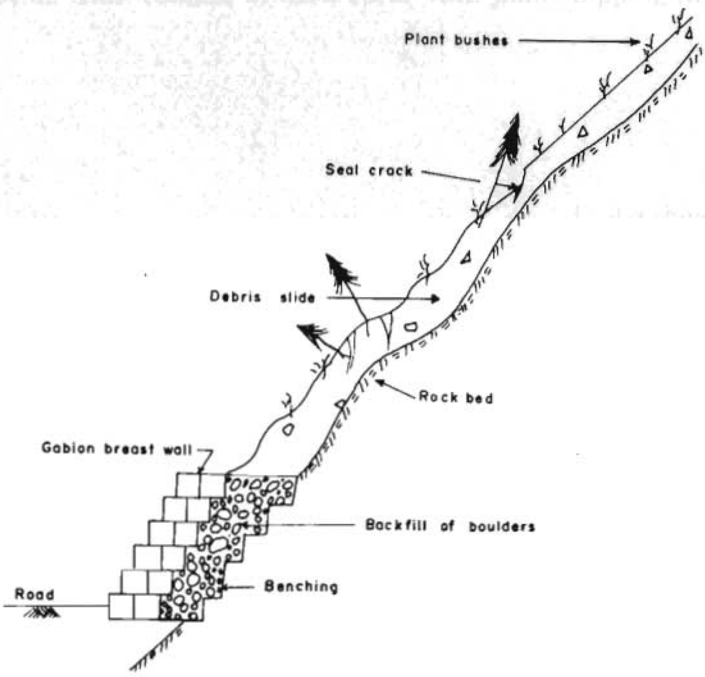
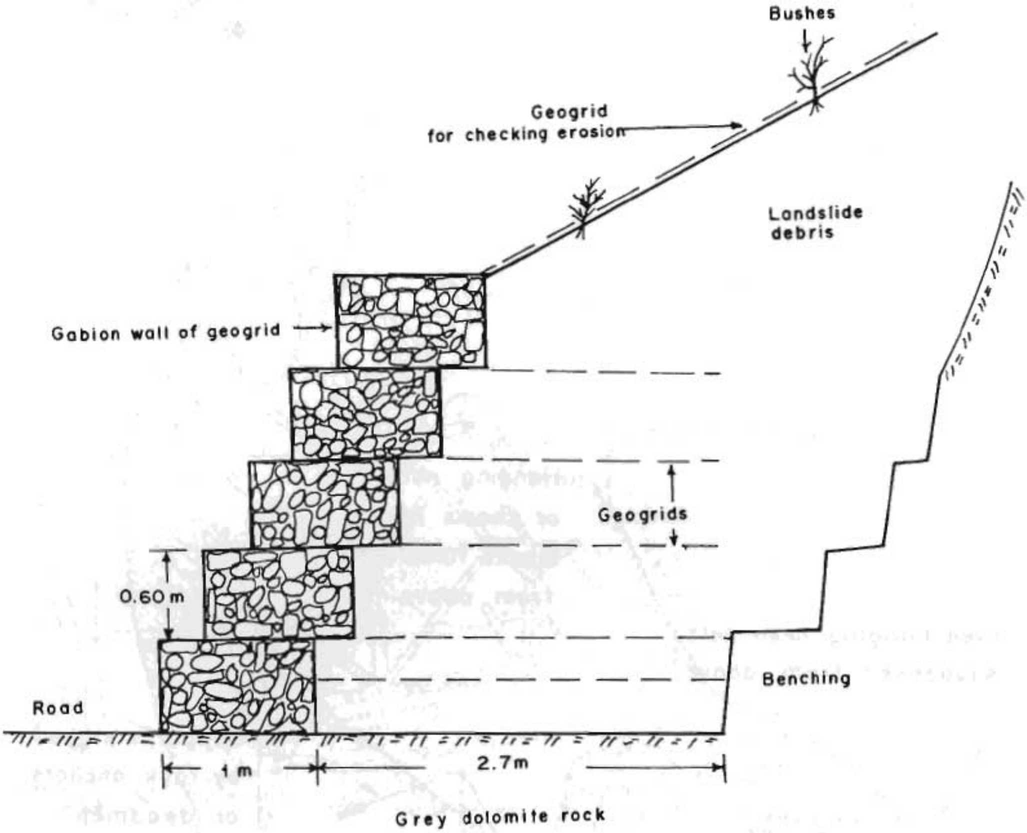


Fig. 24.67 Stabilization of minor landslides by gabion breast walls

Position of lower cracks indicate hump in rock bed.



