TRANSPORT RESEARCH LABORATORY

ODA

OVERSEAS ROAD NOTE 16

Principles of low cost road engineering in mountainous regions

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PRINCIPLES OF LOW COST ROAD ENGINEERING IN MOUNTAINOUS REGIONS, WITH SPECIAL REFERENCE TO THE NEPAL HIMALAYA

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This Overseas Road Note was originally the concept of Mr J W F Dowling, former Deputy Head of the Overseas Centre, TRL. Nepal, its culture and its geotechnical challenge always held a special affinity for this geologist.

The cover photograph was taken by W G Heath, of TRL.

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OVERSEAS ROAD NOTES

Overseas Road Notes are prepared principally for road and transport authorities in countries receiving technical assistance from the British Government. A limited number of copies is available to other organisations and to individuals with an interest in roads overseas, and may be obtained from:

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FOREWORD

Great things are done when men and mountains meet;
This is riot done by jostling in the street.
Gnomic Verses, William Blake (1757-1827)

I believe that this Note is needed because there has been nothing so comprehensively available hitherto and the subject matter is important to those who build in high mountains. It deals with the mountains themselves, their behaviour, investigation and construction on their slopes, and represents a state-of-the-art survey in both conceptual and practical aspects. Although the case histories are drawn extensively from Nepal, the Note is applicable to mountains elsewhere, particularly those with a similar monsoonal climatic regime. Accordingly, it should be of considerable value to road planners, designers and constructers. The unifying theme is road works. However, a lot of the conceptual and practical detail will be of value to academics, engineering geologists and geotechnical, civil and hydraulic engineers, working in high mountain environments.

The famous through mountain routes of history usually occupy convenient, strategic interconnecting valleys and passes. Mountain roads of today are necessarily built in inhospitable terrain because of the need to link communities and access remote valleys. When I first went to Nepal, in 1962, it was by the gruelling weekly flight in an ageing DC3 from Patna (Northern India) to Kathmandu, along with the mail and goods occupying much of the rear of the aircraft. Obtaining a visa was difficult and required a period in residence at Patna to be near the Nepalese Ambassador. The DC flight was the easy option compared with the land route! My first view of the spectacular mountains was cloud!

In the 1960s and early 70s there were few roads capable of taking vehicles even into the foothills of the Himalayas. Movement of goods was, and still is in probably the majority of Nepalese countryside today, by backpacking porters. At that time route planning was exceptionally difficult and a poorly supported undertaking. Mapping coverage was minimal and air photographic coverage usually had problems because of the distortion produced by big changes in the severe relief. Remote sensing imagery was still half a lifetime away, and the hard won experience gained on the Indian designed and constructed Raj Path to Kathmandu was slow to trickle through. Living and working locally was not easy for foreigners, as it still is today away from the cities and bigger towns.

The first major British-funded road into the Nepal Himalayas was a low cost, sealed road, from Dharan to Dhankuta. Although the distance between these towns is only 17km, the length of the road is 52km. We had little published information to draw upon in creating the design, but every metre required careful thought and I consider that the road when completed was an engineering triumph. The overall concept was to climb in a heavily engineered corridor, usually stacks of hairpins, and make distance on the higher, flatter slopes. Experience on that road culminated in an innovative paper (Fookes and others, 1985 - see Bibliography) which distilled experience from that road, other locations in South-East Asia, the Andes, and the slowly growing world literature on the subject.

Mountain roads in the late 1980s and 90s, particularly in developing countries, have become more common and more venturesome. This Note builds upon and takes forward the experience and conceptual approach to mountain road design published in 1985 and, I believe, will be of immense practical value to the new breed of mountain roads.

The mountain environment stems directly from its geology and climate, current and past. The climate, particularly rainfall, dictates the way in which mountain slope erosion systems act and develop. Therefore, high mountains in different climatic zones tend to look and behave somewhat differently from each other and require variations in engineering approaches to investigation, design and construction. The bulk of the experience that has gone into the snaking of this Note has come from the Nepal Himalayas where the climate is warm and monsoonal in the Lower and Middle Mountain Zones. Extending the wealth of information contained in the following to mountains elsewhere in the world is particularly relevant for geologically young mountains with a similar climatic regime. Nevertheless, the majority of the fund of practical design and construction advice contained in the Note will be of direct help or give clearer guidance on ways of tackling severe relief, climate and materials in mountainous regions in different states of geological development and/or in different climatic regimes.

The geological construction of mountain systems is inextricably linked to plate tectonics. Nepal typifies the geological zones and variety of rocks that make up an orogenic mountain chain. Orogenesis (from the Greek meaning `mountain creation') results when one of the earth's crustal plates rides over another, causing thickening and crumpling of the overriding plate. The metamorphic rocks that result form the rocks of the Himalayas: low-grade metamorphic rocks in the Lower Zones and igneous and high grade metamorphic rocks in the Middle Zones. The Siwalik Hills, little-folded, weak, weatherable young rocks, and extensive alluvial plains of the Terai are the results of rapid erosion of the rising mountain chain.

The product of the monsoonal climate and the rapid, albeit jerky, uplift of the Himalayas with its massive rates of erosion dictates their special conditions and those of other high mountains of a similar nature. The Himalayas are among the youngest (about 10 million years) major mountain systems in the world. Slopes are very steep and largely covered in granular rather than clay-rich tropical residual soil more common on less steep slopes. These granular or taluvial slope soils are continually on the move downwards. The extensive slope instability common to young orogenic mountains, the frequency of gullies leading to bigger drainage systems to get rid of monsoon rain, and the overall deep incision of valleys keeping pace with the uplift, produce characteristic investigation and construction difficulties which, when flood and earthquake events are added, bring about most severe engineering conditions. In these conditions investigatory techniques are principally based on geotechnical mapping supplemented by conventional pits and observational methods, all of which have to be further supplemented during construction. Access and major difficulties of conventional site investigation drilling are dominant, with continuing problems in a physical environment which, at best, is commonly only marginally stable.

Typically, the higher the design standard, the greater the problems of construction. My experience is that high road design standards will lead to extremely high capital construction costs, for example in massive retaining works, tunnels and big bridges, and also to much higher repair and maintenance costs. I believe time is beginning to show that the most successful low cost roads are those where maintenance is frequent and well done and the road has been designed and constructed in sympathy with the physical environment. The latter does not necessarily imply high design standards but appropriate engineering for the natural circumstances.

The Transport Research Laboratory has been actively involved in maintaining a professional interest in the engineering geological and geo-environmental aspects of road engineering in unstable mountainous areas. They have collaborated with engineering consultants and government agencies on a number of projects. Building on this and on experience by others, this Road Note is aimed at both private and public sector engineers and planners. The objective has been to produce a manual which can lead, in a straightforward manner, to planning, investigation, design and construction of road works that are in sympathy with the mountain environment to standards that are appropriate for that environment, and costs which make such roads affordable. I see this as having been achieved by the concise style, with many to-the-point practical tables and diagrams, all of which should be straightforward to follow providing the reader has some understanding of the geological and climatic setting which govern the processes operating in the landscape. One of the strengths of the Note is that the chapters are both self-contained and dovetail into each other.

Emphasis on aspects of design for mountain roads is different from that for conventional roads in lowlands: drainage is still the paramount design requirement but with extensive use of offsite drainage. Not only has a huge amount of water to be removed rapidly and efficiently, but the conveyer belt of debris being brought down the mountainside, particularly in times of high rainfall, has to be dealt with. The concepts developed for mountain gully and valley works are not usually required for other landforms. This is also true of hairpin stacks and flood protection arrangements. Earthworks, to a large extent, are not essentially different from those of flatter terrain, only perhaps there are more of them, and retaining structures, slope protection and stabilisation measures are ubiquitous. However, they must be well drained, flexible, extensive and put in quickly in short lengths to prevent a disaster during construction.

The secret of successful high mountain road design and construction is to work with and not against the massive forces of nature. Frustrating most of the time, but I love it.

Prof P G Fookes, F.Eng.

Winchester

September 1996

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PREFACE

In the world's poorer countries. there is an increasing requirement to build roads into the less accessible parts of the country as a means of facilitating development projects in these regions. Where these inaccessible areas constitute a mountainous interior, the problems and costs of road construction multiply. The increased costs of road construction are not simply related to the high capital cost of building an alignment in complex, possibly unstable terrain. Of much greater concerti to international funding agencies is the long term cost of road maintenance in areas where instability is a natural process and inevitably can be expected to affect the road at some time in the future.

The Overseas Development Administration's objectives in terms of road design for development in mountainous regions are threefold. The first is to design an alignment that is affordable within the current development ethic. The second is to adopt a design that is appropriate to the terrain, with a good expectation that it will stand up to the forms and intensity of instability experienced in the region. The third is to design a road that can be maintained to a good standard by the host nation, within the limits of its financial resources and technical capabilities. This last may initially need to be accomplished with assistance, but under ODA's policy of helping nations to help themselves, responsibility for maintenance would, in time, be expected to return to the host nation.

