MOUNTAINEOUS ROADS - SOME IMPORTANT DESIGN AND CONSTRUCTION CONSIDERATIONS

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SUMMARY

Due to the inherent nature of complex topography, geology and environment, design and construction of road system in mountaineous regions demand special considerations. The importance of geological assessment, hydrological study, geotechnical investigation and environmental considerations are highlighted, choice between technical consideration and economy are discussed, and brief descriptions on factors to be considered in stability analyses and methods for slope improvement and stabilization are presented. The paper emphasizes the importance of adequate information collection, proper data interpretation and sound engineering judgement.

1. INTRODUCTION

In Southeast Asian countries, many terrain areas are mountainous with steep and rugged topography. With the rapid economic growth and population increase in recent years, activities and development are inevitably extended to slopelands and mountainous areas, forming many cuts and fill slopes. Landslides and stability of slopes become one of the most important problems in road engineering work.

This article presents some important considerations in the design and construction of roads in mountaineous regions. The scope of discussion is confined to slopes of residual soils, gravelly soils, sandstone-shale formations and some rock formations which are commonly encountered in the Southeast Asian region.

2. FACTORS TO BE CONSIDERED IN DEVELOPMENT OF MOUNTAINEOUS ROADS

2.1 Geological Considerations

The area affected by any highway or road construction stretches for long distances and usually considerable distance laterally. Most mountaineous roads extends into comparatively undeveloped ground. The feasibility, the planning and design, the construction and costs, and the safety may depend critically on the geological conditions of the area. Geological conditions are often so complicated that they do not permit investigations to be conducted as routine work or they may not permit detailed site investigations to be carried out due to the extensive variability. The construction site may be endangered by a geological process of a higher order of magnitude than can be recorded by local boring. The value of a detailed study of the potential construction area by an experienced engineering geologist cannot be neglected. The engineering geologist mediates between the engineer and nature. When examining an area, he should endeavour to conceive its general geological processes that led to its development and the geomorphological forms. He should visualize the effect of the engineering work on the natural environment in the sense whether it might cause an unwelcome acceleration of geological processes. It is important that the engineering geologist is adequately acquainted with the project so that he can correctly assess the factors to be considered.

The geological information is usually supplied in the form of engineering geological report and maps. These information are gathered by the engineering geologist from available geological maps and aerial photographs, plus records of field assessment. In areas where they are not accessible during the planning and preliminary design stage, the engineering report can only be made from available information with the assistance of air photos. However, these information should be re-evaluated during the detail design as well as construction stages.

The types of information which are required about an area or region include the following:

1. Types of geological formation and its history

2. Descriptions of soil and rock exposures

3. Descriptions of ground masses, primarily the discontinuities traversing or dividing the materials

4. Joint survey of rock formations

5. Slope forms and evaluation and the processes at work, e.g. landsliding, stream action, erosion and weathering

6. Slope angles, lengths and heights of both stable and failed slopes

7. Existence of vegetation and evidence of deforestation

8. Existence of special topographic features such as benches, cliffs, proximity to the ocean or to a major stream or to a manmade reservoir.

2.2 Hydrological Considerations

Hydrology is another important aspect which must be carefully considered in the planning and design of roads. Many highway slopes failed due to inadequate drainage to take care of surface runoff and/or subsurface seepage. For mountaineous roads which cover long stretches of areas, regional hydrology rather than local hydrology must be assessed. The catchment area in mountaineous areas can be quite large and demands detail survey. In evaluating effect of seepage and infiltration, not only the permeability characteristics of the soil/rock formations need to be evaluated, but the intensity and duration of rainfall should be considered. This is particularly important in tropical and semi-tropical countries where the rainfalls are usually concentrated with high intensity and short duration. Use of annual precipitation record alone can be quite misleading. Information required to be collected for hydrological studies includes:

1. Annual precipitation, maximum rainfall and duration

2. Year return period

3. Existence of drainage lines, streams, channels, nullahs, ditches, catchpits and culverts with their locations, sizes and conditions

4. Existence and conditions of vegetation and forest coverages

5. Shapes of valley and ridges, their symmetry or asymmetry.

2.3 Geotechnical Considerations

From the geotechnical point of view, it is essential to ascertain the characteristics of soil deposits and rock formation along the proposed road alignment as early as possible. Some of these information should be obtained during the planning and preliminary design stage. Additional ones are aqcuired for detail design. The information required for geotechnical analyses include:

1. The nature of unconsolidated overburden, whether it is talus, colluvium, alluvium or residual soil

2. The depth of each type of soil, the depth of weathered rock, and the extent and nature of weathering

3. The physical and engineering characteristics of the soil and rock, particularly shear strength and permeability

4. The regional watertable and piezometric pressure distribution.

These above information are usually obtained from borings, sampling in-situ and laboratory tests. However, it should be pointed out that borings and testing will only provide specific information at limited localities. The selection of locations for boring, and interpretation of test results for application to a regional problem such as design of mountaineous roads must be made in conjunction with engineering geological and hydrological studies.

Geotechnical considerations would depend to a significant extent on the type of development proposed. For instance, some stability problems may have to be accepted as a matter of economy in developing major roads in mountaineous areas provided that there is no danger to life or property. On the other hand, widening of a hilly road in stretches passing through a highly urbanised area will have to be considered very carefully in order to eliminate any danger to properties, services and even lives. In conventional highway design, the engineer usually adopts the principle of balanced cut and fill for determination of the alignment. With proper consideration of the geological and geotechnical problems, a shift of the alignment could avoid major cuts or fills and thus result in significant savings in the construction cost.

2.4 Environmental Considerations

When a transportation system, like a highway, is built through a region, there are numerous effects on the existing environment. Some of them take place during the construction or immediately after its completion, and some may take years to occur. The major effects on the environment can be divided into:

1. Effect on the natural environment including aesthetics, topograph, vegetation, water resources and quality, etc

2. Effect on the living environment such as noise, vibration, air and water pollution

3. Effect on the social and economic development

4. Effect during construction including noise, vibration, pollution, dump of earth materials, settlement, landslide, and soil erosion.

A geotechnical engineer must be aware of effects of developments which are taking place and will take place on the environment due to the planned construction. Some of the effects may not be immediately obvious and may take considerable period of time to be felt. For example, increase in runoff, and also extent of seepage and infiltration, are often the consequences of land clearing and deforestation. This will lead to surface erosion and porewater pressure. Use of certain form of slope surface protection work will certainly increase the rate of surface runoff.

In short, for a full environmental impact study, a geotechnical engineer must have sufficient vision about the relation between his activity and the environment. The restricted problem-orientated approach adopted by most highway engineers and geotechnical engineers is not sufficient for development of mountaineous roads. Planning and design can be greatly improved by considering the effect of impact which the new construction may have on the environment and the consequences of these impacts on engineering solutions.

2.5 Economical Considerations

In consideration of any project implementation, economy is always one of the important aspects for decision-making. On the other hand, technical soundness and safety cannot be overlooked. It is crucial to reach a realistic balance between the different viewpoints. In development of mountaineous terrain, the question of economy and safety is even more important. It has been accepted by many planners and even engineers that slope failures, especially on cuttings should be "normally expected". A completely no-failure design could be exceedingly costly. In reality, even designs based on detailed site investigations may be liable to unexpected failures due to the inherent variable and unknown nature of the subsurface conditions.

In practical design of mountaineous roads, careful evaluation should be given to the consequences of failures: whether the failures will endanger properties or lives, the effect of traffic disruption and the cost for maintenance and repair. For low-traffic roads in rural areas, it may be cheaper to design the roads with certain amount of risks of failure. On the other hand, for roads of importance, such as expressways with heavy traffic, and vital transportation links for national defence, and roads near urban centers, the adverse effect of disruption to traffic or temporary closure due to slides or failures of the roadway system can be very costly.

As pointed out in an earlier section, proper design of a road system depends greatly on the amount and reliability of information and data collected. In the past, many highway planners had foregone the geological and geotechnical investigation in order to save cost. Due to the lack of information, the engineers either design the project with conservative approach or inadequately. In the latter case, high cost for maintenance and remedial measures would be incurred. This type of "false economy" must be avoided.

The choice between technical consideration and economic consideration must be made by engineers who possess the necessary information, knowledge and experience.

3. GEOLOGY OF VARIOUS TYPES OF FORMATIONS

In the Southeast Asian region, besides sound rock formations, the most commonly occuring geological formations include granitic residual formation, sandstone-shale formation and gravelly soil deposit. The following sections give brief descriptions of these formations. For more information, please refer to papers by BRAND (1984), WOO et al (1981, 1982).

3.1 Granitic Residual Formation

The weathering profile of granitic residual formation varies from soil to rock. Many different types of logging and classification systems have been proposed and used (for example, LITTLE, 1969). One of the most complete and useful system is the one proposed by the the Geotechnical Control Office of the Public Works Department of Hong Kong (1984) to be used for logging weathered soil-rock profiles in Hong Kong according to the grades and zones as described in Table 1. In that system, the grading system is more geologically oriented whilst the zoning system is more based on geotechnical descriptions.

3.2 Sandstone-Shale Formation

The sedimentary sandstone-shale formation is generally characterized by the alternative layers of sandstone and shale. In the sandstone layer, the sand particles are cemented by cementitious material or consolidated under high pressure to form a consolidated rock. The strength properties of sandstone formation are greatly affected by the amount and type of bonding materials in the rock and also its geological history. The formation of shale is similar to sandstone except that it contains mainly fine-grained silt/clay particles, therefore it is more susceptible to the effect of water. Permeability of a shale layer, however, normally is much lower than that of a sandstone layer. Due to tectonic movement of the earth crust, sandstone-shale formations found in mountainous area are often jointed and inclined.