Source: Bhandari 1988

Fig. 24.68 Stabilization of landslide area by earth reinforcement

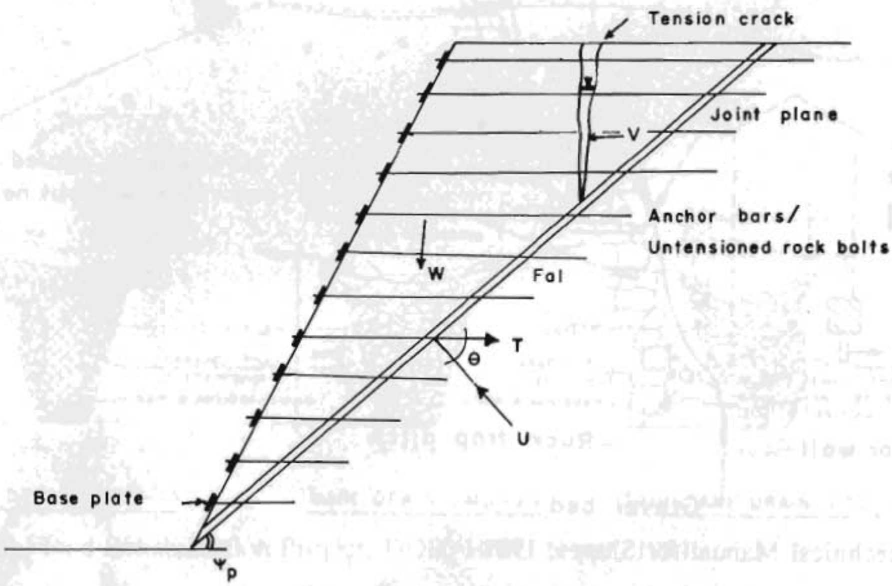
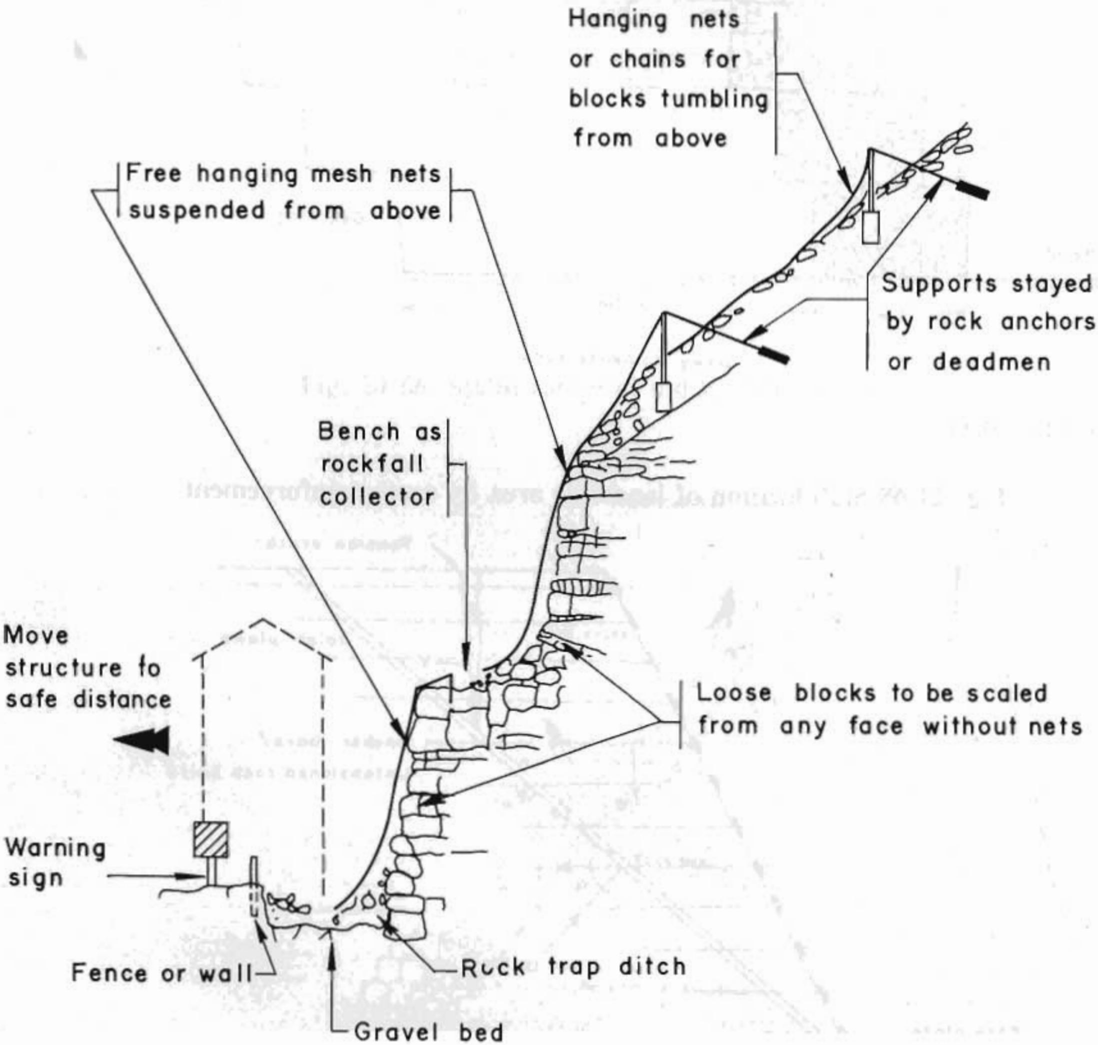


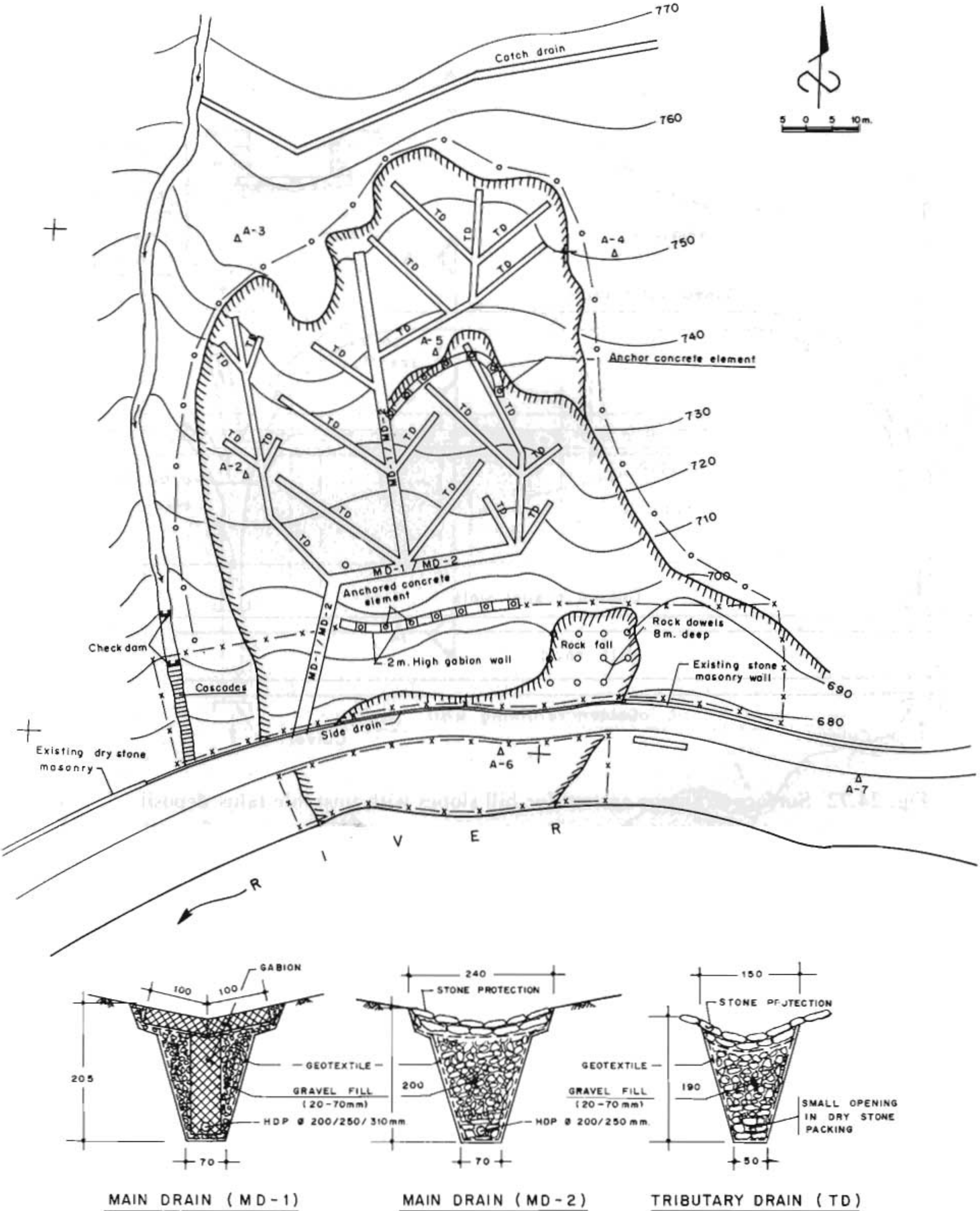
Fig. 24.69 Reinforcement of rock cut with anchor bars

Note: (diameter = 40.50mm, spacing = 1.3m, pattern-square, direction-horizontal, FAL = 3-6m drill hole diameter = 60-70mm)