ODA has strived to construct roads that fulfil these requirements, but thirty years ago, when the Administration first began to plan its road construction programme in Nepal, relatively little was known about the pace and vigour of instability in the Himalayan region. Initially, ODA consulted TRL in matters of alignment and drainage design, and through this work was realised the importance of designing `with the terrain' in such an active environment. For management of the construction of later roads in Nepal, ODA has appointed consultancies with leading expertise in geomorphological and geotechnical investigations. As a result, ODA's roads in Nepal have a record of stability that equals the best.

For a variety of reasons, organisations and individuals who have become involved with road design and construction in Nepal have remained on the scene, on and off, for many years. These organisations and individuals have been in the fortunate position (for posterity) of being involved throughout the phases of route planning and survey, road construction and beyond. As a result, they have been able to see the effects of design decisions taken perhaps fifteen or twenty years previously. They have also been able to monitor the effectiveness of a wide range of maintenance practices over periods of years.

The organisations who are behind Overseas Road Note 16 are among this group. By virtue of its association with ODA, TRL has had unrivalled opportunity to follow progress in road construction in Nepal. Through ODA's funding to TRL, TRL has been involved in providing advice to ODA on road engineering in Nepal and other mountainous regions, first in the late 1960's and periodically ever since. Through visits to Nepal over this long period, TRL staff have kept up observations more or less annually on ODA-funded roads and, incidentally, on other major roads in Nepal. TRL has been present on roads under construction and in the aftermath of occasional natural disasters. Latterly, TRL has carried out research into bio-engineering methods of slope protection. Also, in collaboration with Scott Wilson Kirkpatrick it has used ODA research funds to measure rates of natural instability on slopes in east Nepal.

In unstable mountainous environments there is no substitute for practical experience. The importance of rigorous application of standard design practice is not in question. But in an environment where unpredictable natural forces are always at work, reliance on standard design solutions should be tempered by a preparedness to use the eye and intuition to vary the 'book' design in places where the terrain demands it. The terrain and the forces of nature are paramount in a mountainous region. They dictate design, and any design that does not take account of the ever-changing environment is doomed to failure.

The experience gained by the authors and their colleagues from observation of successes and failures on roads in Nepal and elsewhere has been brought together in this Overseas Road Note. The principles of good engineering practice described in the guide are derived from extensive experience and a deep affinity for landscape held by its contributors. Through its publication, ODA's three objectives of designing roads for development, mentioned above, can perhaps become more widely attainable.

C J Lawrance

Overseas Centre

PRINCIPLES OF LOW COST ROAD ENGINEERING IN MOUNTAINOUS REGIONS, WITH SPECIAL REFERENCE TO THE NEPAL HIMALAYA

1 INTRODUCTION

1.1 Geologically young fold mountains, especially those experiencing humid climates with seasonally intense rainfall, are among the most steep and unstable landscapes in the world. They are consequently the most difficult in which to construct and maintain roads. Common problems encountered include steep and irregular topography, and difficult excavation and founding conditions due to the deeply weathered and transported nature of many of the soils. Intense and prolonged rainfall lead to locally high groundwater tables. saturated soils and large quantities of surface runoff during the wet season. These problems are usually compounded by landsliding, erosion, river flooding. river incision and periodic seismicity.

1.2 Unfortunately, these conditions occur in many developing or newly industrialising countries that contain a relatively poor and remote mountainous interior. In these situations, engineering approaches frequently have to be applied on a low cost and low-technology basis. Often, relatively high population densities co-exist with an extremely fragile terrain. Cultivatable land is at a premium and these communities usually practice intensive terrace and irrigation farming systems that create additional problems for drainage management along road alignments.

1.3 The Himalayas constitute one of the most tectonically active fold mountain belts and experience some of the highest rates of erosion in the world. The difficulties presented by conditions of adverse geology, topography and land use are made more acute iii the humid sub-tropical and humid warm temperature zones, where rapid rock weathering and heavy rainfall act to induce landsliding and erosion.

1.4 Nepal typifies these geographical conditions. Nepal is a country where the road network has expanded rapidly since the 1950s, in connection with the large number of development projects that have been implemented there since that time. In support of ODA's own development programme in Nepal, the authors' organisations have gained considerable practical experience of road construction methodology and techniques in this fragile environment. The experience has provided the technical expertise upon which much of this guide is based. For this reason, the report is illustrated with examples mostly from Nepal. However, observation of the condition of slopes along most of Nepal's roads over many years has led to the identification of ground rules for good engineering practice in steep and unstable terrains generally.

1.5 Therefore, although the Nepal Himalaya forms the main subject area of this guide, the discussions and recommendations should be equally applicable to the adjacent Hindu Kush-Karakorams, parts of the Andes and, perhaps with some modification to account for differing engineering technologies and land use patterns, most of the fold mountain chains of South East Asia. The guide will also be relevant to many aspects of road engineering in volcanic terrains, although volcanic hazards themselves are not dealt with. High altitude glacial mountains are excluded from this guide, as are problems of snow and ice on mountain passes.

1.6 Road design, construction and maintenance under Himalayan conditions requires a different approach to that conventionally adopted for less severe flat or rolling terrains. While the engineering principles remain the same, design parameters and design priorities are quite different. For instance, the shortest road alignment is not necessarily the easiest, quickest or cheapest option to construct or maintain. Frequently, topography, slope stability, flood hazard and erosion potential are likely to be the most significant controls on the choice of the most suitable alignment and design of cross-section. Variations in geology and slope greatly influence road design and hence the cost of construction, and these variations can occur over very short lengths of alignment. Geology, geomorphology and hydrology, therefore, are key factors in the design, construction and maintenance of roads in these regions. Nevertheless, an appreciation of these factors alone is not enough if roads are to be constructed in an environmentally sympathetic way, without premature damage. Road geometry, earthworks, retaining structures and drainage measures must be designed in such a manner as to cause the least impact on the stability of the surrounding slopes and natural drainage systems. Excessive blasting, cutting, side tipping of spoil and concentrated or uncontrolled road drainage often lead to accelerated instability and erosion. Although many of these effects are often unavoidable to a certain extent, the design and the construction method adopted should aim to minimise them.

1.7 Bearing in mind the destructive nature of slope and drainage processes that fashion the landscape in these areas, financial and technical commitment to road maintenance is as important as environmentally-compatible engineering design. The practicability and cost of road maintenance in many developing countries must be borne in mind when deciding upon road standards and allocating project funds. Again, geology, geomorphology and hydrology will exert significant controls on what can be designed, constructed and maintained within a low cost framework.

1.8 The design of roads under Himalayan conditions is a relatively little-explored and documented subject. Road construction has been carried out by many agencies, often under development aid, but many using local resources. Some of these roads have remained in place due to costly investments in construction and maintenance, while others have succeeded because they are models of sympathetic engineering design in terms of alignment, road width and cross-section to suit the terrain encountered. At the other end of the spectrum, some roads have suffered such instability that they have ruined their environment and have become impassable for long periods. Alignments are sometimes abandoned due to inadequate planning, or trigger such instability and erosion that they cannot be maintained. Road drainage systems are then allowed to fall into disrepair, thus exacerbating the problems. One of the principal lessons to be learnt from these road failures is the fact that consideration of the design of a road cannot be limited to the right of way alone, but must take adequate consideration of ground conditions throughout the landscape in which it is to be constructed.

1.9 It is not enough to simply apply practices evolved in the mountains of Europe, North America and the steep terrain of Hong Kong. In these industrialised regions, the difficulties are surmounted by intensive geotechnical analysis and the most modern and capital intensive construction materials and techniques. These approaches are generally inappropriate under fold mountain conditions in other regions where neither the money nor the expertise are available to build and maintain roads in this manner. In addition, the frequency with which destructive geomorphological processes occur and cause road damage, no matter what level of engineering investment is employed, demands a different philosophy to design.

1.10 Although users of this guide should be aware of the full range of potential problems and design considerations described, some aspects will be more pertinent than others, depending upon the particular circumstances at hand. For instance, resources required to be spent on geotechnical hazard assessment and mitigation will vary from one project to the next. Furthermore, while the guide has been prepared for practising road engineers, both in the private and public sector, some of it is aimed specifically at specialist support staff, such as geotechnical engineers and hydrologists. Consequently, the layout of the guide has been designed to allow selective reference.

1.11 This is the first edition of the new guide, compiled from the extensive experience of many workers but not yet benefitting from the opinions of the readership. TRL would welcome comments from readers on the usefulness of the guide and any suggestions they may have for improvement.