3.3 Gravelly Soil Deposit

Gravels and rock fragments were often carried by ancient rivers in the past and sedimented along the banks or mouths of the river. Subsequently, the sediments were elevated due to tectonic movement of the earth. In a gravelly desposit, filler materials are usually composed of mixtures of sand, silt and clay particles. The gravel deposits may or may not be cemented. Because of the depositional process, many gravels in their "undisturbed" state have definite preferred particle orientations. Since the regular arangement of gravels gives a well interlocking between the particles, many natural slopes and even manmade slopes can stand at a very steep angle at considerable height.

4. MODES OF SLOPE FAILURES

For analyses and design of slopes, the probable mode(s) of failure of the slopes should be considered. Broadly speaking from the kinetic point of view, there are four major types of failures, that is, creep, sliding, flow and fall. The exact mode(s) of failure of a slope is different for different materials and geological conditions. They can be classified as:

Earthfill Slope - (a) Soil flow
(b) Circular slip
(c) Surface erosion

Residual Soil Slope - (a) Planar failure
(b) Shallow non-circular slip
(c) Surface erosion

Gravelly Soil Slope - (a) Toppling
(b) Circular slip
(c) Surface erosion

Sandstone-Shale Slope - (a) Plane failure
(b) Wedge failure
(c) Weathering

Other Rock Slopes - (a) Toppling
(b) Plane failure
(c) Wedge failure
(d) Rock fall

When a loosely compacted earthfill slope is fully saturated with water, the soil may lose all its strength and behave like fluid. This type of quick moving debris is usually called soil flow. In a homogeneous earthfill slope, if the shearing resistance of the soil is not sufficient to resist the gravitational driving force, a circular slip may occur. Erosion occurs on the surfaces of unprotected soil slopes when the surface runoff is so high that it carries away the soil particles. When the surface runoffs are concentrated, eroded gullies are often formed.

Common failures in residual soils are shallow and non-circular in shape. These failures are usually controlled by the depth of infiltration during rainstorms. Slopes of residual soils may fail either in planar failure or shallow non-circular slip. Eroded gullies are also a common phenomenon in residual soil slopes.

Natural undisturbed gravelly soil slopes can stand at a rather steep inclination. Failure of this kind of slope is mainly caused by surface erosion or undermining. The most probable mode of failure is toppling. Disturbed or transported gravelly soil slopes where the gravels are not in an orderly orientation can have circular slip as ordinary soil.

Sandstone-shale formations generally have distinct bedding planes and the rock profiles consist of alternating sandstone and shale layers. Sliding can occur in a planar manner along these bedding planes. Rocks with bedding planes together with joint planes or fissure planes may form a wedge type mass sliding along the intersection of two planes. This kind of failure is called wedge failure. Many sandstone-shale slopes are susceptible to weathering after exposure to air and water. Weathering may change the rock-like material to a soil-like material and slope may fail within the weathered zone.

Failures in unweathered rock are controlled by the joint system. .Toppling is a failure mode of rock slopes involving overturning of interacting columns, formed by bedding planes, cleavage or joints. Rockfalls occur when a mass of rock becomes detached from surrounding bedrock and is free to move downward. Plane and wedge failure in rock slope are similar to that of the sandstone-shale formation as mentioned in the above paragraph.

Typical modes of failures in soil and rock slopes are illustrated in Figs.1 to 3.

5. ANALYSIS AND DESIGN OF SLOPES

A general design and construction procedure for soil and rock slopes is presented in Fig.4. In analysis and design of slopes in residual and colluvium soil deposits, geotechnical approaches are usually used for assessment of landslip potentials. For rock slopes, engineering geological assessment becomes more important. At the present, there is no conclusive rational method of stability analysis which can be applied to analyze gravel-deposit slopes. Combinations of geotechnical and geological assessments, along with engineering judgement are used (WOO et al, 1982).

5.1 Geological Assessment

For a large and difficult site, it is advisable to carry out surface geological mapping if the area is accessible. The methods used when plotting the data collected are usually based on the Geological Society of London Working Party Report (1972) on the Preparation of Maps and Plans in Terms of Engineering Geology. Figure 5 shows an example of such a survey map.

The soil and rock exposures are generally indicated on the geological map under generic names. In addition, they should be fully described on field data sheets for subsequent correlation with the results of subsurface exploration. The following features should be recorded:

(a) Color
(b) Grain size
(c) Texture and structure
(d) Weathered state (for rocks)
(e) Lithological characteristics
(f) Estimate of strength and permeability, and
(g) Other engineering characteristics.

Unlike common soil slopes which may fail along any one of infinite number of surfaces, rock slopes tend to fail along well defined discontinuities, such as fault, joints, fissures and bedding planes. For the purpose of rock slope design, a joint survey should be carried out to collect information on these discontinuities from the natural exposures or from pre-existing manmade slopes. Information pertaining to the discontinuities should include:

(a) dip angle and direction of dip
(b) width, spacing and persistence
(c) roughness of surface
(d) infilling
(e) extent and location of seepage.

The collected survey data are usually plotted onto a stereonet as illustrated in Fig.6. From the equal density contours of pole concentrations and geometry of the slope, the possible mode(s) of failure can be rapidly assessed. Details of the method and its application have been described in HOEK and BRAY (1981). However, use of stereoplots for assessing stability of slopes directs attention only to joint concentrations but not to the isolated single joints. Single joints, which may cause instability, must be individually identified on site.

It should be pointed out that not all sites and potential slopes are accessible or exposed for mapping during the design stage. This is particularly true for highways or mountaineous roads crossing virgin lands. The engineering geologist or geotechnical engineer responsible for the work should also visit the site after the analyse are completed and during the construction stage in order to check the possible presence of joint sets which are not identified during the initial survey and any other random joints which could lead to the development of instability.

5.2 Geotechnical Evaluation

For practical reasons, geotechnical evaluations or analyses are usually carried out at certain specially selected locations, particularly at those places where stability or instability is more critical. Therefore the results of evaluation are limited or confined. The selection of locations for detail analyses is usually made on the basis of information obtained from geological assessment, hydrological study, topographic survey, geotechnical investigation and engineering judgement. Since a road project usually stretches over many kilometers, it is practically and economically unfeasible to analyze every slope which will be encountered. The importance of judgement made by experienced geotechnical engineer cannot be overemphasized.

Before carrying out a detail geotechnical analysis or evaluation of a particular slope or terrain, it is necessary to gather sufficient quantitative data from the site. These data are usually obtained by field investigation, in-situ and laboratory testings. MOH (1984) has discussed the various aspects and techniques of field investigation which are adaptable for mountaineous terrains. Many different methods of field investigation can be employed. The most common method consist of boring and sampling. Standard Penetration Tests are usually carried out to determine the relative consistency of the subsoil strata and piezometers are installed to monitor the variation of subsurface water pressure. At locations where seepage is anticipated to be a major problem, in-situ permeability tests are performed. Laboratory tests are carried out in representative and undisturbed soil samples or rock cores to determine the physical properties, strength and permeability characteristics of the various subsurface strata. In areas where settlement is anticipated to be a problem, consolidation tests are performed.

It should further be pointed out that results from few boreholes are inherently limited. Interpretation and application of these data to an area should be exercised with caution.

5.3 Methods of Stability Analysis

Many methods of stability analysis are available for the design of soil and rock slopes. The most commonly used methods are based on limit equilibrium analyses from which numerical safety factors are obtained. Table 2 lists the better known methods of analysis which are being used to solve soil/rock slopes problems with consideration of water pressure. The advantages and limitations of these methods are given and recommendations are also made to their application.

5.4 Factors to be Considered in Analysis

5.4.1 Shear strength of material

In order to determine the factor of safety of a particular slope against sliding, the strength of materials comprising the slope must be known. Shear strength parameters of the materials are obtained by testing samples of soils and rock joints obtained from the site. The samples should be tested under in-situ stress conditions. In residual soil deposits, because of the high permeability of the material, it is likely that rainwater can infiltrate into the soil and achieve saturated condition at shallow depth. Stability computations should therefore be made in terms of effective stresses. It is generally found that the laboratory measured values of cohesion intercept are quite small and the stability analyses are most frequently carried out with the angle of shearing resistance predominating.

The shear strength values of shale formations

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The shear strength values of shale formations depend upon the clay content, the degree of cementation and the degree of saturation. For this type of formation, the bedding planes or joint surfaces are usually the weak planes where most of the slope failures occur. It is extremely difficult to accurately determine the strength characteristics along these planes. A conservative approach is to determine the residual angle of shearing resistance of samples by direct shear tests along artificially cut surfaces of intact rock specimens to similate bedding planes. Samples soaked with water before testing can represent the actual site conditions better if the potential slip surface were below the groundwater table. In some cases, large scale in-situ direct shear tests have been carried out. But this method is rather costly and sometimes very difficult. If joint-fill materials were found existing in an interbedded rock formation, the shear strength of these joint-fill materials should be determined and used in analysis.

For unsupported slopes, the <u>residual</u> strength parameters should be used, whereas for slopes which incorporate restraining measures, e.g. when the stability of slope is enhanced by prestressed anchors, the peak parameters may be used in the design.

In most rock masses where rock joints are not smooth or planar, the roughness of the surfaces contribute additional shearing resistance. PATTON (1966) expressed the peak shear strength τ of rough rock surfaces in terms of a linear relationship:

$$\tau = \quad \tan(\phi + i) \qquad (1)$$

where $\bar{\sigma}$ = effective normal stress,
$\bar{\phi}$ = angle of shearing resistance of smooth rock surface, and
i = angle of roughness of rock surface.