Source: Geotechnical Manual for Slopes, 1981

Fig. 24.70 Rockfall control measures



Source: Road Flood Rehabilitation Project, DOR, HMG Nepal, 1991

Fig. 24.71 Landslide stabilization by sub-surface pipe drains

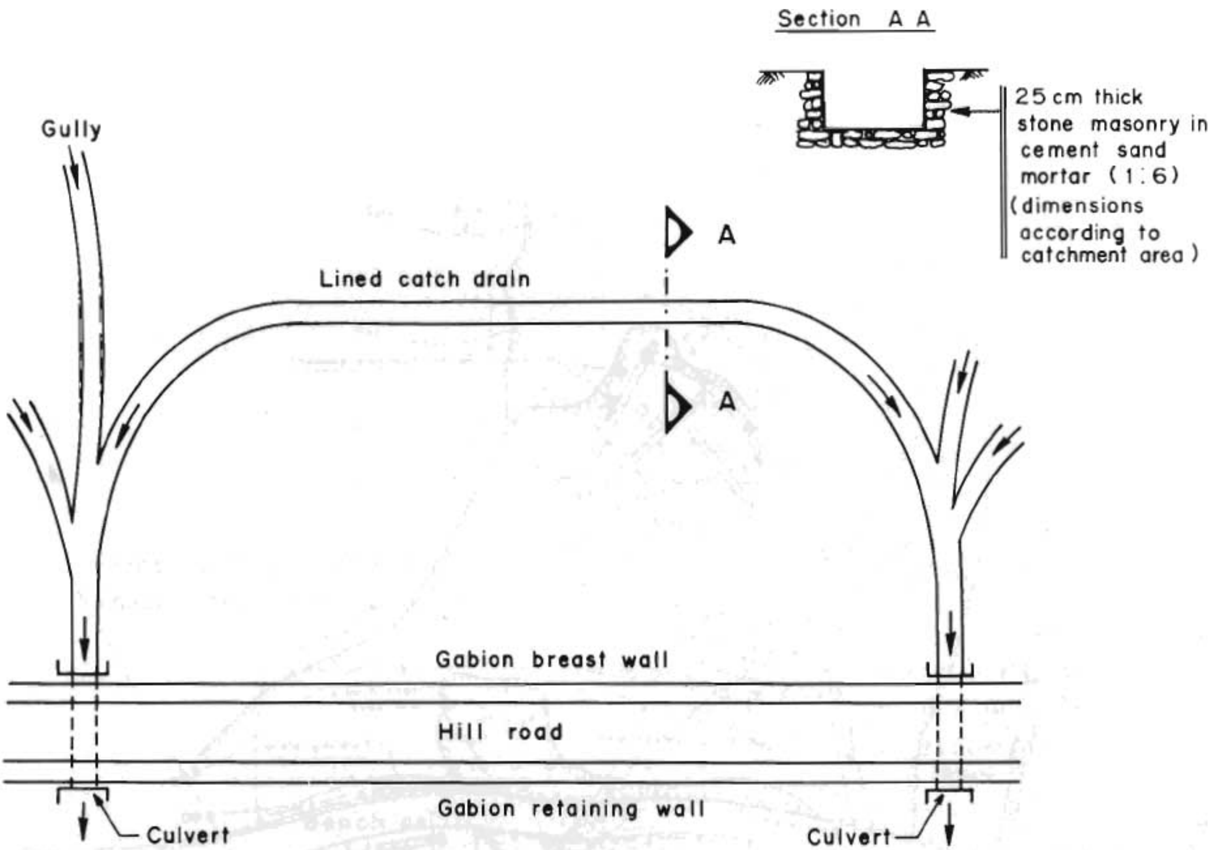


Fig. 24.72 Surface drainage system for hill slopes with unstable talus deposit