2 GEOGRAPHICAL SETTING OF THE NEPAL HIMALAYAS

GEOLOGY AND TOPOGRAPHY

2.1 The Himalayan ranges (Figure 2.1) extend in a 2400 kilometre-long arc between the Indus River in Pakistan to the west and the Brahmaputra River in Assam to the east, and are between 250 and 400 kilometres in width, being bordered by the Ganges Plain to the south and the Tibetan Plateau to the north. The Himalayas evolved through collision of the Indian and Eurasian continental plates, compressing a deep basin of sediments that lay between them and thrusting these sediments up into a series of great folds or nappes (recumbent folds). Although the main period of land rise is over, uplift continues at a mean rate of approximately 1 nun per year, but local rates vary widely, with rates of about 2mm per year in eastern Nepal and up to 5mm per year reported for the foothill ranges of Pakistan, for example.

2.2 The lithology and structural geology of the Nepal Himalaya (Figure 2.2) reflect this compression and thrusting through the formation of continuous northwest-south-east lineations. These lineations have controlled the distribution of uplift and elevation, differential weathering, drainage patterns and river downcutting to the extent that the major elements of the topography comprise a series of roughly northwest-southeasterly trending ridges and valleys that extend the entire length of the mountain chain. The ridges have been dissected by the major southward flowing antecedent rivers which have cut through the emerging northwest-southeast relief lineations as the land mass has risen. Earthquakes and ground uplift continue to occur predominantly in association with seismic events at shallow depth along active thrust faults, and particularly the Main Boundary Thrust (Figure 2.2).

2.3 The geological and relief lineations described above have combined to create five main physiographical zones (Figure 2.3), separated by the principal thrust faults, within which climate, geology, slope morphology, soils, drainage and land use patterns are broadly similar. These zones are described in paragraphs 2.11 to 2.18, and comprise, from south to north: the Terai (Gangetic Plain); the Siwalik Hills; the Mahabharat Lekh (these two mountain ranges collectively constitute the Low Himalaya); the Middle Himalaya or Middle Hills; and the High Himalaya. The glacial snow and ice landscapes of the High Himalaya are excluded from thus guide as road construction is rare in these areas.

CLIMATE

2.4 The climate of the region varies from humid sub-tropical over the Low Himalayan ranges and on the valley floors and adjacent slopes of the main Middle Himalayan rivers, through humid warm temperate in the remainder of the Middle Himalaya to alpine over the High Himalaya. The rainfall regime of the Low and Middle Himalayas is controlled by the southeast Asian monsoon with as much as 80% of the 1,000-4,000mm (and occasionally in excess of 5,000mm) annual precipitation falling during the summer months of June through to September. However, even during the summer, lengthy dry periods are common, and are often followed by rainstorms during which daily totals frequently exceed 100mm and occasionally 300mm. In the east of Nepal and neighbouring Indian Himalaya, catastrophic downpours amounting to 500-1,000mm in 3-4 days are reported to occur on average every 20-25 years and are accompanied by widespread flooding and slope failure. These problems are recurrent throughout the Low and Middle Himalayas of Nepal and India.

2.5 Annual rainfall tends to decrease from east to west, owing to the orientation of the Himalayan ranges with respect to the southeasterly direction of the monsoon. The Low Himalaya (especially the Mahabharat Lekh) is the first major topographic barrier to this air flow and tends to receive the most intense rainfall (10-20 minute duration intensities of 100-150mm/hour are not uncommon). Annual totals vary between 1,500 and 2,000mm. The rain shadow cast by the Mahabharat Lekh is often responsible for a much drier climate in the Middle Himalaya immediately to the north, with annual rainfall frequently less than 1,000mm. Nevertheless, the rain shadow effect tends to be localised, so rainfall amounts and intensities can still vary considerably within distances of only a few kilometres. Further north again, on the higher ridges of the Middle Himalaya, rainfall tends to increase to between 2,000 and 4,000mm per annum, although intensities are considered not to be as high as they are over the foothill ranges.

GEOMORPHOLOGY

2.6 Cycles of landscape uplift and drainage incision, followed by periods of relative stability have given rise to sequences of steep slopes, that were once adjacent to incising and eroding drainage lines, separated by valley side benches that represent earlier floodplain terrace levels and erosion surfaces. However, these sequences are frequently disrupted and often replaced by landforms dominated by geological structure, by major valley side instability and by deep tributary catchments that have undergone extensive erosion and slope failure. Figure 2.4 shows a mountain model based on these geomorphological concepts of landscape development. The model is described in Box 2.1. Generally, lower valley sides are by far the most unstable. For example, between 80% and 95% of landslides mapped from aerial photographs in parts of the Mahabharat Lekh occur on the lower valley sides where river erosion regularly gives rise to slope undercutting, and where groundwater tables approach the ground surface most frequently. Although steeper slopes at higher elevations in the landscape are generally more stable, they are often in an equilibrium state and therefore remain susceptible to disturbance by external processes.