The variation of shear strength of a rough rock joint is usually non-linear in relation to the effective normal stress. BARTON (1971) proposed the following equation for the peak strength of rough joint surfaces:

$$\tau = \bar{\sigma} \tan \left(\bar{\phi} + \mathrm{JRC}\ \mathrm{Log}_{10} \frac{\sigma_j}{\bar{\sigma}}\right) \qquad (2)$$

where JRC = 20 for rough undulating joints,
JRC = 10 for smooth undulating joints,
JRC = 5 for smooth nearly planar joints, and
σ_j = the uniaixal compressive strength of the rock material adjacent to the joints.

(Note: the maximum value in brackets is 70 degrees).

5.4.2 Groundwater

The stability of slopes is seriously affected by the groundwater pressure. For all types of slopes, the groundwater conditions should be monitored during the site investigation and afterwards by means of piezometers. The groundwater pressures measured within a short period of time may not represent the highest or critical groundwater pressures in the slope for long term stability. The groundwater condition in a particular slope is dependent on the rainfall intensity and duration, the infiltration capacity of the slope forming material, slope topography, extent of vegetation cover, and environment.

For design of residual soil slopes in Hong Kong, estimate of rise in water table after a rainstorm is determined by the wetting band as suggested by LUMB (1962). The wetting band or zone of saturation caused by a rainstorm will extend downwards from the ground surface under the effect of gravity. The relationship between rainfall, infiltration of unprotected slope and thickness of wetting band proposed by Lumb is:

$$h = \frac{k_t}{n\ (S_f - S_o)} \tag{3}$$

where h = depth of wetting band,
k = coefficient of permeability,
n = porosity,
Sf, So = final and initial degree of saturation,
t = duration of rainfall, for rainfall intensity of two times of the permeability.

Based on the calculated wetting band, thickness is determined from rainstorms of different return periods (Fig.7) together with records of groundwater level, estimation on the probable rise in groundwater level can be made. The recommended return period for Hong Kong condition is 1 in 1,000 years.

In sandstone-shale slopes, perched water table normally exists because of the different permeability of the sandstone and shale layers. Figure 8 illustrates the probable water tables of an inclined sandstone-shale formation. To determine the groundwater pressure in this type of slope, piezometers should be installed in every layer of the rock formation at a number of locations.

There are many cases showing that the groundwater table in a slope can change due to environment changes. For example, on the banks of a reservoir, the groundwater level rises after storing of water and fluctuates with the water level in the reservoir. This phenomenon is shown in Fig.9. Another example is seen in Fig.10 for a highway embankment crossing a valley. If the drainage capacity of the culvert becomes insufficient because of blockage,or other reasons, a temporary pond will be formed at the intake of the culvert. The ponding water may create seepage through the embankment which may be originally designed without any consideration of excessive water seepage.

5.4.3 Factor of Safety

The minimum factor of safety against failure which can be accepted for a slope will depend on the risk of life, cost of consequence associated with the failure of that slope. The choice of an appropriate factor of safety in the slope design is subjected to the judgement of the designer. Table 3 presents some suggested values of safety factor for slope design. In the application of a factor of safety to slope stability, the possible change in groundwater pressure and influence of external loading should be considered. In design of a slope, it is a good practice to check the sensitivity of the factor of safety against each parameter. For example, the factor of safety of a slope may increase 100 per cent between saturated and drained condition, this indicates that the use of drainage would be effective to control the instability of that slope.

The stability of a slope is also affected by dynamic loading like earthquake and blasting. The response of a slope to an acceleration can be simplified to a horizontal force acting on the center of gravity of the potential sliding mass towards the free face of the slope. Since the dynamic force occurs only in a short duration, a lower factor of safety under dynamic condition is acceptable. HOEK (1976) suggested a reduced factor of safety of rock slope be:

$$FD = Fs - 2.3\ a \qquad (4)$$

where FD is the factor of safety under dynamic condition,
Fs is factor of safety without any applied acceleration, and
a = horizontal acceleration in terms of the acceleration g due to gravity.

5.5 Factors to be Considered in Design

In designing roads through mountaineous regions, besides alignment, the one most important feature in design of slopes, cut or fill. The gradient or angle of a cut slope is generally determined by one of the following two approaches:

(i) from geological examination and geotechnical investigation - this approach has been discussed in the previous section

(ii) from past experience in similar subsurface formations - this is an empirical approach.

In addition, there are several other important considerations which deserve attention in practical design work. They are:

1. In soil cuttings if the bedrocks dip steeply into the cuttings, sliding of the entire overburden above the cuttings along the bedrock surface may occur.

2. The surface of cut slopes will be subjected to weathering and erosion.

3. When empirical approach is adopted for design cut slopes, the potential of "insignificant" or "uncritical" slides may be allowed. In those cases, wider right of way should be provided for catching debris from slips.

4. In design of embankment fills, it is important that the fill materials are properly keyed in with the original ground. It is particularly critical in cases of half cut-half fill sections. Benching may be necessary.

5. The toe area of an embankment fill is an inherently weak zone and requires special attention for strengthening.

6. In designing drainage facilities for both cut and fill slopes, the drains should be designed in such a way that they will be self-cleaning and require minimum amount of maintenance.

7. It is important to recognize that design of slopes for mountaineous roads needs to be "flexible". In other words, it is more often than exceptional that revision of design will be necessary during construction. The degree of this need depends upon the amount of geological and technical information available during the design stage. Additional site investigation should always be considered and included in the design tender document.

6. CONSTRUCTION OF SLOPES

6.1 Formation of Soil Slopes

As pointed out in an earlier section, it is a common practice for highway planners to adopt the principle of balanced cut and fill. Theoretically, this approach may be more economical. However, from the engineering point of view, this approach may not be desirable particularly in areas where the slope formations will be partly in cut and partly in fill. These slopes are potentially unstable and difficult to construct. It is more preferable wherever possible to form the slope either entirely in cut or entirely in fill. Realignment of the roadway may prove to be more stable, trouble free, and easy to construct. Economy in the long run should be taken into consideration when designing roads in mountaineous areas.

Cut slopes in residual soils are usually made at 40 to 60 degrees to the horizontal, depending on the expected duration of the slope. Fill slopes are normally formed at gradient ranging from 27 to 33 degrees. Each flight of the slope is usually 8 to 10m high and separated with 1.5 to 2.0m wide intermediate berm.

The construction of fill slope should be strictly under engineer's control. The fill materials should consist of well-graded soils, free of organic matters, having 20 to 40 per cent by weight of fines, and with uniformity coefficient not less than 50. The fill is normally placed in layers of 30 to 50cm in thickness and compacted to at least 95 per cent of the maximum dry density determined by standard proctor test. To ensure that the surface of the slope has also compacted to the required degree of compaction, the slope is normally slightly over-built by about 1m, and then trimmed back after compaction to the required profile. Vibratory rollers are commonly used for compaction.

6.2 Surface and Subsurface Drainage for Soil Slopes

It is of prime importance to install adequate surface drainage system to collect and safely convey the run-offs from catchment areas above and adjacent to the slope and from the slope itself to an appropriate point of discharge. For this reason, concrete surface drainage channels should be provided along the top, the toe and along each berm of the slope. Herring bone drainage system consisting of chevron drains and stepped channel is constructed along the slope surface to reduce the flow velocity of surface run-off, thus minimizing soil erosion.

In areas where the subsurface porewater pressures are high or where the potential of seepage and infiltration of water is great, subsurface drains should be installed. The subsurface drains are usually constructed of perforated PVC pipes installed at an inclination to the slope surface. In order to maintain proper functioning of the subsurface drains, the drain pipes must be adequately protected with filter material to prevent clogging up the holes by the surrounding soil particles.

6.3 Surface Protection of Soil Slopes

In order to prevent soil erosion and minimize infiltration of water into the slopes, the slopes should be protected with grassing or covered with impermeable layers as soon as possible after formation. Grassing can be placed by means of turfing or hydroseeding, together with fertilizers on the slope surface. When hydroseeding is to be carried out in wet seasons, it is suggested that the seeding be applied together with erosion control fabrics or straw mats. Apart from reduction of erosion on slopes during the wet seasons, other advantages of using fabrics are the discouraging of birds from eating the germinating seeds, retention of moisture during droughts and prevention of dislodging seeds due to careless watering.

Other types of surface protective measures include chunaming, guniting, stone-pitching and concrete revetment. Chunaming which is very commonly used in Hong Kong is the placement, usually by hand, of a plaster of cement-lime stabilized soils available locally in layers to a thickness of 5 to 8cm on the slope surface. Being more durable and less susceptible to cracking problem, guniting is the spraying of cement-sand mixture in mortar form onto the slope. The thickness of guniting layer is commonly 5 cm but sometimes is increased to 7 to 10cm when reinforcements are incorporated. The concrete revetment consists of precast or cast-in-place interlocking grids filled with a layer of no-fine concrete, onto which top soil and turf are placed.

6.4 Soil Slope Improvement

Sometimes it is impossible to design stable cut slopes which will meet the development requirement of a particular site, especially in steep urban terrains where the economic use of land is of major importance, or in rural areas where it is considered to be more economical to accept a higher probability of failure in cut slopes and to rectify failures when they occur, rather than to design stable slopes at the outset. The latter case is particularly justified for low cost roads which accommodate small traffic volumes. Slope improvement or rectification measures are therefore necessary for many instances:

1. Flattening of side slope - This is the most effective means to increase the stability of slopes, if geometric constraints allow. The principle is simply to reduce the weight of the soil/rock mass which has a sliding potential.