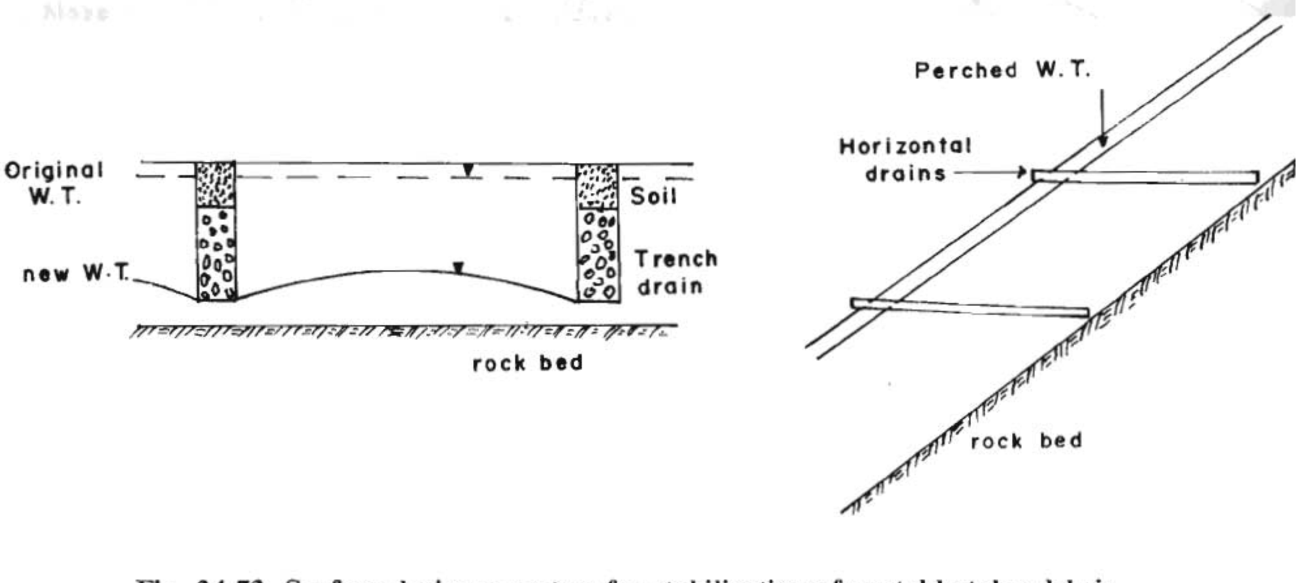
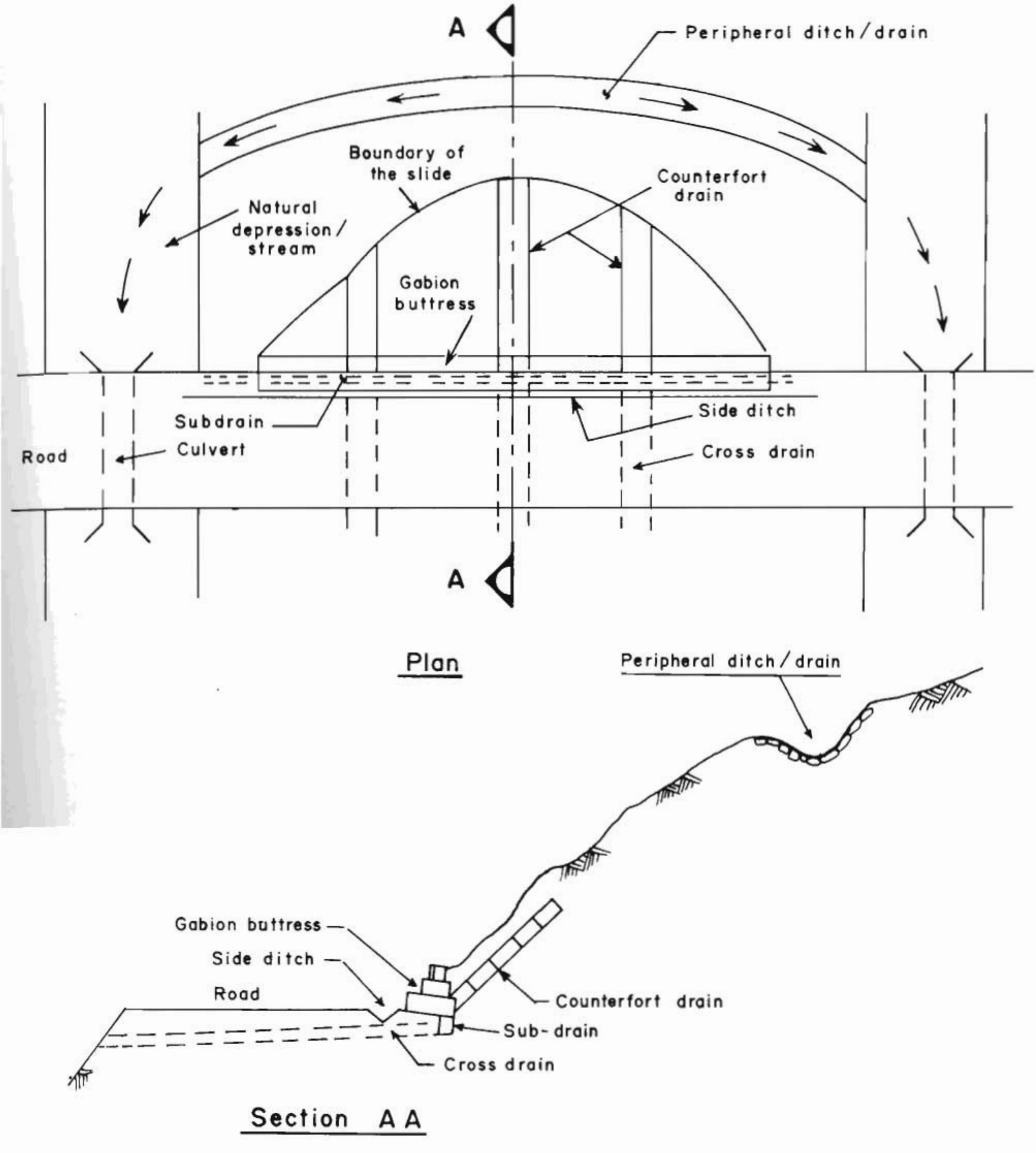
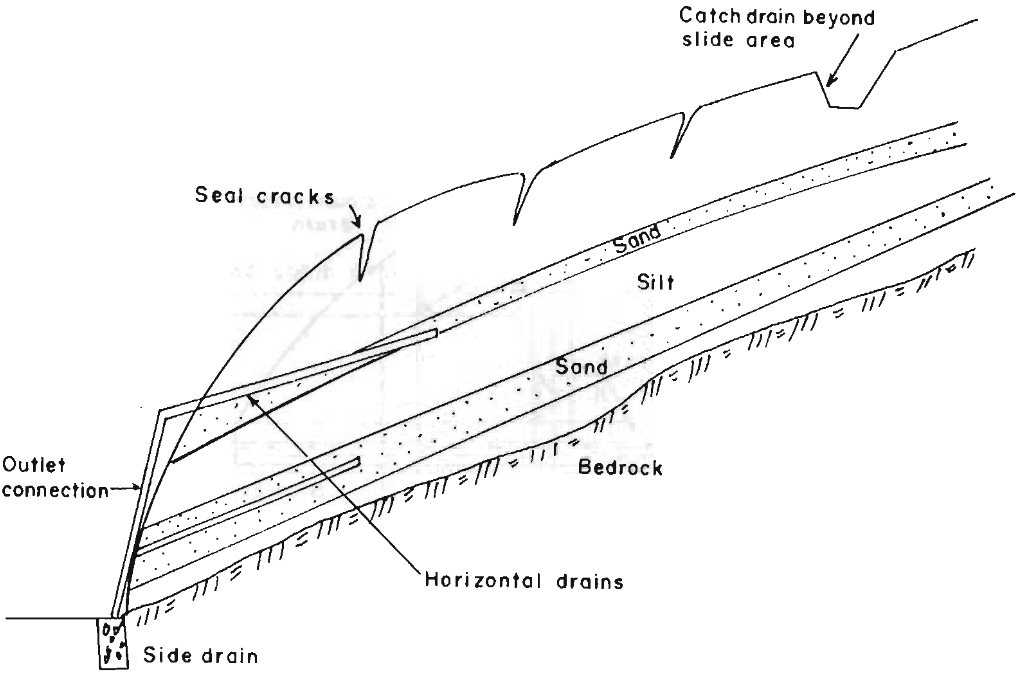


Fig. 24.73 Surface drainage system for stabilization of unstable talus debris



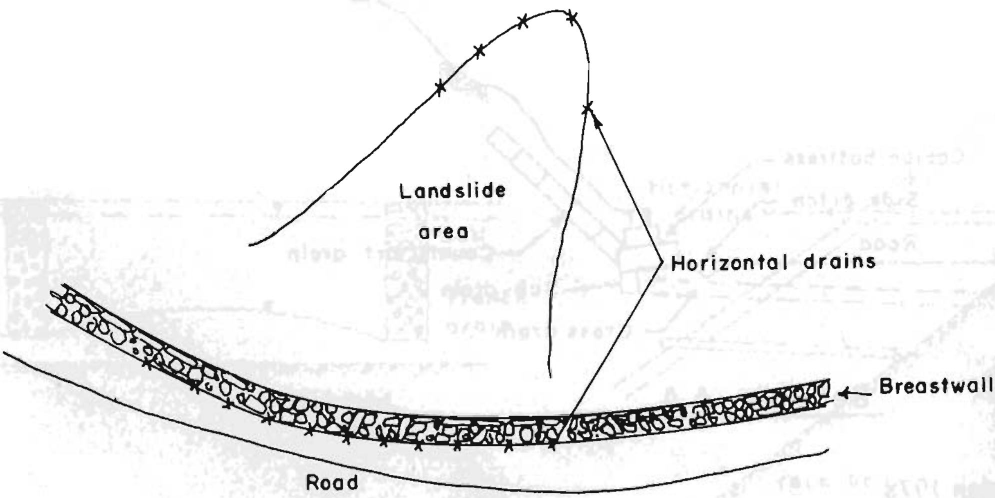
Source: Kojan 1978

Fig. 24.74 Stabilisation of landslide (debris flow) by counterfort French drains
(Western Hill Road, Nepal)