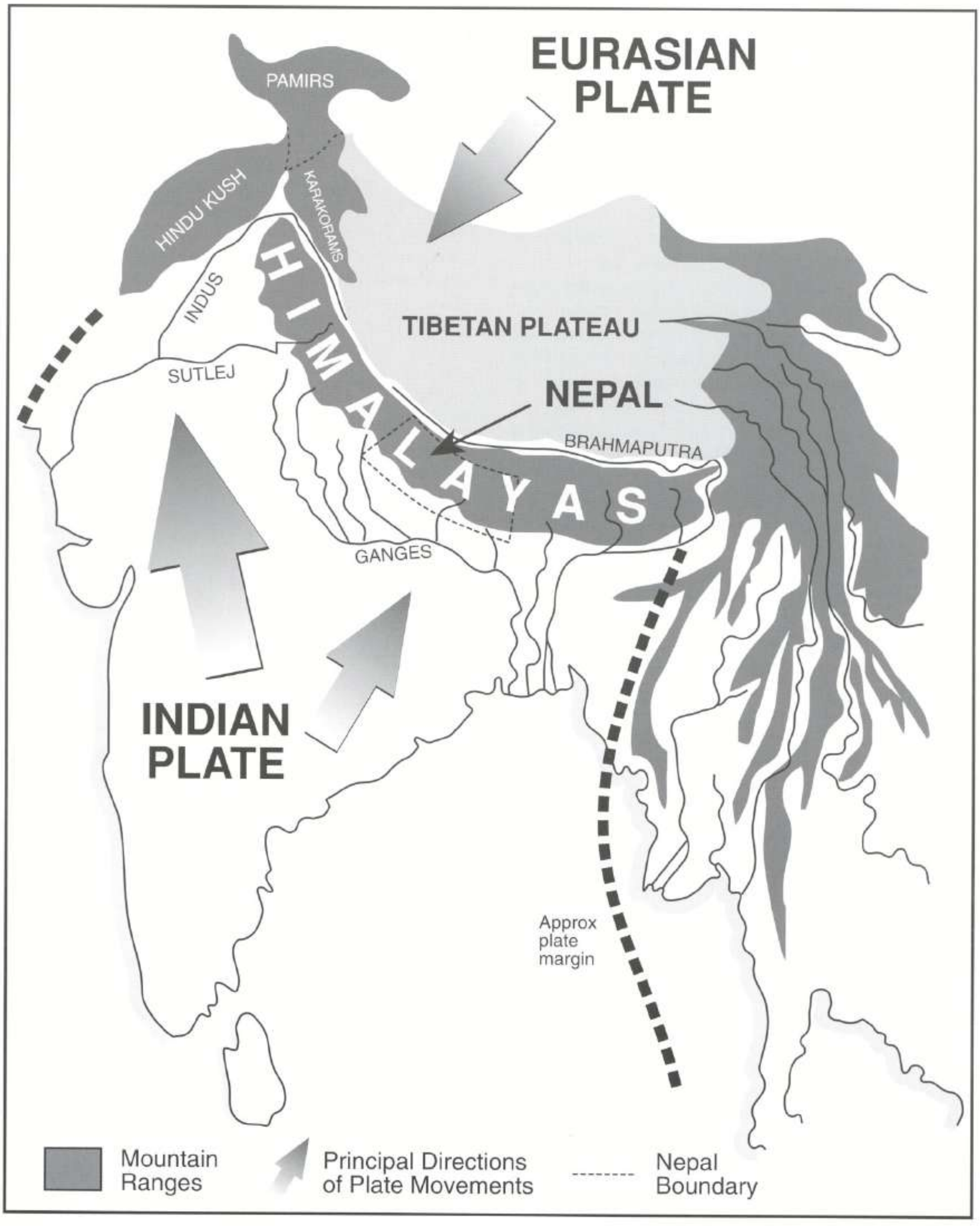


Figure 2.1 The Himalayas and their regional setting

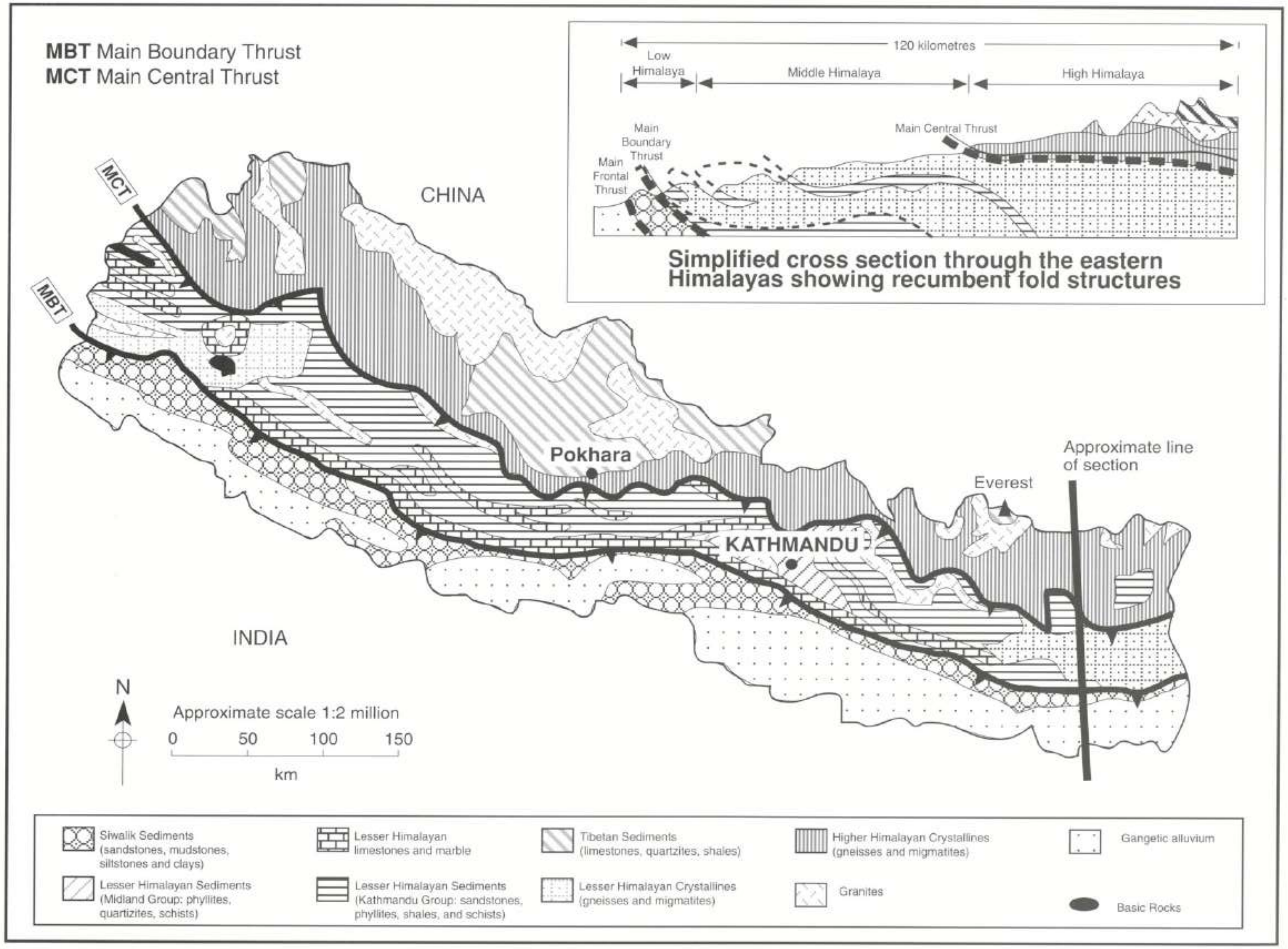


Figure 2.2 Summary geology of Nepal (based on data published by Department of Mines and Geology, Kathmandu)