2. Subsurface drainage - The long-term stability of steep slopes in areas with shallow GWL can be enhanced by modifying the seepage pattern in the slope, thus suppressing the development of unfavorable seepage force and diminishing the pore pressure acting on the potential slip surface. This can be achieved by installing inclined drain pipes, drainage blankets and galleries with sandwiched filter materials into the slope to intercept the ground or perched water in subsoils and discharge it out from the slope, thereby relieving any potential of hydraulic pressure build up. It is normally required to place a drainage blanket between the original ground and fill in embankment construction.

3. Stabilizing piles/caissons - Another means to increase the stability of the slopes is to construct a series of stabilizing piles/caissons on the slope to provide additional resistance against sliding through pull-out and dowel actions. It is preferred to have the piles/caissons keyed into hard soil stratum and spaced at least one diameter apart to avoid any damming effect on the upslope areas.

4. Retaining structures - Due to aerial constraint, structures are sometimes needed to support the slopes and these take the forms of conventional reinforced concrete cantilevered wall, contiguous bore-piled wall, either cantilevered or supported by ground anchors, crib walls, gabbion walls and buttress wall.

6.5 Stabilization for Rock Slopes

Rock cuts are usually made at 60 to 75 degrees to the horizontal, depending on the nature of rock mass, with 1m wide benches at every 50 ft height of slope. Excavation is normally carried out by blasting in unrestricted remote areas and by pneumatic drills in restricted urban areas. The design of blasting should be aimed at achieving a good production of well fragmented rocks with least damage to the rock slope left behind. Blasting, if not properly controlled, will cause back-break, reducing the strength of joints and thus causing rock falls, and instability of the finished slope surface. In order to eliminate unnecessary remedial works caused by overblasting, controlled blasting technique using presplitting should be commonly employed for the final face of the rock cut slope, and the primary blasting is carried out to a distance of about 15 ft from the final face. Presplitting blast holes are loaded with light, well distributed charges completely stemmed and fired simultaneously before any adjoining main excavation area is blasted. Delay blasting technique is strongly recommended.

After the formation of rock cut slopes, an engineering geological survey should be carried out to determine the overall stability of the cut slope, the stability or instability of localized zones, joints and rock surfaces. If potential instability were found, remedial measures will be necessary. The following lists some of the most commonly used methods for stabilizing rock slopes:

1. Drainage system and surface protection - Where applicable are similar to that for soil slopes (Fig.11).

2. Joint plane stabilization - After the final cut face is formed, if daylighting joint planes which can form a potential failure wedge are identified, the potential sliding rock mass can be arrested by the use of dowel and Shear bars, rock bolts and anchors.

3. Surface strengthening - For highly fractured and uneven rock faces, either inherent or as a result of overblasting, it is sometimes necessary to cover the slope surface with gunite or shotcrete, preferably with reinforcement, to prevent further weathering and to arrest fall of rock debris.

4. Fencing and/or netting - In areas where it may not be economically practical to eliminate rock falls from a cut face, protective fence or netting can be erected to keep the rock fragments from falling onto the site.

FOOKES and SWEENEY (1976) summarized some appropriate stabilization measures for different rock slope failure patterns. This is given in Table 4 and Figs.12 and 13. Among the various techniques, the control of water and drainage is normally most effective and economical.

7. CONCLUDING REMARKS

This paper describes some of the important considerations for designing roads in mountaineous regions. Due to the very nature of the complex topographic and geological conditions of mountaineous areas, the engineer responsible for planning and design of mountain roads should not lose his vision by considering the engineering aspect alone. Geological, hydrological, geotechnical, environmental as well as economical considerations all play important roles. In the practical design and construction work for mountain roads, adequate data collection, proper evaluation and interpretation of data with sound engineering judgement are essential elements for a safe but economical construction project.

8. REFERENCES

Barton, N. (1971), A Relationship between Joint Roughness and Joint Shear Strength, Proc., International Symposium on Rock Fracture, Nancy, France, p.1-8.

Bishop, A.W. (1955), The Use of the Slip Çircle in the Stability Analysis of Slopes, Geotechnique, Vol.V, pp.7-17.

Bishop, A.W. and Morgenstern N.R. (1960), Stability Coefficients for Earth Slopes, Geotechnique, Vol.X, pp.129-150.

Brand, E.W. (1984), Landslides in Southeast Asia, A State-of-the-Art Report, Proc., 4th Int'l Symposium on Landslides, Toronto, pp.1-43.

Fookes, P.G. and Sweeney, M. (1976), Stabilization and Control of Local Falls and Degrading Rock Slopes. The Quarterly Journal of Engineering Geology, Vol.9, pp.37-55.

Geological Society of London (1972), The Preparation of Maps and Plans in Terms of Engineering Geology, Geological Soc. Working Party Report, Quart. J. Engrg. Geology, Vol.5, pp.295-382

Geotechnical Control Office (1984), Geotechnical Manual for Slopes, 2nd Ed., Public Works Dept., Hong Kong.

Goodman, R.E. and Bray, J.W. (1976), Toppling of Rock Slopes, Proc. Specialty Conf. on Rock Engineering for Foundations and Slopes, ASCE, Colorado, Vol.2, pp.201-234.

Hoek, E. (1976), Rock Slopes, Proc, Specialty Conf. on Rock Engineering for Foundations and Slopes, ASCE, Colorado,Vol.2, pp.157-171.

Hoek, E. and Bray, W.J., (1981), Rock Slope Engineering, Institution of Mining and Metallurgy, Revised 3rd Ed., Asian Ed., MAA Publishing Co. Ltd.

Janbu, N. (1972), Slope Stability Computations, Embankment Dam Engineering, Casagrande Volume, Ed. R.C. Hirschfield and S.J. Poulos, Wiley, New York, pp.47-86.

Lambe, T.W. and Whitman, R.V. (1969), Soil Mechanics, Wiley, New York, 553 pp.

Little, A.L. (1969), The Engineering Classification of Residual Tropical Soils, Proc. Specialty Session on Lateritic Soils, 7th ICSMFE, Mexico City, Vol.1, pp.1-10.

Lumb, P. (1962), Effect of Rainstorm on Slope Stability, Proc., Symposium on Hong Kong Soils, pp.73-78.

Lumb, P. (1975), Slope Failures in Hong Kong. The Quarterly Journal of Engineering Geology, Vol.8, pp.31-65.

Moh, Z.C. (1984), Site Investigation, a paper presented at Seminar on Design and Construction of Roads in Mountaineous Terrain in Malaysia, organized by Jabatan Kerja Raya, Peninsula Malaysia.

Moh and Associates (1978), Report on Study and Design for Slope Stabilization at 14k and 16k of the Taiwan N-S Freeway, Report submitted to the Taiwan Area Freeway Bureau, Taipei.

Moh and Associates (1973), Report on Geotechnical Study for an Existing Cut Slope at Mile 21, Jeli Side, E-W Highway, West Malaysia, Report submitted to Malaysian Thai Development Sdn Bhd.

Morgenstern N.R. and Price, V.E. (1965), The Analysis of the Stability of General Slip Surfaces, Geotechnique, Vol.15, pp.79-93.

Patton, F.O. (1966), Multiple Modes of Shear Failure in Rock, Proc., 1st Congress of ISRM, Lisbon, Vol.1, pp.509-513.

Woo, S.M., Moh, Z.C., Yu, K. and Guo, W.S. (1981), A Study on the Causes of Some Rock Slope Failures Along Highways in Taiwan, Proc., 3rd Conf. Road Engineering Assoc. Asia and Australasia, Taipei, pp.761-777.

Woo, S.M., Guo, W.S., Yu, K and Moh, Z.C. (1982), Engineering Problems of Gravel Deposits in Taiwan, Proc., Specialty Conf. on Engineering and Construction in Tropical and Residual Soils, ASCE, Honolulu,Hawaii. pp.500-518.

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Table 1(a): Weathering Grade Classification System Recommended for Use in Hong Kong by the Geotechnical Control Office (1984)

GRADE	DESCRIPTION	TYPICAL DISTINCTIVE CHARACTERISTICS
VI	Residual Soil	Soil formed by weathering in place but with original texture of rock completely destroyed.
V	Completely Decomposed Rock	Rock wholly decomposed but rock texture preserved. No rebound from N Schmidt hammer. Slakes readily in water. Geological pick easily indents surface when pushed.
IV	Highly Decomposed Rock	Rock weakened - large pieces can be broken by hand. Positive N Schmidt rebound value up to 25. Does not slake readily in water. Geological pick cannot be pushed in surface. Hand penetrometer strength index >250 kPa. Individual grains may be plucked from surface.
III	Moderately Decomposed Rock	Completely discoloured. Considerably weathered but possessing strength such that pieces 55mm diameter cannot be broken by hand. N Schmidt rebound value 25 to 45. Rock material not friable.
II	Slightly Decomposed Rock	Discoloured along discontinuities. Strength approaches that of fresh rock. N Schmidt rebound value greater than 45. More than one blow of hammer to break specimen.
I	Fresh Rock	No visible signs of weathering: not discoloured.

Table 1(b): Weathering Zone Classification System Recommended for Use in Hong Kong by the Geotechnical Control Office (1984)

ZONE	DESCRIPTION OF WEATHERING ZONE
A	Structureless sand, silt and clay. May have boulder concentration at the surface.
B	Residual material with corestones. Rock percentage is less than 50% and corestones are rounded and not interlocked.
C	Corestones with residual materials. Rock percentage is 50 to 90%, and corestones are rectangular and interlocked.
D	More than 90& rock. Minor residual material along major structural discontinuities which may be considerably iron stained.