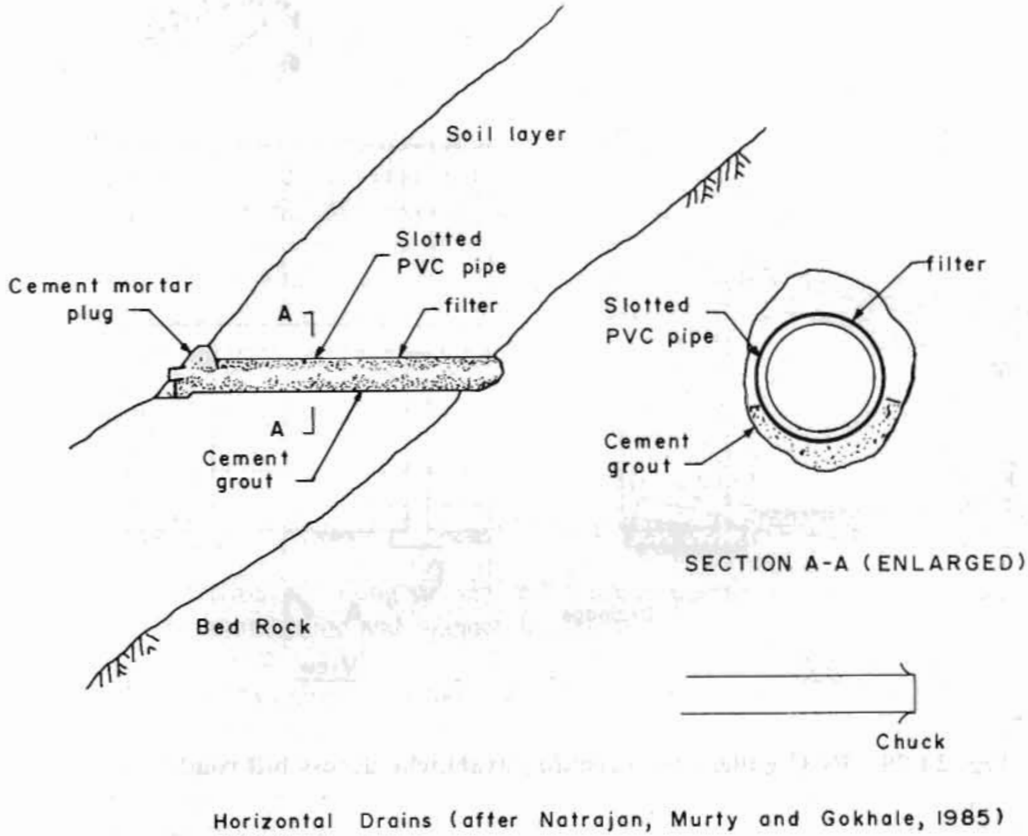
Source: Natrajan et al. 1985

Fig. 24.75 Stabilization of landslides by horizontal drains



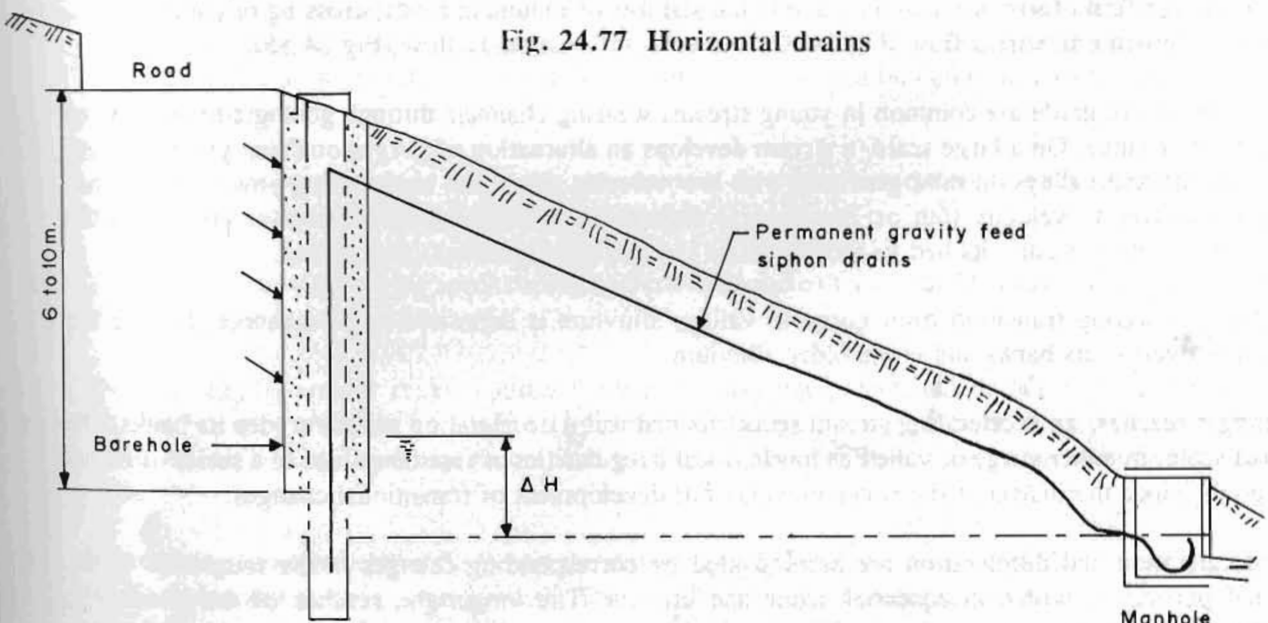
Source: Basker et al. 1985

Fig. 24.76 Planning of horizontal drains in landslide areas, plan of drains not shown



Source: Natrajan et al. 1985

Fig. 24.77 Horizontal drains



Source : Hydro-geo Company, 1990.

Fig. 24.78 Automatic syphon system for vertical drains to stabilize the landslide

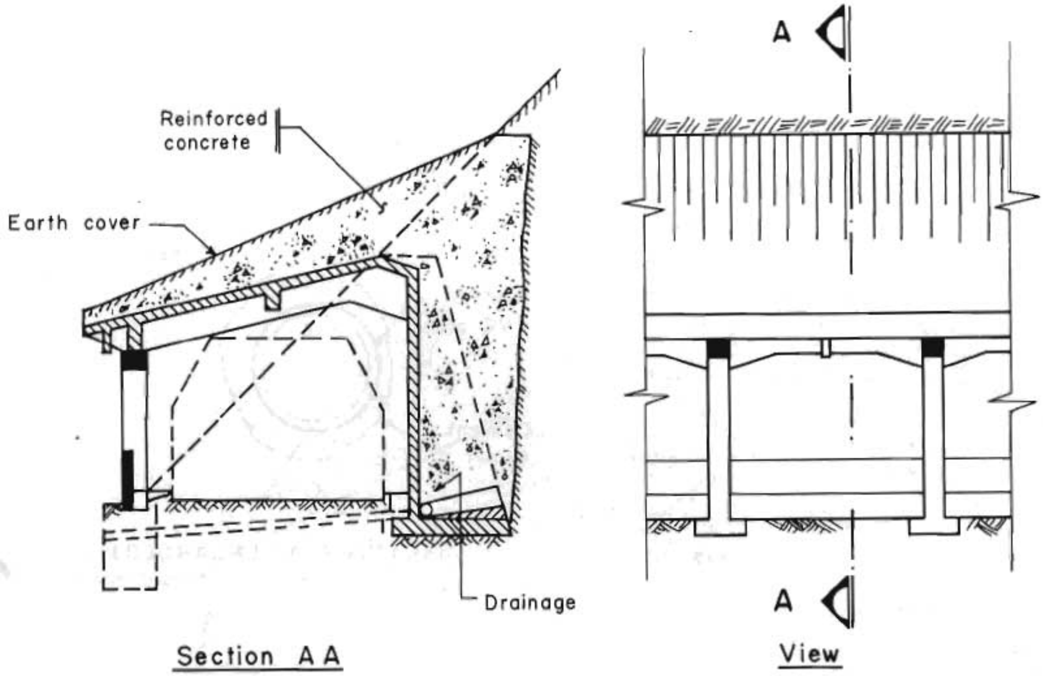


Fig. 24.79 RCC gallery for breaking avalanche across hill road

The most significant form of erosion related to the stability of mountain roads, crossing or parallel to the streams, is erosion in varied flow (Fig 24.82) and erosion in unsteady flow (Fig 24.85).