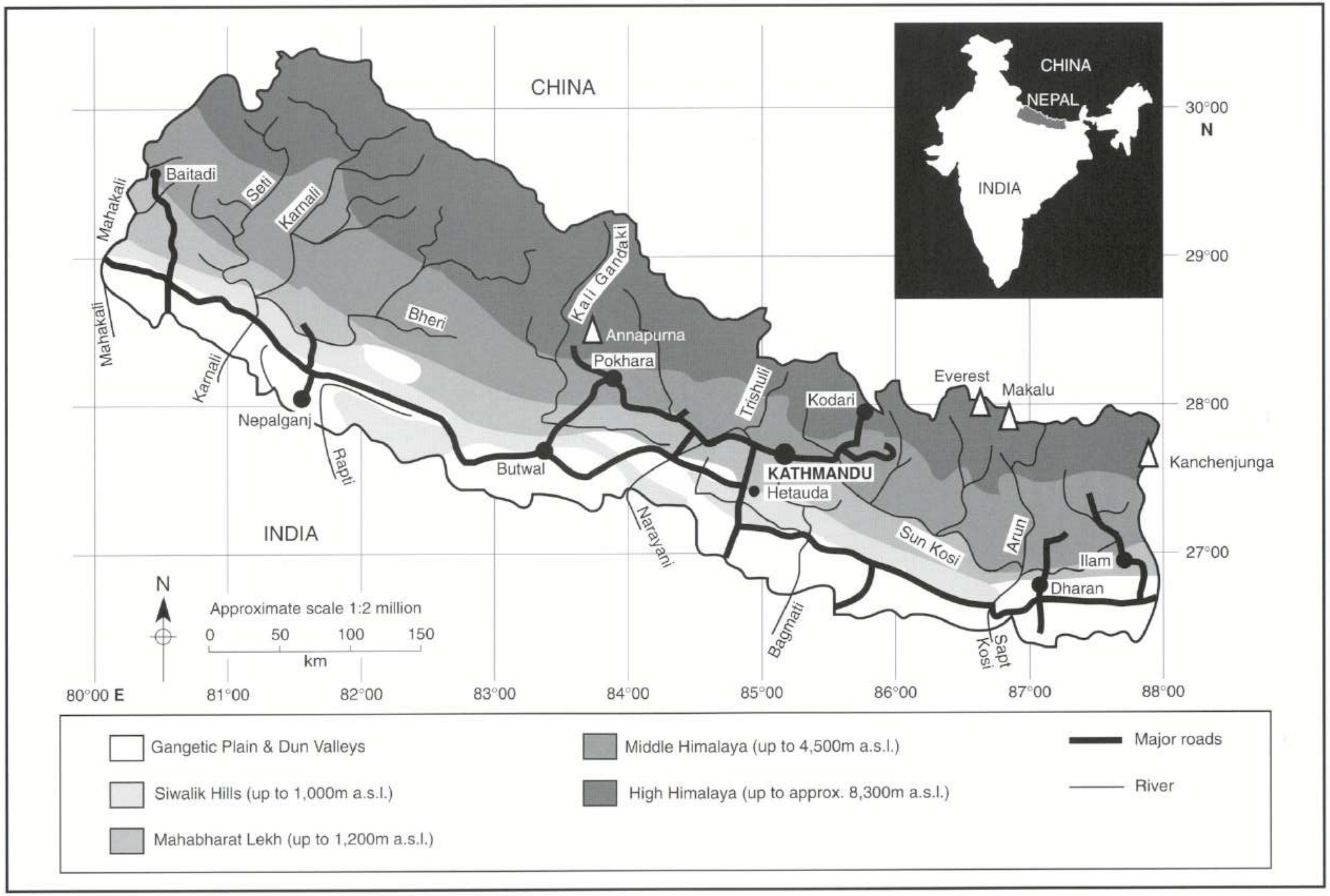


Figure 2.3 Geography of Nepal showing the principal Himalayan zones

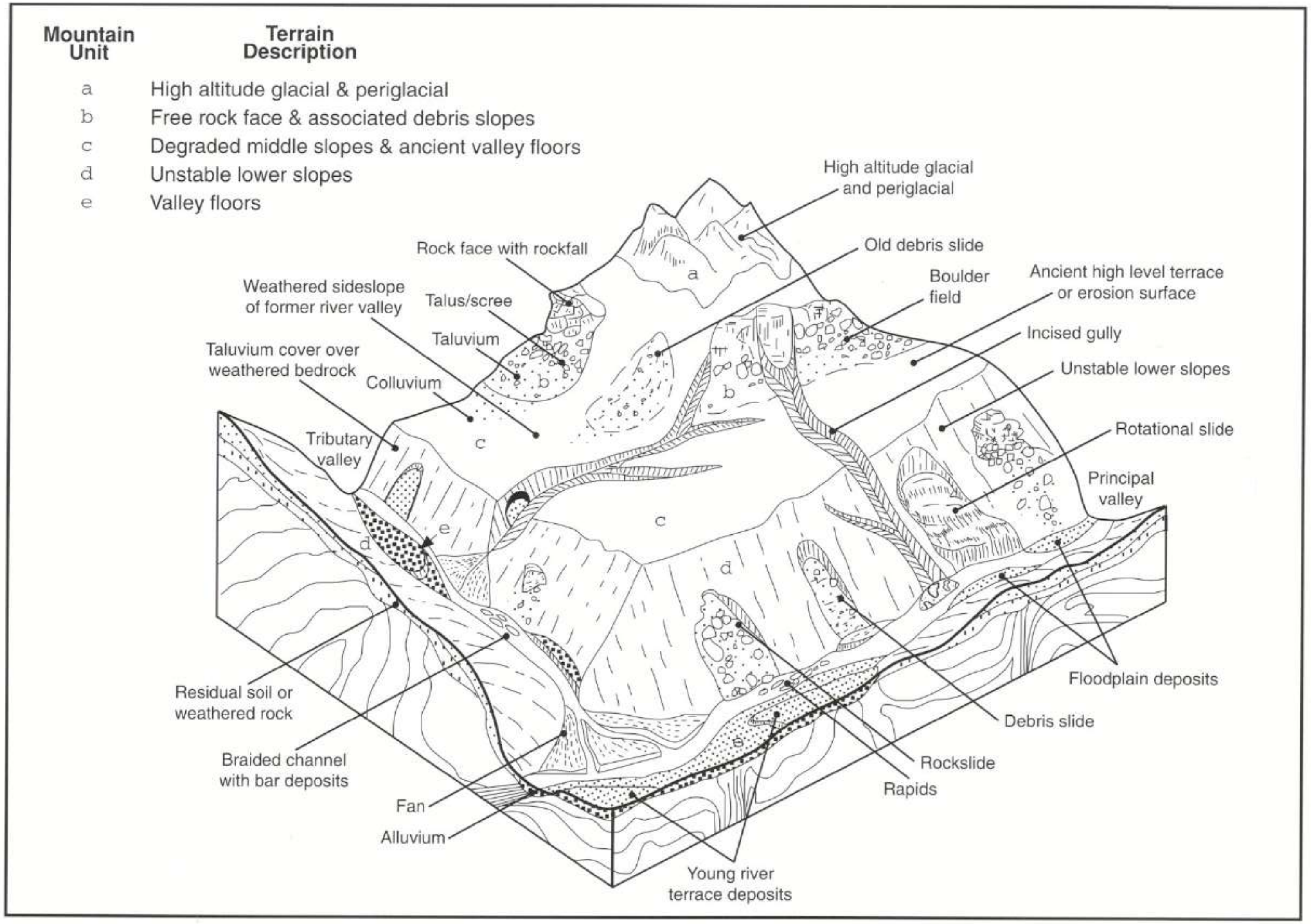


Figure 2.4 A Model for young fold mountains (after Fookes et al, 1985)

2.7 The production of large volumes of debris from eroding gullies and slopes, and its transportation through the drainage system, can occur in response to rainstorms and floods of only moderate magnitude. Within a given area, these floods may occur every year in small catchments (1-5km^2) and every 5-10 years in medium-sized catchments (10-20km^2). More widespread flooding and erosion in large and multiple catchments may recur every 20 years or so. Events that result in extensive slope failure, erosion, flooding and significant drainage and geomorphological readjustments, may recur over a period of 50-100 years.

2.8 Rainfall intensities in excess of 50-100mm per hour sustained over a 30 minute period are reported to be the threshold above which slope failures and erosion take place, although this threshold will vary widely according to slope and drainage conditions. Even in such an active landscape as the Himalayas the intensity of these processes varies markedly according to geographical and physiographical zones described in paragraph 2.3. In the Low Himalaya, for instance, denudation rates of 2-5mm per year are considered not uncommon, and in small catchments may reach double figures, whereas in most of the Middle Himalaya rates of ground lowering by erosion etc may be considerably less than l nun per year.

2.9 Quite clearly, an early appreciation of these various physiographical and geomorphological controls on slope and drainage processes and rates of erosion is crucial to the sensible location of road alignments and road design in general.

2.10 The proportion of land covered by forest in Nepal varies according to slope steepness, accessibility (for felling), population density and, more recently, conservation measures. Slopes in excess of 35° are generally too steep for cultivation and are usually forested although, where close to main trails and populated areas, they have usually suffered depletion for fuel, fodder and building construction. The least forest cover is found in the less steep and intensely cultivated parts of the Middle Himalaya. Further north, in the higher altitudes of the Middle Himalaya, it is in excess of 60% due to the lower population densities, steeper slopes and poor soil conditions that are generally unsuitable for cultivation.

HIMALAYAN ZONES OF NEPAL

Siwalik Hills

2.11 On the southern margin of the Himalayas, the Siwalik Hills rise abruptly to between 500m and 1,000m above the Gangetic Plain along the Himalayan Frontal Thrust. Dun valleys occur at intervals within the Siwalik chain and form enclosed alluvial basins surrounded by hills. The Siwalik rocks themselves are formed from debris derived from the erosion of the rising Himalayas to the north, and comprise soft sandstones, mudstones and siltstones.

Box 2.1 The five-unit mountain model

Geologically young fold mountain areas typically contain the following features: a prevalence of older, fairly stable land form features on the upper valley sides, steep lower valley sides affected by instability, and sediment-filled valley floors. The instability is kept active by periodic episodes of vigorous downcutting by the rivers. The concept of fold mountain landscapes containing this range of land forms in ordered assemblage has led to the development of a terrain model for these regions (Fookes and others, 1985). The model is illustrated in Figure 2.4 and its units are described below. The Himalayas follow this pattern. and its regions are given below as examples.

Unit a. Glacial and periglacial topography (typical of the High Himalaya). The rock and ice terrain of the high peaks.

Unit b. Free rock faces and associated debris slopes (typical of the High Himalaya and highest elevations of the Middle Himalaya). Steep rock ridges and cliffs, with screes and boulder fields at the base. The kinds of instability frequently observed include rockfalls, rock avalanches, rockslides and wedge and toppling failures.

Unit c. Degraded middle slopes and ancient valley floors (typical of parts of the Mahabharat Lekh and the lower elevations of the Middle Himalaya). This is a denudational topography, in which prolonged weathering and less active erosion have produced gentler slopes bearing thicker soils. Slope stability problems are much reduced, although rilling and gully erosion are rife. The unit is densely populated and cultivated. Note that the unit is at moderately high altitude, `sandwiched' between the rock ridges and steep slopes of unit 2 and the deep valleys of unit -1.

Unit d. Active lower slopes (typical of many parts of the Mahabharat Lekh and some of the more confined slopes adjacent to the major rivers in the Middle Himalaya). These are a continuous feature, occupying almost all lower valley slopes. They have been formed as a result of a period of renewed uplift (attended by renewed downcuttmg of all the streams). Long, steep, straight valley sides are covered by a thin mantle of colluvial soil (transported slowly by gravity) over weathered rock. Instability is normal in this terrain; shallow translational landslides and gully erosion are the commonest forms.

Unit e. Valley floors (typical of the Low Himalaya and, to a lesser extent, the Middle Himalaya). The valley floors are almost always occupied by large quantities of debris from the slopes above in the form of fans, terraces and the bed of the liver. The chief engineering problems present in this part of the landscape are high flood levels, debris-laden flows, shifting river courses and shifting accumulations of debris (see Chapter 13).

2.12 High levels of slope instability and erosion in the Siwalik Hills are attributable to steep slopes, intense rainfall, the softness of the rocks and the high rates of chemical weathering under the humid sub-tropical climate. The fine-grained rocks and weathered materials are subjected to cyclical wetting and drying which causes cracking and encourages further ingress of water into surface materials. Saturation is often enhanced by poor drainage and a seasonal rise in groundwater tables. The more granular slope materials are easily eroded upon weathering and the removal of the vegetation cover. Furthermore. underlying bedrocks are highly fractured with frequently steep and unfavourable bedding orientations within fold structures.

Mahabharat Lekh

2.13 The Mahabharat Lekh forms a steeper and more rugged ridge system to that of the Siwalik Hills and rises 1,800-2,000m above sea level. The height and steepness of this zone is due to the uplift associated with continued tectonic movements along the Main Boundary Thrust and rapid incision of the drainage network. Relative relief in the deeper valleys may be up to 1,500m, with average slope angles in excess of 35°. The Mahabharat Lekh is composed of a sequence of limestones, shales, quartzites and phyllites (fine-grained sedimentary and low-grade metamorphic rocks). Significant changes can occur over very short distances in rock type and structure, weathering grade, strength and depth of overlying soil mantle. Although rocks exposed at the surface are often in a weathered and fractured state, the constant stripping of this soil mantle due to surface runoff, landslides and erosion on steeply-sloping ground frequently prevents deep soils from developing. Soils are usually up to lm in depth, although residual soils on rounded and stable spurs can be in excess of l0m deep. Locally, landslide and colluvial deposits can exceed 30m in depth.

2.14 Vast quantities of water and sediment are transported on a seasonal basis. Erosion is clearly evident throughout much of this zone, although problems of instability, sediment accumulation and scour can be extreme in lower valley side and riverside locations. Most erosion can be attributed to natural forces; the influence of man in building well-managed agricultural terrace systems on virtually every slope less than 35° may have little more than a minor effect, if any, on erosion rates. However, mismanaged land uses have had significant and immediate effects locally. For instance, small catchments that have undergone widespread deforestation, overgrazing and poor land management can erode at rates of between five and ten times those that would be expected under normal slope conditions. Increased rates of erosion, siltation, widening of river beds and a tendency towards more flashy and ephemeral river flow are direct consequences of poor land management.