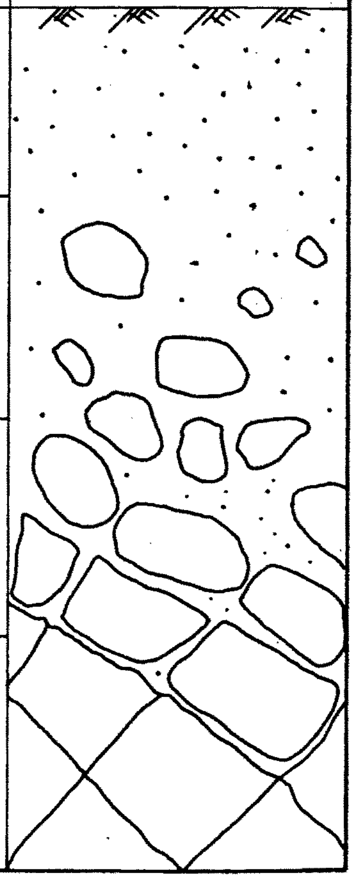


Table 2(a): Stability Analysis Methods (Soil)

METHOD	FAILURE SURFACE	ASSUMPTIONS	ADVANTAGES	LIMITATIONS	REFERENCE	RECOMMENDATION
Hoek's Charts	Circular	Sliding mass considered as a whole. Lower bound solution, assuming normal stresses are concentrated at one point	Slope angles from 10^{o} to 90^{o} given. Very simple to use.	Limited to homogeneous soils and five specified groundwater conditions.	Hoek and Bray (1981), p.312	Very useful for preliminary calculations or for small low risk slopes.
Bishop and Mogransterns Charts	Circular	Uses Bishop's simplified method with an average r_u value.	Simple to use. More accurate than Hoek's charts.	Limited to homogeneous soils and slopes flatter than 27^{o}	Bishop and Mogannstern (1960)	Limited usefulness.
Janbu	Non-Circular	Generalised procedure considers force and moment equilibrium on each slice. Assumptions on line of action of interslice forces must be made. Vertical interslice forces not included in Routine procedure and calculated F then corrected to allow for vertical forces.	Realistic shear surfaces can be used. Routine analysis can be easily handled by a programable calculator or by hand.	Published f_o factors are for homogeneous materials and routine procedure can give large errors in slopes composed of more than one material. Factor of safety is usually underestimated in these cases. Generalised method does not have the same limitations.	Janbu (1973) Routine method given in Hoek and Bray (1981) pp.247-253.	Very useful for the majority of soil slopes in Hong Kong. Limitations of routine method must be considered.
Morganstern and Price	Non-Circular	Considers forces and moments on each slice, similar to Janbu Genralised procedure.	Considered more accurate than Janbu. Computer programs readily available.	No simplified method. Computer solution necessary, often very time consuming.	Morganstern and Price (1965)	Usually unecessarily detailed for HK soils where strength and porepressure are not known with accuracy. Most useful for back analysis of land-slides.

Table 2(b): Stability Analysis Methods (Rock)

METHOD	FAILURE SURFACE	ASSUMPTIONS	ADVANTAGES	LIMITATIONS	REFERENCE	RECOMMENDATION
Plane Failure	Single plane with tension crack	Both sliding surface and tension crack strike parallel to the slope surface. Release surfaces are present so that there is no resistance on lateral boundaries.	Water pressures in tension crack and on sliding plane can be included. Simple analysis method.	Moments not considered in anlaysis. Can give overestimate of factor of safety on steep slopes where toppling could occur.	Hoek and Bray (1981) pp.150-198	Useful where plane failure can be assumed such as on sheet joints.
Wedge Failure	Two joint planes form three-dimensional wedge	Line of intersection of joints dips less steeply than rock face and daylights within the face. Both joint planes remain in contact during sliding.	Tension crack and water pressures can be included in analysis. Charts, which consider friction only, are available.	Moments not considered.	Hoek and Bray (1981) pp.199-225 and pp.337-351	Useful. Charts can be used for a preliminary assessment.
Toppling Failure	Stepped cross joints	Analysis assumes that some blocks will slide and some topple. Water pressures not included.		Limited to a few simple cases with suitable geometry.	Hoek and Bray (1981) pp.257-270	Not yet a rock slope design tool but may occasionally be useful.

Table 3: Suggested Values of Factor of Safety for Slopes

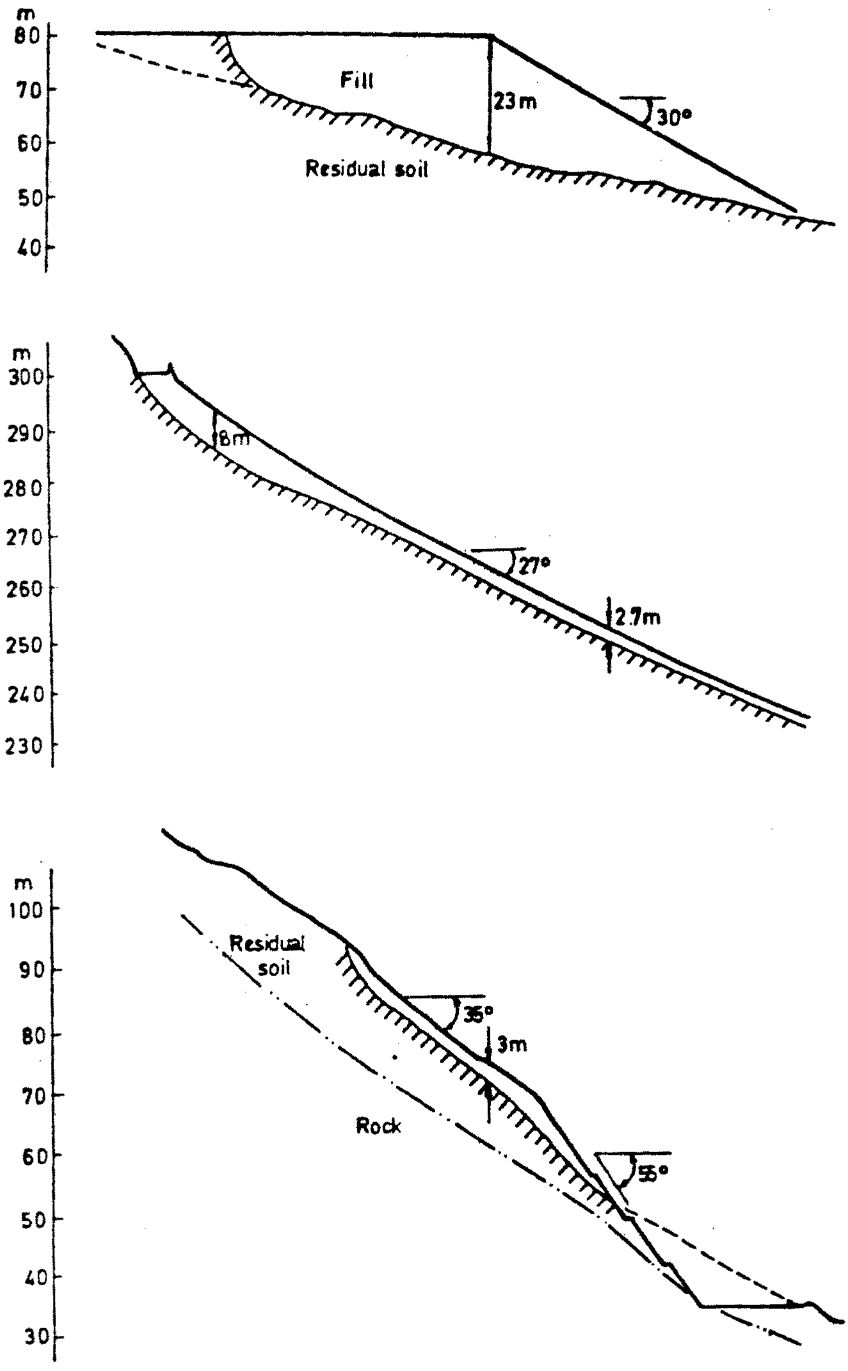
F.S.	Remarks	Source
1.2 1.4	Slope with low risk Slope with high risk	GCO (1979)
1.2 1.5	Temporary slope Important slope	Hoek and Bray (1981)

Table 4:
Rock Slope Stabilisation Measures

STABILISATION MEASURES

Failure Type	Excavation				Structural					support					Drainage					Rockfall control				
	Flatten slope	Bench	Local excavation		Gunite facing	Permeable (masonry) facing	Local structural "dentition"	Buttress	Anchored wall	Strap	Dowel	Bolt	Anchor		Drainage ditch	Screeded (paved) surface	Short drainholes	Long drainholes /Adits		Move structure/highway	Rock trap ditch	Rock trap fence/ wall	Netting	Scaling of loose blocks
Plane Failure	✓	✓						✓	✓			✓	✓		✓	✓	✓			✓	✓	✓		
Wedge Failure	✓							✓	✓		✓	✓	✓		✓	✓		✓		✓				
Toppling	✓											✓	✓		✓	✓		✓			✓			
Rock or debris fall & general degradation	✓	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓			✓	✓	✓			✓	✓	✓	✓	✓

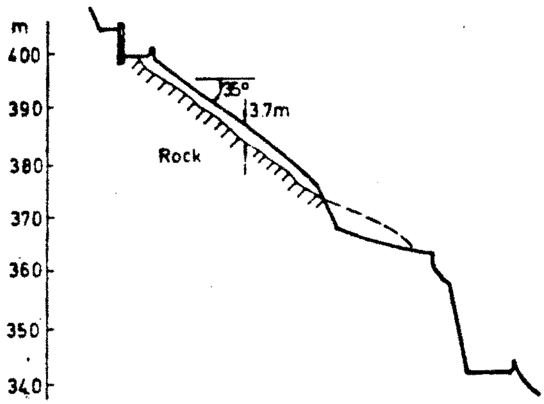
(Modified from Fookes and Sweeney, 1976)