Natural breaks in grade are common in young streams wearing channels through geologic formations of different resistance. On a large scale, a stream develops an alternation of gorges on steep gradients with high velocity and valleys on mild gradients with low velocity. Since the transporting power of a stream is very sensitive to velocity (6th or 7th power), the transition from valley to gorge is erosive as the accelerating stream scours its bed to accommodate the increase in transporting power.

On the decelerating transition from gorge to valley, alluvium is deposited in the channel, forcing the stream to overrun its banks and erode older alluvium.

On longer reaches, an accelerating stream scours its bed and a decelerating stream erodes its banks. On a small scale, in either gorge or valley channels, local irregularities of resistance lead to a series of riffles and pools, since the inertia of the water prevents full development of transitional changes.

The acceleration and deceleration are accompanied by corresponding changes in the roughness of the channel perimeter, with consequential scour and erosion. The longer the reaches of the alternating roughness, the more serious the hazard. Erosion is especially hazardous at the downstream end of a very smooth reach, whether natural or artificial.

Unsteady flow introduces several contingencies that contribute to erosion hazard. One is the saturation of banks on a rising stage, peeling off soil fragments at shrinkage cracks before capillary tension is restored. On the falling stage, free groundwater springs from the bank, AB, (Fig 24.85) transporting small particles and differential hydrostatic pressure triggers slipouts of weak or unprotected banks, CD, (Fig. 24.85). The transporting power of a stream is very sensitive to velocity, being expressed by some observers as proportional to the 6th powers for size and the 7th powers for volume of solids moved in unit time. On the rising stage, the stream's increasing transport power trains loose bed materials of progressively larger size and uses them to corrode underlying harder materials in a pattern of general scour. Scour near a bank undercuts or steepens the bank, exposing it to erosion. Later, at the falling stage, the stream redeposits the bed materials, progressively by smaller sizes, in layers and bars along the channel.

24.10.2 *Problems*

The following are the problems of river/stream training in mountainous areas.

- o The hydraulic behaviour of young streams in the upper reaches is considerably affected by human activities such as deforestation and channel diversions.

- o The flows of rivers or streams in the mountainous areas of the Hindu Kush-Himalayas are varied and unsteady due to non-homogeneity of bed and bank materials, irregularity of channel sections, landslide-dam or glacial lake outburst floods, cloudbursts, landslides into the river, and tributaries feeding the main river with debris-loaded discharges. Hydraulic characteristics of young streams/rivers in the Hindu Kush-Himalayas can not possibly be assessed from the traditional formulae for mature rivers.

- o Location of a road parallel to the erodible banks of young rivers subjects the road to high hazard and risk since the length of exposure and need for continuous protection is excessive. Either protection can be provided at the points of greatest hazard or protection can be deferred until these points are marked by advancing erosion.

- o Gabion spurs or revetments in the young mountainous rivers are susceptible to early failures by wire breakage from scour and subsequent loss of stones.

- o Gabion or masonry launching aprons are not effective because of scour and because of early breakage of wires and/or loss of stones.

- o River training by armour rocks requires the use of heavy equipment such as bulldozers, and cranes and this is rarely possible because of i) problems of access to a river with deep water and steep banks, ii) non-availability of equipment, and iii) the scarcity of skills to operate the equipment in such works.

- o Construction of revetment works below the scour depth in perennial rivers with deep water requires the diversion and pumping out of water. This is difficult during emergency work.

- o There is no formula to estimate local scour for young rivers.

24.10.3 *Guidelines*

- o Revetments of masonry in cement mortar with foundations below the scour depth, stone masonry up to normal water level, and gabions on the top are the most effective protection in narrow and young mountain rivers. The upstream and downstream end of the hill has to be properly anchored and protected against erosion. See Figure 24.86.

- o Where revetment walls are placed above scour depth, the launching aprons should be made of flexible and heavy duty panels. See Figures 24.86 to 24.88.

- o Determine whether scour will occur or not by assuming the velocity near the structure as 2v, where v is the mean velocity of the river, and assessing whether this 2v velocity of the river, is big enough to move the particle size of the bed material (rock, cobbles, or boulders). Scour holes should be observed wherever possible, and the foundation of the revetment wall should be placed at a depth where the bed material is big enough to be moved.

- o Massive gabion or cement masonry work should not be used for temporary protection.

- o The most ideal method of avoiding expensive river training works on mountain roads is to avoid the problem by setting the road back at an adequate distance from the river. Normally, roads should not be aligned through gorges at less than 60m height from the water level.

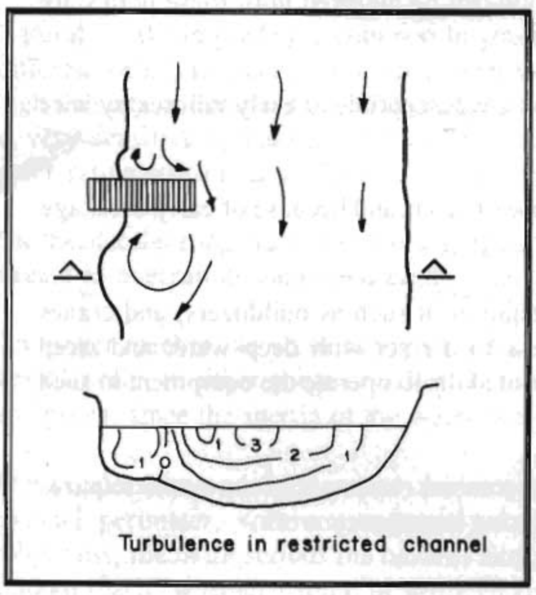


Fig. 24.80 Turbulence in restricted channel (eddy erosion)

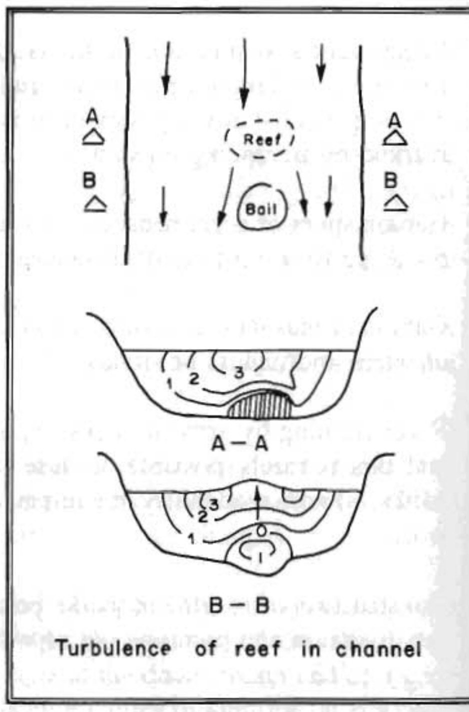


Fig. 24.81 Turbine at reef in channel (Kolk erosion)