Middle Himalaya

2.15 The Middle Himalaya rises to an average elevation of 1,000-2,000m above sea level, although elevations increase to between 3,000 and 4,500m asl on the ridge tops in proximity to the High Himalaya. The rocks of the Middle Himalaya are predominantly schists and gneisses. They are mantled by weathered colluvial deposits and residual soils on the more gentle spurs and sloping ground at lower elevations, but they frequently form comparatively fresh and strong outcrop on the steeper slopes and in the zones of higher elevation to the north.

2.16 Generally, the Middle Himalaya is more stable than the zones to the south, because the landscape is less steep and the intensity of rainfall less extreme. With the majority of present-day seismic activity occurring in the vicinity of the Main Boundary Thrust, the Middle Himalaya generally falls within a seismic shadow and does not display the high rates of uplift and down cutting that are apparent in the Mahabharat Lekh. This zone is intensely cultivated and irrigated, and thus water management in agricultural areas is a key issue with respect to erosion and slope stability.

2.17 Along the main rivers, steep valley sides are often subject to rockfalls and rockslides, especially where under-cut by river erosion. This situation becomes more pronounced in the deeper and steeper-sided valleys towards the north, where the topography comprises cliffs, steep slopes, ravines and deep-seated slope failures. Many areas of flatter ground are likely to represent the remnants of ancient failure deposits. Slope erosion is far less a hazard here due to the general lack of erodible soil mantle, stronger underlying bedrock and a less intense rainfall and runoff regime. However, almost all main valleys emerging from the High Himalaya carry some threat of glacial lake outburst flood (GLOF), as demonstrated by a number of events during the last few decades.

High Himalaya

2.18 The High Himalaya comprises the great peaks that rise to 8,000m above sea level and associated inter-montane basins. The rocks of the High Himalaya are mostly high grade metamorphic gneisses and intruded granites, and the climate is periglacial and glacial. These regions are characterised by snow fields, drifting snow and snow avalanches, glaciers, scree slopes, rock falls, rock slides and rock avalanches, meltwater and flooding. Although road engineering in this zone does not form the subject of this guide, some references contained in the Bibliography provide background information.

3 ENGINEERING PROBLEMS

GENERAL CONSIDERATIONS

3.1 The various facets of the physical environment described in Chapter 2 present a range of difficulties and hazards that require careful consideration at all stages of a road project, but particularly during feasibility and design. Some of the more important considerations are listed below:

- choice of practicable and stable alignment
- choice of cross-section in difficult ground conditions with due regard of cost and alignment specifications
- drainage control and prevention of flood damage
- control of sediment and prevention of erosion
- foundation stability for retaining walls and other structures in colluvial or weathered soils, especially in riverside locations vulnerable to scour
- construction materials
- seismic design for major structures
- environmental impact, land acquisition and compensation, and drainage management through irrigated land

3.2 These considerations are discussed in the relevant chapters of this guide. A summary of engineering problems found in the Himalayan zones and mountain units defined in Figures 2.3 and 2.4 is given in Table 3.1. A brief description of terrain hazards that pose regular problems along Himalayan roads is given in the next section. Definitions of soil and rock terms used throughout this guide are given in Box 3.1.

TERRAIN HAZARDS

Landslides

3.3 Table 3.2 summarises the common forms of landslides and mass movements found in the Himalayas and outlines their typical engineering significance. Slope failures may involve two or more mechanisms, occurring either at different places on the slope, at different depths or at different times due to changes in ground conditions once initial failure has occurred. Furthermore, the engineering significance of a slope failure will vary according to whether it is a first-time or reactivated failure. First-time failures have an immediate effect on roads in their path but may not represent a continual maintenance problem because the failed mass may come to rest at an angle significantly lower than that from which it failed, and remain stable unless disturbed by toe erosion or seismic shaking. Reactivated slope failures on the other hand, and those that are expanding upslope due to progressive failure, are often the most problematic for road construction and maintenance

Box 3.1 Definitions of soil and rock terms

Soil and colluvium
Soil is completely weathered material that has lost all trace of rock structure and now consists only of mineral particles. Because of rapid weathering and constant movement downhill under gravity, most sub-tropical mountain soils are lacking in plastic clay minerals and are structureless. They also tend to be very stony. Colluvium is the name given to soils that lie on mountain slopes. Colluvium typically has the above features. It is 0.5-2m deep and often contains rock debris of different types, mixed up as the soil creeps down slope. The base of the colluvial soil profile tends to pass abruptly into weathered rock.

Weathered rock.
Weathered rock, in practical terns, is any rock that has undergone chemical degradation by the action of water, aided by warm temperatures. The effect of weathering is to weaken the rock and open up micro and macro fractures, allowing more water to get in and hasten the weathering process. Rock may be only slightly weathered, in which case its strength and integrity will be little affected. But when rock is highly weathered it becomes very weak and highly permeable. In this condition it may behave more like a soil than a rock. For instance, when failure takes place, shearing may take place along a plane through the rock mass (like a soil) rather than along pre-existing planes of weakness, as in a hard rock. These hybrid materials, which may represent the bulk of materials in an area, do not lend themselves to analysis by standard methods. Slopes often contain hard rocks, moderately soft rocks and very soft rocks, all within the same sequence.

A weathered rock can be described loosely as a soft rock, referring to its low strength. In this case the two terms have the same meaning, but 'soft rock' also refers to a specific type of material (below).

Soft rock
Soft rock can be defined as a rock whose fabric (body) is inherently weak. This weakness is due not to weathering but to inherently weak bonding between the mineral particles or assemblages of minerals. The rock can have a fully developed system of joints (internal parallel fracture sets). Geologically very recent rocks tend to have these characteristics; geological processes have not acted long enough to fully harden the rock.

Hard rock
Hard rock can be defined as any rock in a little-weathered state which, in failure, would shear along one or more of its natural planes of weakness. Standard techniques of rock slope analysis would apply.