(After Lumb, 1975)

Fig. 1(a): Examples of Sliding in Residual Soil Slope

(to be continued)



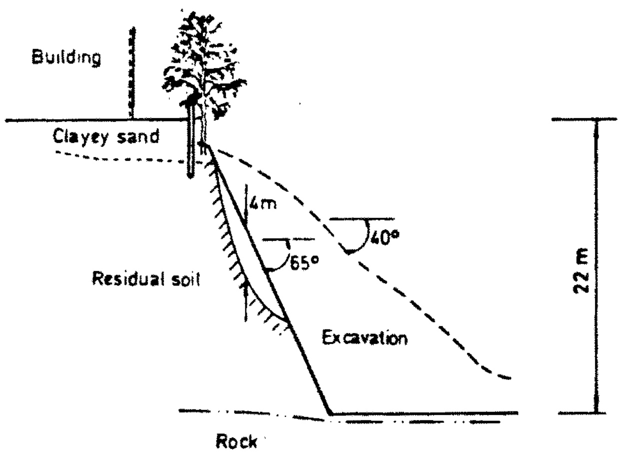


Fig. 1(b): Examples of Sliding in Residual Soil Slope
(/.....continued)

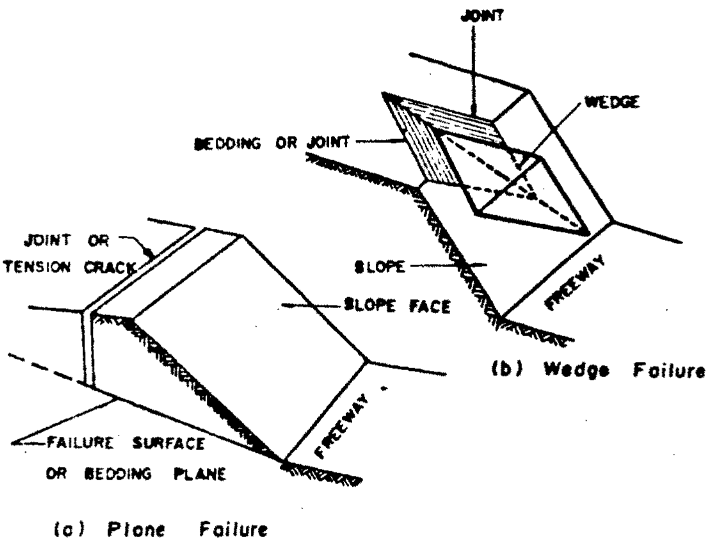
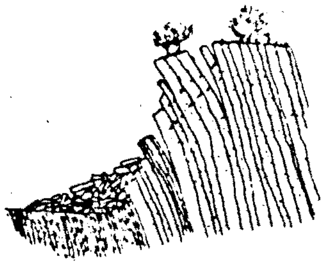
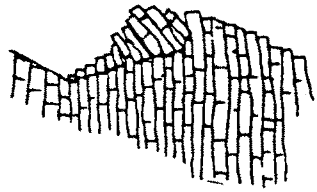


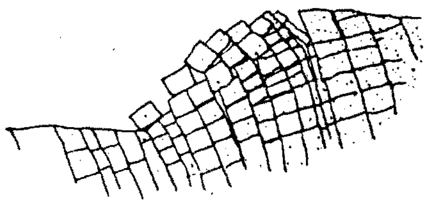
Fig. 2: Plane Failure and Wedge Failure of Rock Slope



a) Flexural Topping



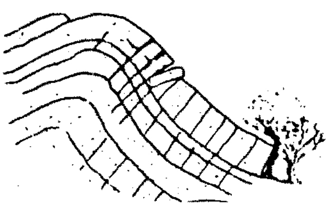
b) Block Topping



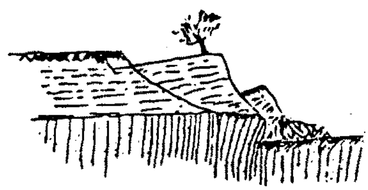
c) Block Flexural Topping

(after Goodman and Bray, 1976).

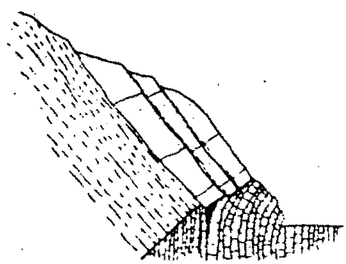
Fig. 3(a): Common Classes of Topples in Rock Slope



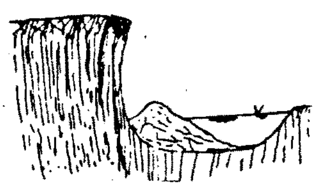
a) Slide Head Topping



b) Slide Base Topping



Slide Toe Topping



d) Tension Crack Topping

(after Goodman and Bray, 1976).

Fig. 3(b): Secondary Classes of Topples in Rock Slope

1 PLANNING AND PRELIMINARY DESIGN
- project requirement
- site selection
- preliminary site conditions

↓

2 INVESTIGATION
- exploration
- field and lab testing
- field measurements

↓

3 IDEALIZATION
- natural condition
- properties of material
- potential mechanism

↓

4 ANALYSIS
- numerical results
- sensitivity check
- factor of safety

↓

5 DESIGN
- design criteria
- design concept
- structure and details

↓

6 TENDER

↓

7 CONSTRUCTION
- field inspection
- additional investigation
- modification of design

↓

8 MAINTENANCE AND MONITORING
- instrumentations

Fig. 4 Design - Construction Sequence of Slopes

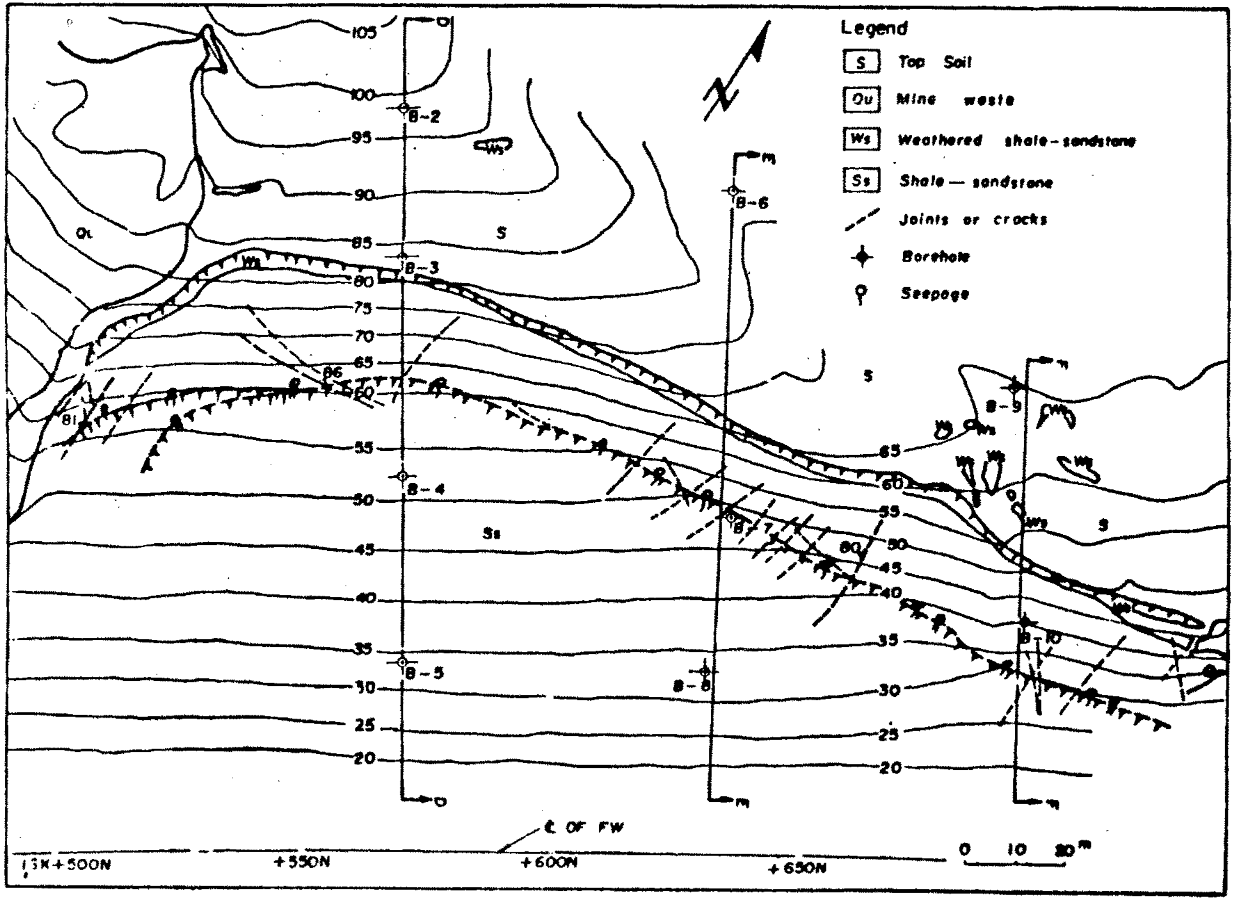
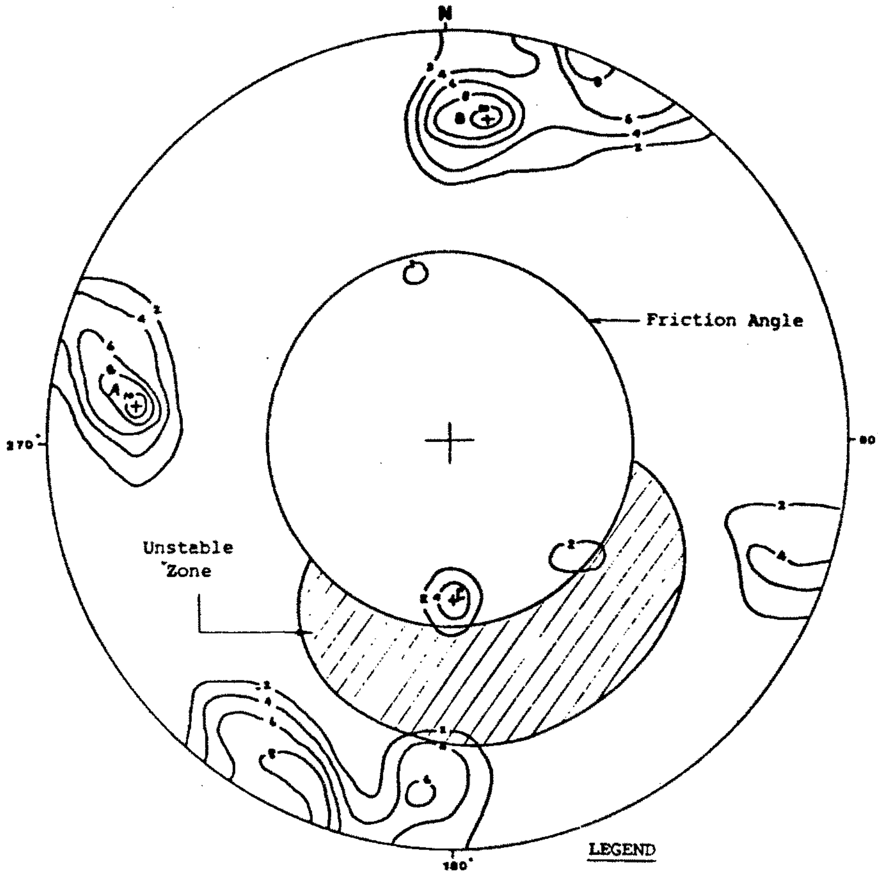