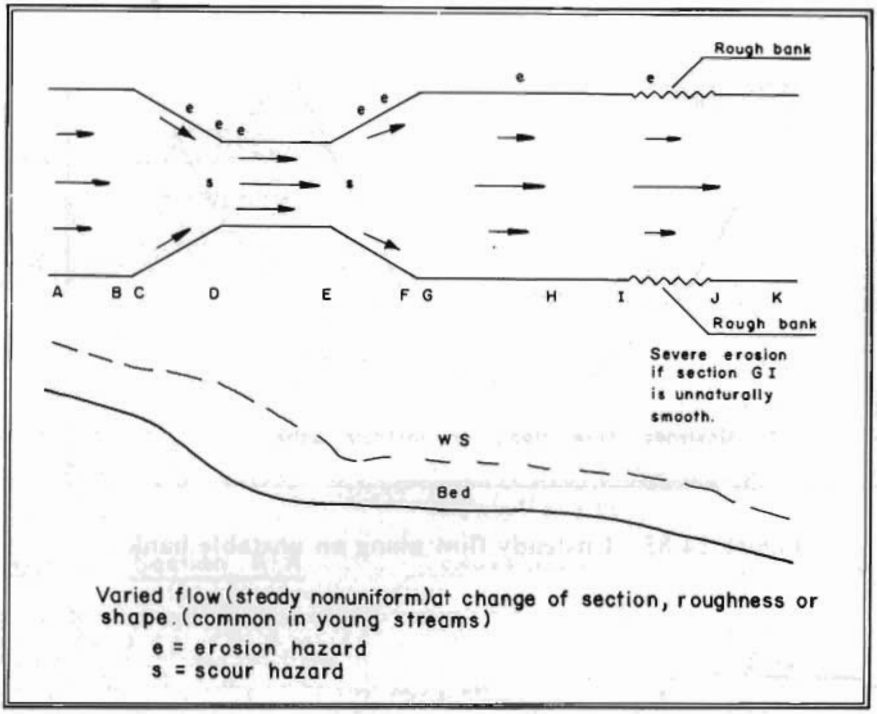


Fig. 24.82 Varied flow (steady non-uniform) at change of section, roughness or shape (common in young streams)

e = erosion hazard; s = scour hazard

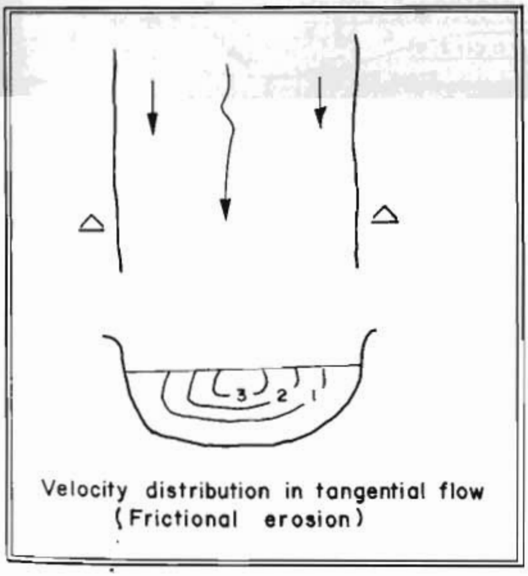


Fig. 24.83 Velocity distribution in tangential flow (frictional erosion)

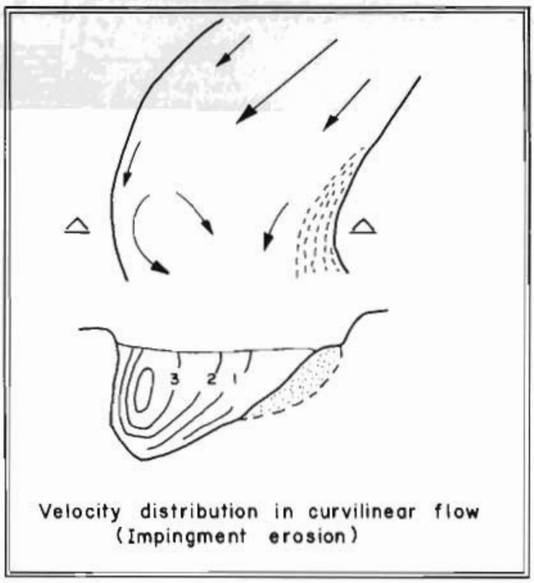


Fig. 24.84 Velocity distribution in curvilinear flow (impingement erosion)