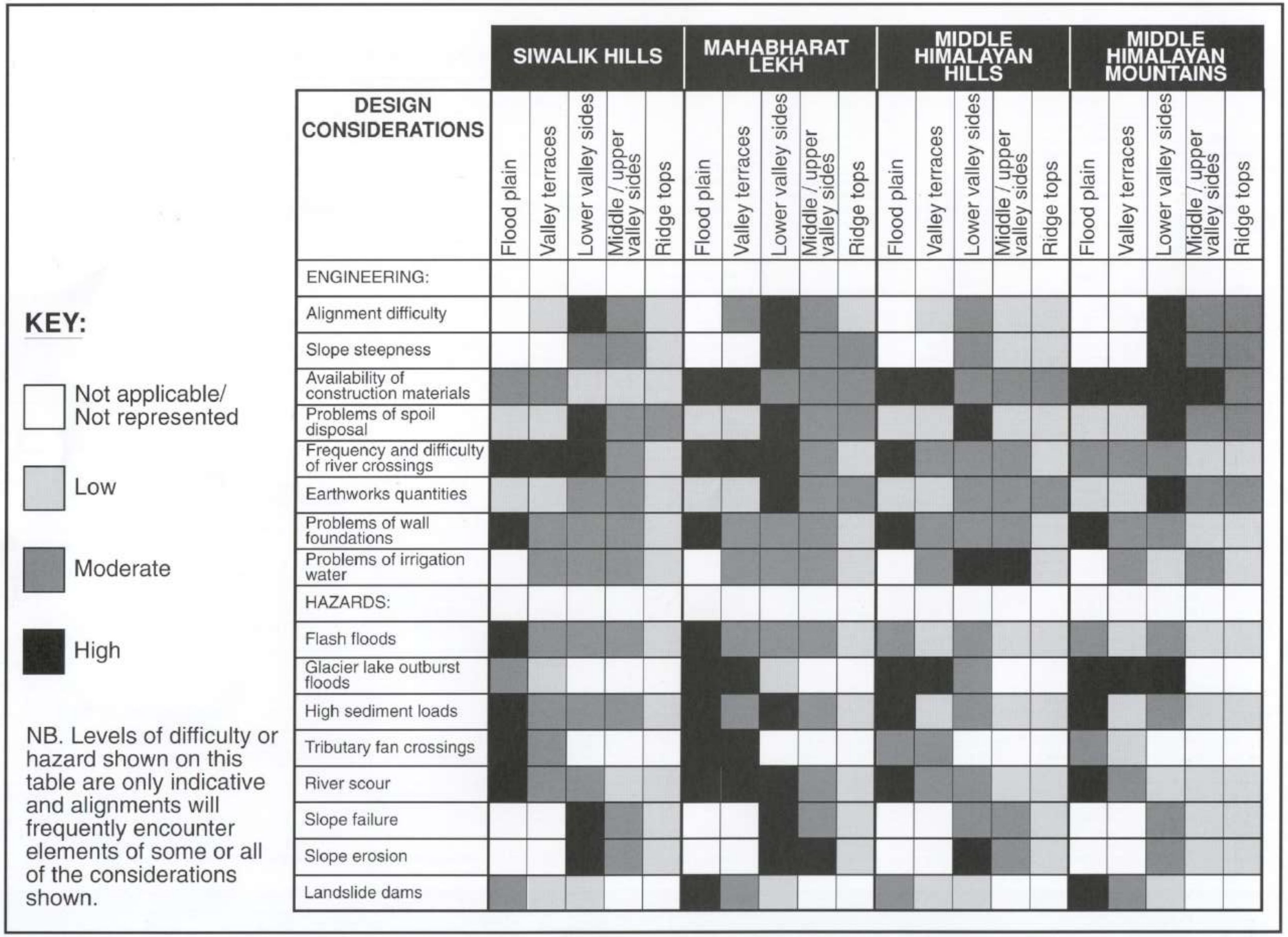


Table 3.1 Design considerations by physiographic zone in Nepal

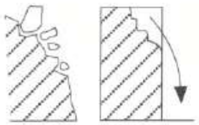
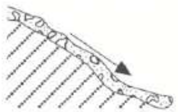
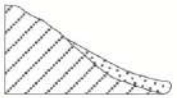
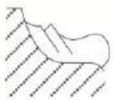
MECHANISM	TYPICAL GEOGRAPHICAL OCCURRENCE	TYPICAL FREQUENCY OF MOVEMENT	TYPICAL RATE OF MOVEMENT	TYPICAL DEPTH OF MOVEMENT	TYPICAL VOLUME OF MOVEMENT	COMMON CAUSES	COMMON ROCK / SOIL TYPES	GEOMORPH-OLOGICAL SIGNIFICANCE	COMMON IMPACTS ON ROAD CONSTRUCTION / MAINTENANCE
 SOIL FALL	From vertical river terrace banks and road cuttings	2 - 4 times each year	Instantaneous	0.5 -1m	$100m^3$ / yr per 100m of bank, or 50m of road cut	Undercutting, soil saturation	All coarse-grained soils	Very gradual terrace bank recession (<1m / yr)	Blockage to side drains from cut slope falls. Undermining if road is on terrace edge
 ROCK FALL	From mid-slope cliffs, steep lower valley-side slopes and road cuttings	Once each year - once every 20 years	Instantaneous	1 - 2m in road cuts, 5 - 10m on natural cliffs	$300m^3$ - $1000m^3$ / yr per km of cliff, per km of road cutting	Oversteepening, rock dilation, seismicity	Limestones, quartzites, granite-gneisses	Development of extensive talus slopes and river blockage in extreme cases	Blockage to side drains, carriageway and damage to road surface and structures
 DEBRIS SLIDE	Along soil / rock interface on steep lower valley-sides, in topographic depressions and road cuttings	2 - 4 times each year	1 - 3m / yr on natural slopes, instantaneous in road cuttings	Usually 1m, occasionally 3m	$2000m^3$ -$20000m^3$ / km^2 of natural slopes and 500 - $1000m^3$ / 100m of road cutting	Undercutting, over-steepening in excavations and soil saturation	All coarse-grained soils	Scarred hillsides following heavy rain, often forming upslope extensions to gully heads	Blockage to side drain and culverts and occasional road carriageway blockage
 MUDSLIDES	On moderately sloping lower valley-side slopes	Once or twice each year	1m / yr	Usually 1m, occasionally 3m	$2000m^3$ -$3000m^3$ / km^2 of natural slopes. Can occupy 50% of cut slopes in mudstone.	Undercutting, slope over-steepening and soil saturation	Fine-grained soils developed on clayey colluvium, mudstone and shale	Infrequent and slow moving	Progressive failure in cut slopes or loss of support from below. Gradual deformation of road surface and drainage structures
 ROTATIONAL SLIDE	Steep lower valley-sides and usually always adjacent to eroding streams	Usually instantaneous initial failure with later movements uncommon	Usually instantaneous followed by stability or slow (1m / yr) movement	Usually less than 10m, but on rare occasions 20m	Usually up to $250000m^3$. Rare on natural hillsides, although shallow rotational failures up to $5000m^3$ may occur in 5-10% of road cuttings	Undercutting of steep slopes, high groundwater. Failure may occur partially along a fault surface or clayey sub-stratum	Mudstones, shales, phyllites and schists	Comparatively rare except on clayey Siwalik and thrust materials. Slopes will usually fail to shallow angles and regain stability immediately	Depending on size, depth and configuration, first time failure will lead to road loss and later movements will cause deformation
 ROCKSLIDE	On steep lower valley-slopes frequently irrespective of rock structure and on hillsides where depth of strata is parallel to or slightly less than topographic slope	Usually instantaneous	Usually instantaneous	Usually up to 5m but occasionally 10m	Usually up to $10,000m^3$ but on rare occasions can be catastrophic (0.5 - 1million m^3)	Undercutting, high groundwater, adverse structural geology, seismicity	Limestones, phyllites, schists and granite gneisses	Rockslide debris can block watercourses and river valleys in extreme conditions. Rock relaxation and progressive rock fall can occur in rockslide back scar	Depending on size, depth and configuration, rockslide can result in various degrees of road damage including road loss

Table 3.2 Common forms of failure and erosion found in the Himalayas

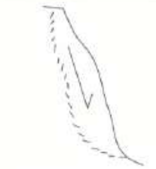
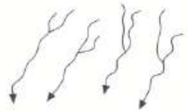
MECHANISM	TYPICAL GEOGRAPHICAL OCCURENCE	TYPICAL FREQUENCY OF MOVEMENT	TYPICAL RATE OF MOVEMENT	TYPICAL DEPTH OF MOVEMENT	TYPICAL VOLUME OF MOVEMENT	COMMON CAUSES	COMMON ROCK / SOIL TYPES	GEOMORPH-OLOGICAL SIGNIFICANCE	COMMON IMPACTS ON ROAD CONSTRUCTION / MAINTENANCE
 SACKUNG (para 3.8)	Steep and long rock slopes, usually adjacent to deeply incised rivers	Creep	Very low (a few mm / yr)	10 - 30m	Could involve entire valley side	Plastic deformation and dilation of fractured rock masses due to gravitational stress	Fractured hard rocks	Possible precursors to major rock avalanche failures	Very rarely encountered
 SLOPE EROSION	On unprotected soils and especially below footpaths and abandoned or poorly managed agricultural terracing	Significant erosion may occur up to 5 times each year on eroding slopes	Not applicable	Usually much less than 10mm depth of soil loss each year as a regional average	Usually less than $10m^3$ of soil from every $10,000m^2$ of eroding slope each year	Concentrated runoff and less frequently sheet wash over inadequately protected slopes	Silty, sandy soils	Mostly confined to 20 - 30° slopes in residual soils and finer-grained colluvial soils	Initial phase of cut slope erosion, blockage to side drains and erosion of unprotected embankments and spoil disposal areas
 GULLY EROSION	Found on steeper slopes where slope erosion concentrates into rills and where slope drainage encounters loose colluvium, silty soils or terrace gravels	Significant erosion may occur up to 5 times each year on eroding slopes	Not applicable	Usually less than 0.5m vertical incision / yr, although 5-10m scour depths can occur instantaneously due to concentrated runoff over loose soils	10,000 - $100,000m^3$ / per year from heavily eroding slopes or catchments of $1km^2$ in area	Concentrated runoff from natural sources or road drainage onto erodible soils / highly weathered rocks	Weathered (especially silty) soils and loose colluvium or terrace gravels	Gully erosion can develop rapidly following a change in established drainage patterns. Gullying will often initiate debris sliding on adjacent slopes	Scour of culvert outfall protection works and adjacent embankments, carriageway and culvert blockage
 DEBRIS FLOW	Steep channels with slope failure or erosion in headwaters or on adjacent slopes. High angle debris deposits (15°) prone to saturation	3 times each year	5m / s in steep channels, up to 1m / s on fan surfaces	Usually 1 - 2m, flow depth	10,000 - $50,000m^3$ of debris and water on at least 3 occasions each year	Saturation of debris deposits on steep slopes and concentration of wet landslide debris into steep stream channels	Coarse-grained soils and boulders	Rapid transfer of debris through catchments. Erosion of side slopes. Rapid fan development on adjacent flood plain terraces	Blockage of culverts. Impact damage to bridges and destruction of scour protection works
 FAN DEPOSITION	At abrupt concavities in stream profiles, especially where unstable channels discharge onto flood plains	Significant fan deposition may occur up to 5 times each year	Not applicable	Deposition rates of 1-2m during each event	Up to $2000m^3$ during annual maximum runoff from catchment area of $1km^2$	Highly unstable catchments and source areas with steep, unhindered transport	All rock and soil types, but especially weathered schists, phyllites and shales	Fan deposition on flood plains can alter existing flood plain drainage pattern and force flow against opposite valley-side	Frequent (up to 10 times each year) bridge blockage and / or carriageway inundation
 FLOOD PLAIN SCOUR	Where river flow is forced against river banks and valley meander bends in artificial and natural valley constrictions	Once or twice each year with extensive erosion every 10-20 years	Not applicable	Scour depths of 3.5m are normal. Occasionally 10m. Terrace banks can retreat tens of metres during individual floods	Not applicable	High sediment load and flash floods particularly in steep (>1 in 25) river channels	Loose terrace gravels and silty soils and fractured / weathered rock	Flood plain hydrology and topography can change dramatically during floods greater than 10 year rec. int.	Damage to river training works, road protection works and road formation at vulnerable sites

Table 3.2 Common forms of failure an erosion found in the Himalayas (continued)

because they have a Factor of Safety equal or close to unity, and are highly susceptible to comparatively minor changes in slope conditions brought about, for instance, by some road construction and maintenance practices. However, rates of slope movement may be relatively low and tolerable over short lengths of alignment.