Fig. 5 Example of Engineering Geological Map

Project IB303-MS26J(D)



LEGEND

—4— Percentage of pole concentration

Station : CH 128+850 to CH 129+300

No. of Joint Survey : 115

Slope Condition:		Parameters:	
Dip Angle	: 65 °	γ : 23	kN/m³
Dip Direction	: 344 °	C : 0	kN/m²
Height	: ____ m	Ø : 38	°

Fig. 6 Steronet of Joints

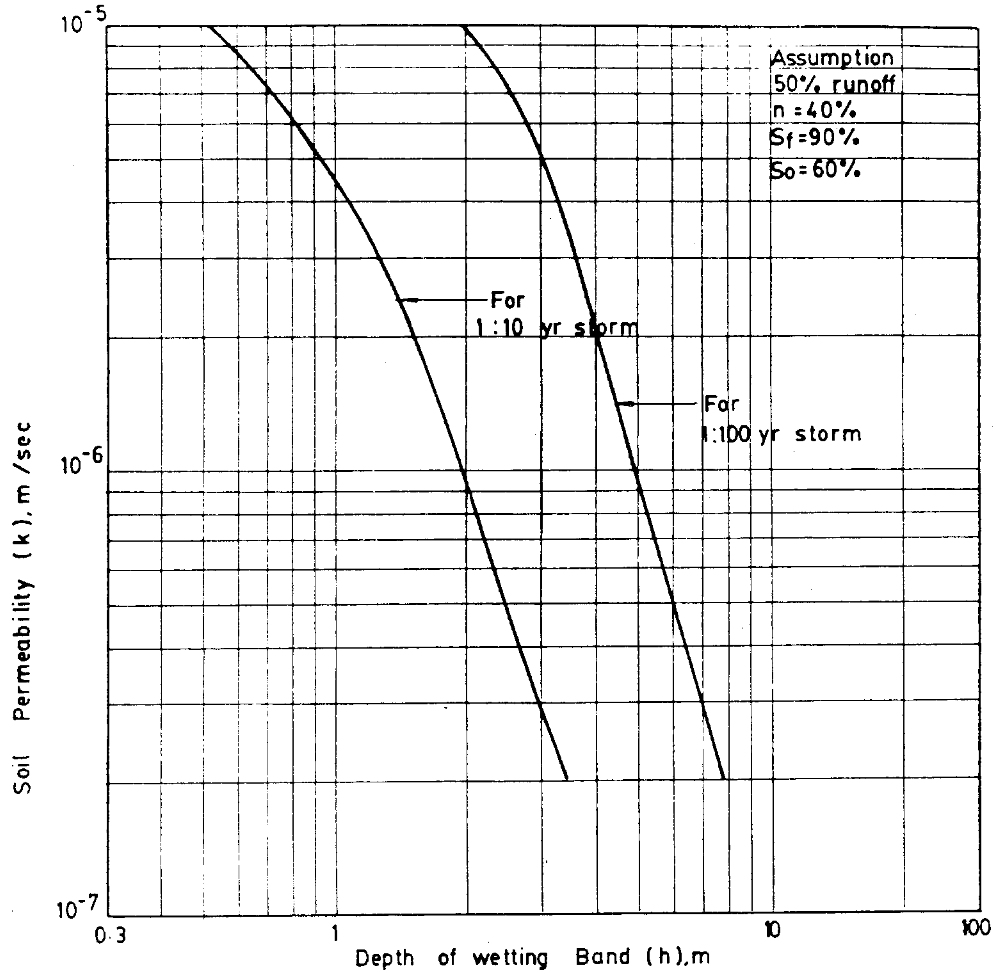


Fig. 7 Effect of Permeability on Wetting Band Thickness

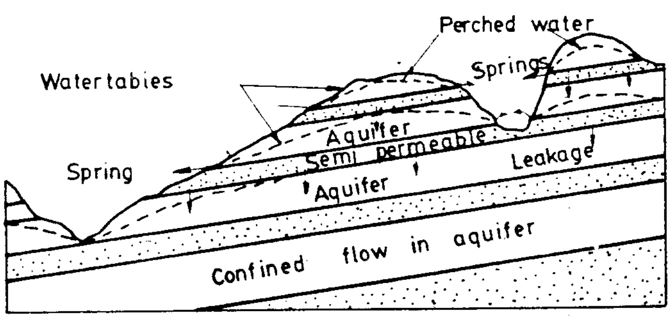


Fig. 8 Groundwater Condition in a Sandstone - Shale Formation