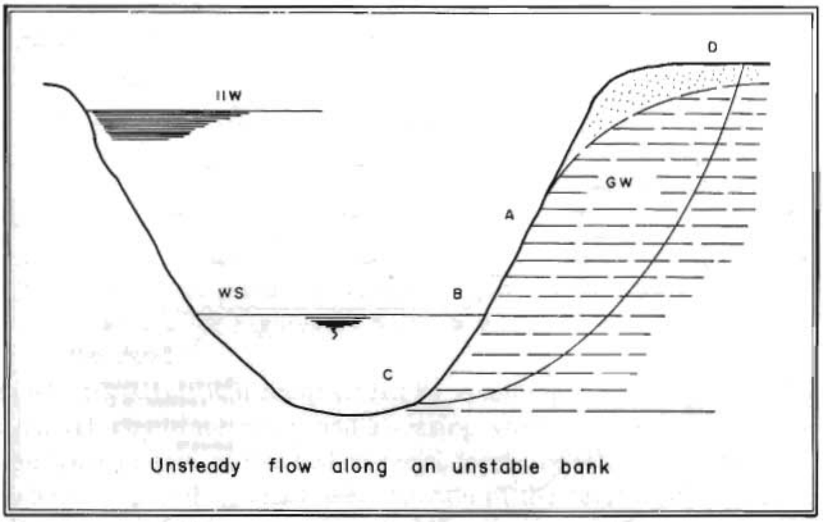


Figure 24.85 Unsteady flow along an unstable bank

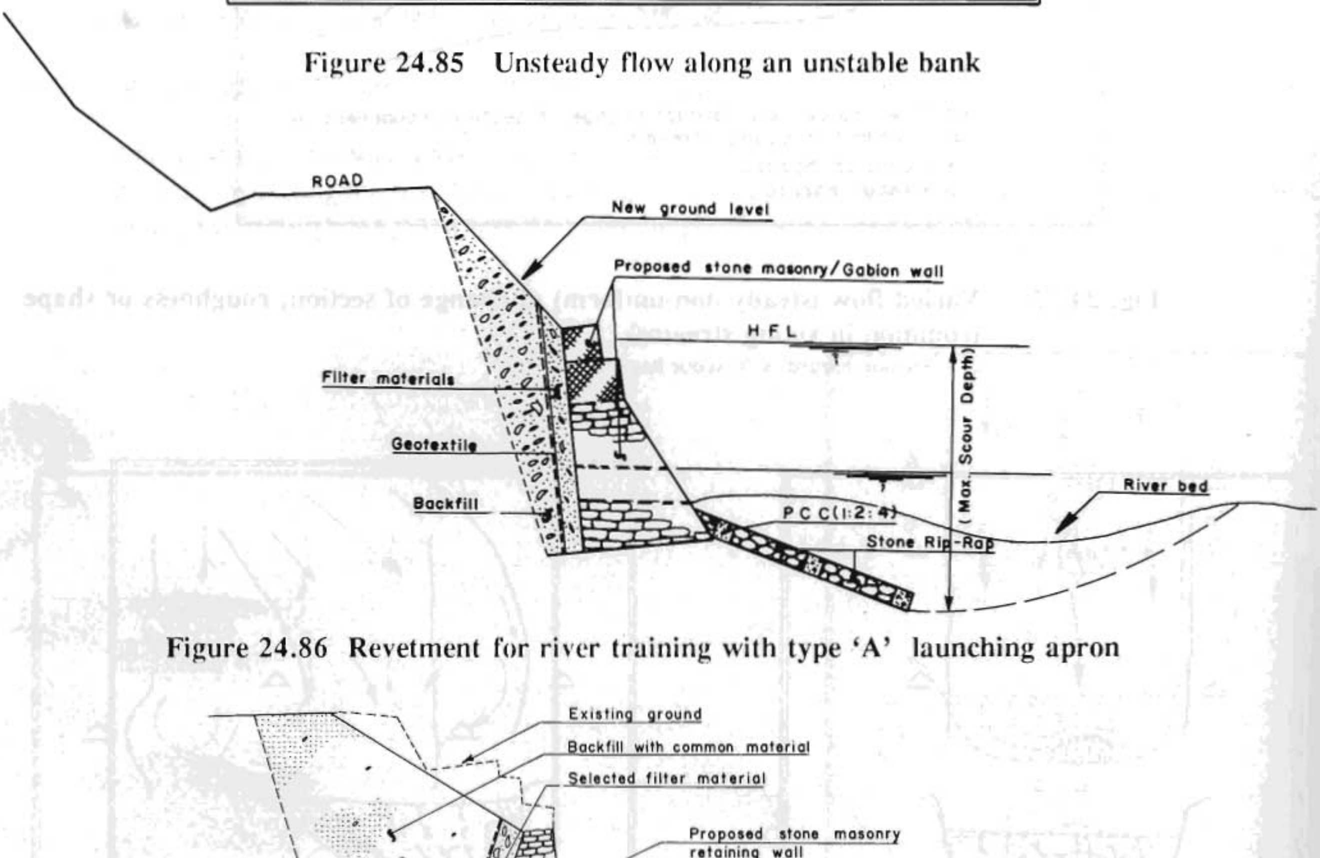


Figure 24.86 Revetment for river training with type 'A' launching apron

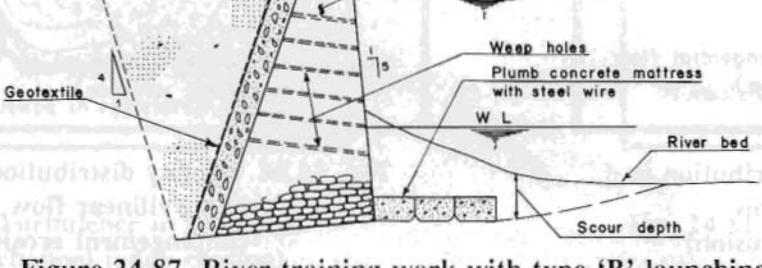


Figure 24.87 River training work with type 'B' launching apron

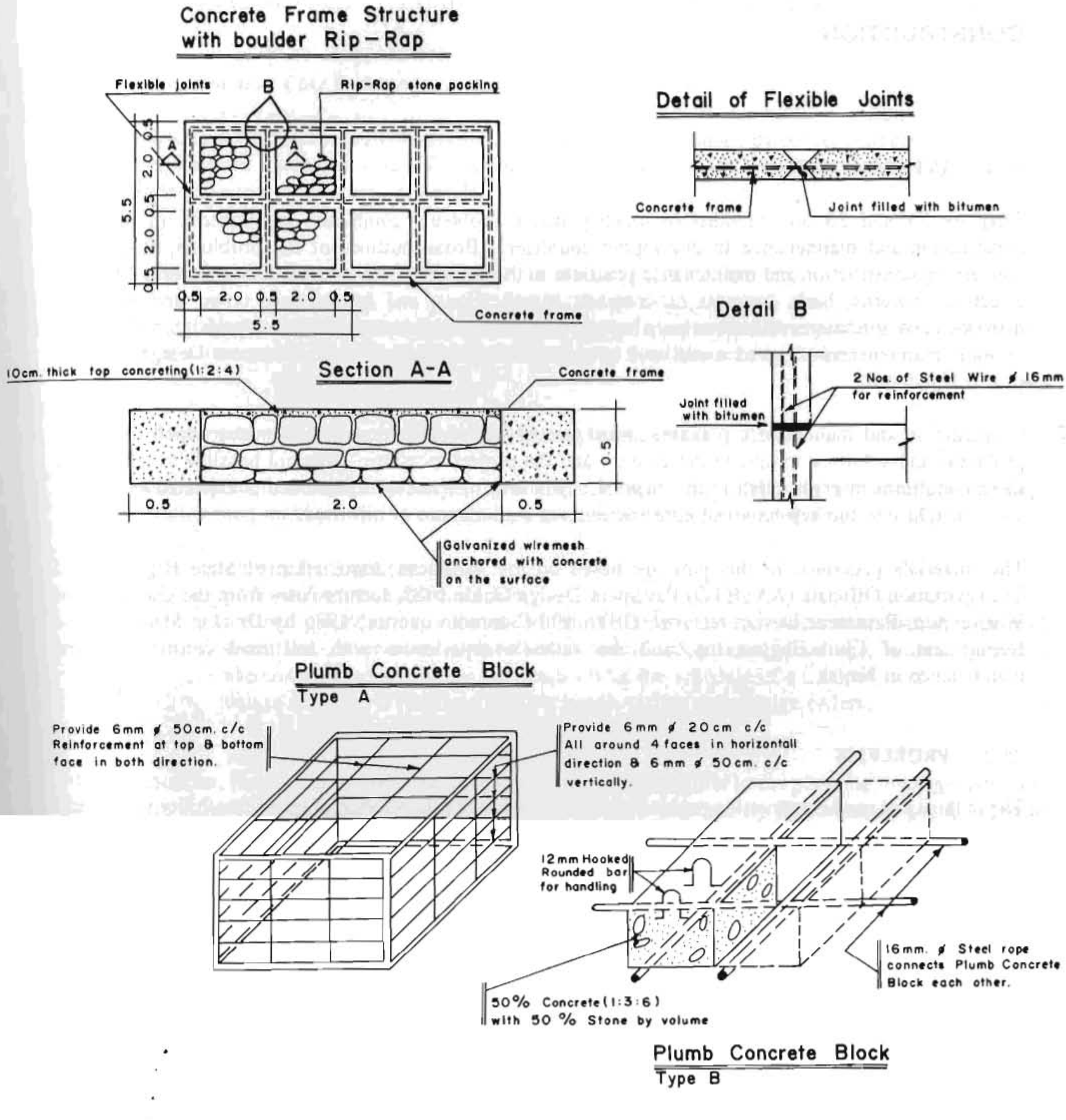


Figure 24.88 River Bank Protection