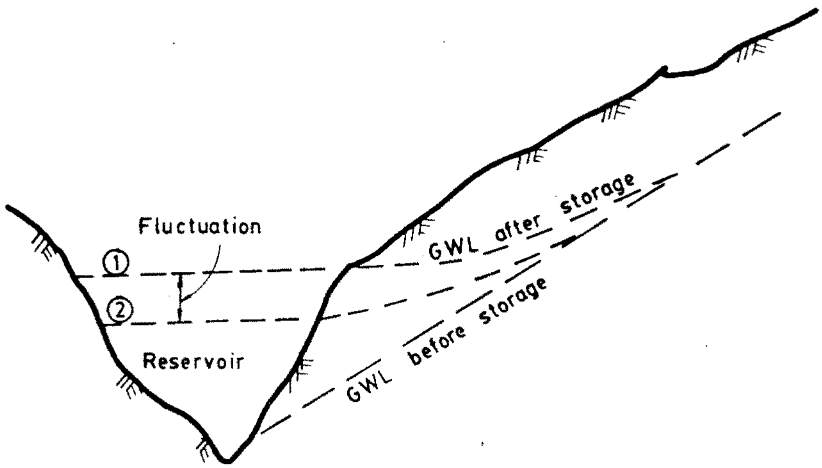


Fig. 9 Effect of a Reservoir on Groundwater Level

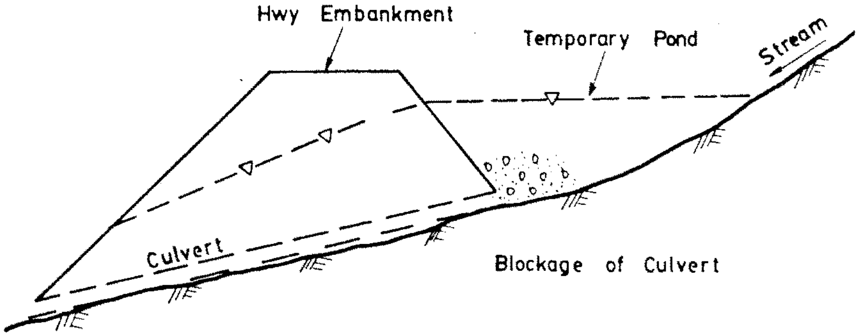


Fig. 10 Effect of a Temporary Pond on Groundwater Level in An Embankment

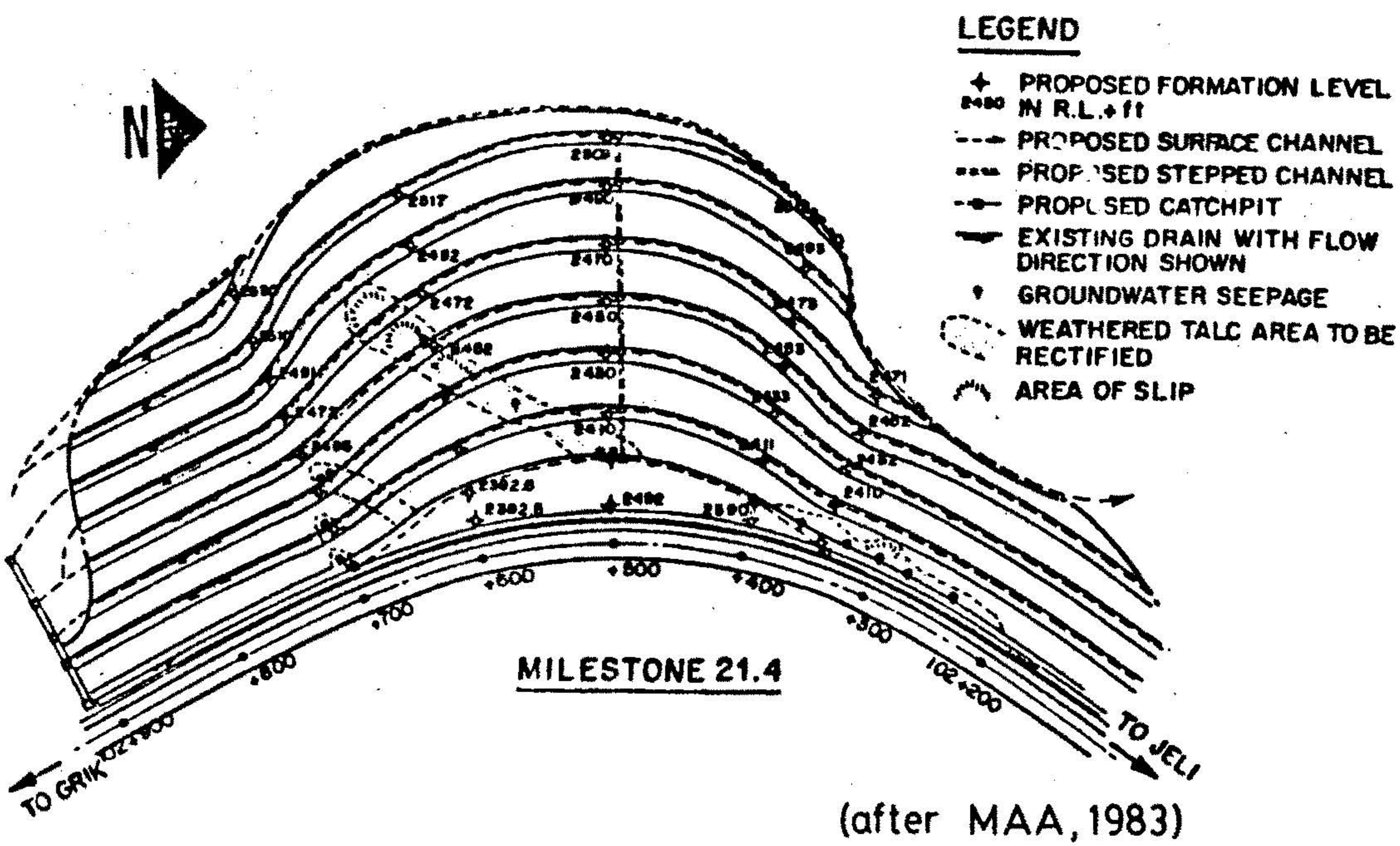
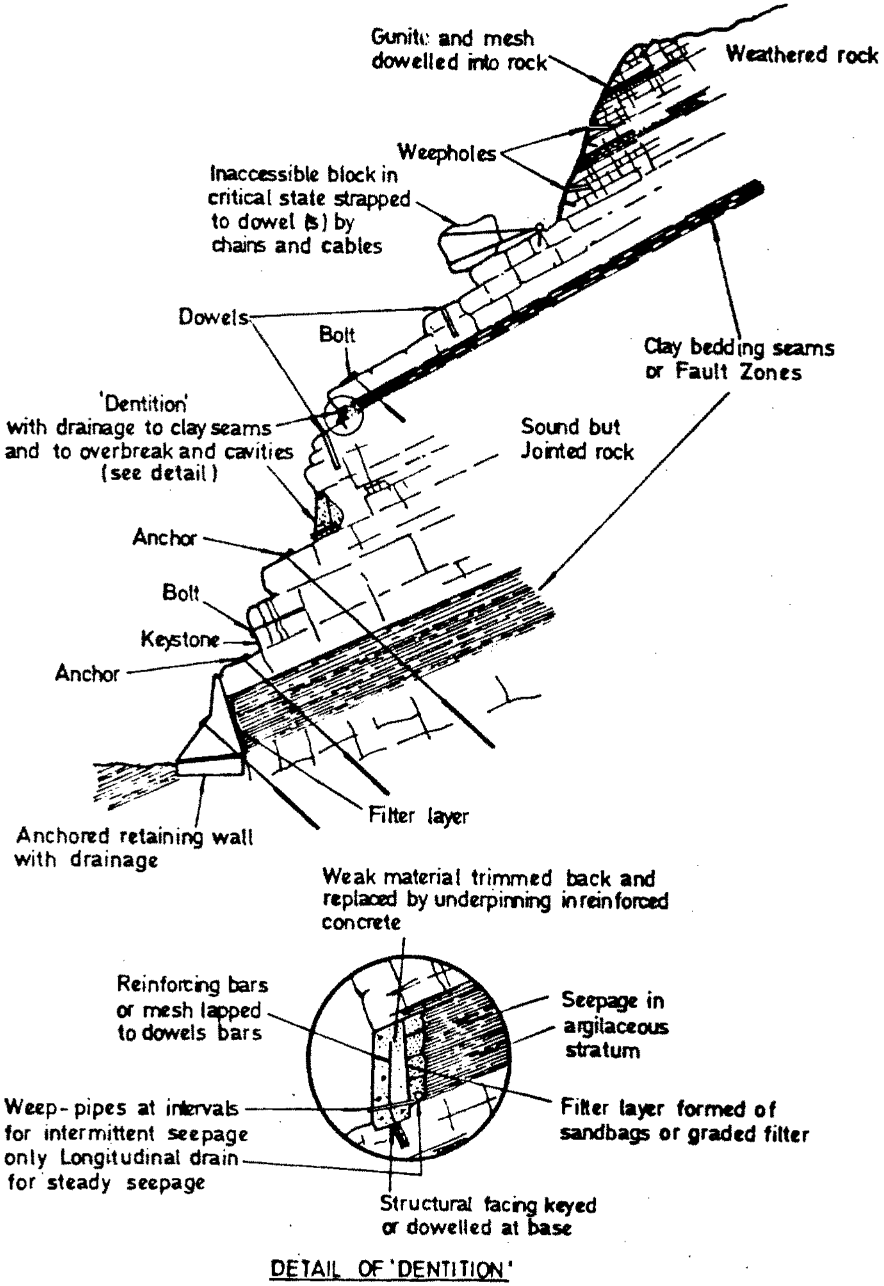
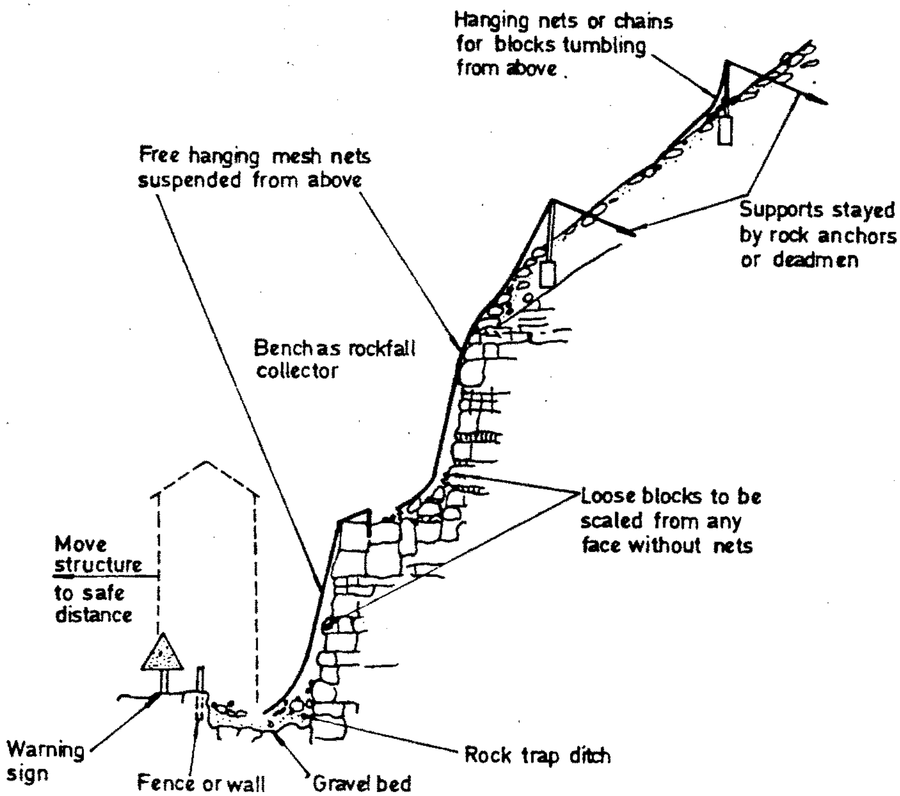


Fig 11 Example of Drainage System for Slope Design



(after Fookes and Sweeney 1976)

Fig. 12 Various Methods of Stabilising Rock Slopes



(after Fookes and Sweeney, 1976)

Fig. 13: Methods to Prevent Rockfall