3.4 Shallow soil falls, rock falls and rockslides, up to a depth of a few metres, tend to occur with intermediate or high frequency. Soil falls occur frequently from undercut river terrace banks and steep cuttings in soil, while rock falls are common on steep slopes formed in fractured rock, and especially from cut slopes during heavy rain. Shallow rockslides usually occur as planar failures along adversely-dipping bedding, foliation or joint surfaces, or as wedge failures along intersecting joint planes. However, in highly jointed rock masses typical of the Himalayas, potential failure surfaces are frequently encountered whatever the bedding/ foliation orientation with respect to the slope.

3.5 Debris slides are shallow planar failures in granular soils. Despite being the most frequent hazards encountered, debris slides and shallow slope erosion usually have little more than a nuisance effect on mountain roads. Debris slides usually occur in the soil or weathered mantle in response to toe erosion, rapid saturation of granular soils or a release of negative pore pressures during heavy rain in more fine-grained materials. Slip surfaces often occur along the interface between the weathered mantle or colluvium and the underlying rock. Debris slides are frequently found in the heads of eroding gullies and are subsequently quickly incorporated into an expanding drainage system. For instance, in the eastern Mahabharat Lekh, between 80 and 95% of debris slides have been found to occur adjacent to stream channels. Their life span is typically up to five years, although failure can continue for longer if toe support continues to be removed by erosion.

3.6 Mudslides are shallow planar failures in fine-grained, cohesive materials such as weathered mudstones and shales. They are often triggered by undrained loading. caused by failed material being deposited from above, or by toe erosion. Due to the predominance of granular non-cohesive soils on mountain slopes, mudslides tend to be infrequent.

3.7 The term rotational slide describes the mechanism of circular failure in fine-grained, cohesive soils and argillaceous rocks. Shallow rotational failures are almost exclusively confined to the more clayey colluvial soils and weathered mudstones. Deep-seated rotational slides are comparatively rare due to the preponderance of structurally-controlled planar or wedge failure, although they can form important components of complex failures in highly fractured rock masses when a single discontinuity or wedge structure is unable to control the entire failure mechanism.

3.8 Deep-seated planar rock slides usually take place along major bedrock discontinuities, or weak zones that are adversely orientated relative to the slope direction. They usually defy any low cost attempts at stabilisation or control, but fortunately they are comparatively infrequent. Landforms resembling those created by deep-seated rock relaxation (sackung), reported from the European Alps and North America, have also been identified on steep (70"), high valley sides in the mountainous Middle Himalaya of east Nepal. Sackung is caused by plastic deformation of rock masses due to gravitational stresses. It leads to gradual downslope creep and the creation of parallel head scarps above the failing mass. Loss of support by river erosion or seismic shaking, or even road excavation, can then trigger rock avalanche-type failure of the dilated rock mass.

Slope erosion

3.9 Slope erosion occurs almost exclusively where silty and sandy soils on slopes greater than 20° or so have no protective vegetation cover, or where sub-surface erosion pipes collapse to form gully heads. Most soils of the Low and Middle Himalayas fall into this category. Residual soils developed on phyllites, schists and gneisses, that have had their- clay minerals leached out in groundwater, are often prone to erosion in this way and are sometimes associated with an intricate pattern of rills and gullies that can erode to depths of 15 metres or more. Erosion usually occurs wherever surface drainage is concentrated onto unprotected slopes and is common below paths, roads and leaky irrigation canals. Slopes are at their most vulnerable to erosion during pre-monsoon and early monsoon rains when vegetation is least developed and storms are often most intense.

Stream erosion

3.10 The morphology of drainage systems adjusts to the moderate or high magnitude floods to which they are regularly subjected. The ephemeral and seasonal flows of the intervening periods have comparatively little effect on the drainage regime. Thus, the erosional effects of large storms and associated slope failures can remain visible and comparatively fresh for perhaps many years, until modified by another high magnitude event. The frequency of these events appears to be highest in the Low Himalaya, and may represent a recurrence interval as low as 5-10 years. Scour depths of up to 5 metres during these storms are not uncommon in this zone.

Box 3.2 Stream erosion in Nepal

Measurements taken before and after a particularly heavy storm of perhaps 50 years recurrence interval in the eastern Mahabharat Lekh indicated that stream channel cross-sections had increased by between 200 and 400%. By contrast, measured rates of channel erosion in small tributary catchments of the Arun Valley in the Middle Himalaya amount to an increase in cross-sectional area of between 7 and 15% per year. These lower rates of channel growth are thought to be representative of `normal' conditions.

Sediment transport and flood plain scour

3.11 Sediment tends to move in pulses through the drainage system, in response to intense storm runoff or the burst of a landslide dam or Glacier Lake Outburst Flood (GLOF). As the water subsides, the sediment is rapidly and unevenly deposited. The distribution of sediment on the valley floor is constantly changing, thus modifying the flow pattern of the river itself. The engineering consequences are threefold:

- the river scours different parts of the valley from season to season, removing terraces and undercutting the side slopes, causing new landslides
- deposited sediment can bury structures or block outlets
- the hydraulics of stream flow around bridge abutments and piers can alter significantly.

3.12 Where sediment yields are high, the abrupt decrease in slope angle at the junction of steep tributary streams and the adjacent valley floor results in the formation of debris fans. Fan surfaces may rise by 2m or more during the course of a single storm when floodwaters are insufficient to remove the debris being supplied. Fan incision during the following storm may remove most of the deposited material before further aggradation occurs. This cycle of deposition and erosion can occur on perhaps four of five occasions per year and give rise to problems of foundation scour and bridge or culvert blockage. Boulders up to 5m or more in diameter have been observed to move across fan surfaces during extreme flooding. Rapid fan growth from unstable tributaries has been known to temporarily block trunk rivers, impounding water and giving rise to a flood wave when the debris is breached in a manner similar to that of a landslide dam (paragraph 3.16).

River flooding

3.13 River flooding can have the following effects on roads and associated structures built in vulnerable locations:

- overtopping and associated scour of gravel road surfaces
- creation of temporary head and conditions of instability behind permeable road retaining structures if the rate of drainage dissipation during flood recession is slower than the fall in river stage
- temporary submergence of drainage structures with possible damage to bridge soffits by drift material
- blockage of drainage structures by sediment, and scour of embankments, retaining walls and flood control /protection structures
- undercutting of lower valley side slopes to create conditions of slope instability beneath road alignments.

3.14 Severe flooding can be either rainfall-generated or triggered by the failure of a landslide dam or glacier lake outburst. Although spring snow melt can significantly increase river levels, the effects on river hydrology are comparatively minor further downstream.

Rain-fed floods

3.15 These are by far the most common and are generated by (i) short, high intensity storms that have a comparatively limited geographical spread but have devastating flooding and erosion effects locally, and (ii) less intense but prolonged rainfall that frequently occurs over large areas of the Low Himalaya. In extreme cases, flood discharges of between 10 and 15 cumec/km^2 of catchment can be expected.

Landslide dams

3.16 On occasions, slope failures can be large enough to temporarily block river courses. This phenomenon is common in tributary valleys up to 501an2 in area, but less so in larger trunk valleys. A landslide dam is especially likely to occur at places where a constriction in the valley floor coincides with an unstable valley side. The problems for road alignment on valley floors or lower valley sides in this case are:

- the river level on the upstream side of the landslide dam rises rapidly, resulting in inundation, sedimentation, and disturbance of the hill slopes around the margins of the temporary lake
- a flood occurs, due to the sudden breach of the dam by overtopping or bursting through. The flood can turn into a debris flow super-charged with sediment and capable of transporting large boulders.

Glacier lake outburst floods (GLOFs)

3.17 A GLOF is a large surge of water caused by the breach of a dam of glacial debris impounding the waters of a glacier lake, high in the headwaters of a river valley. Valley glaciers in the Himalayas are located at altitudes of 3,000-5,000 metres above sea level and have been retreating since the turn of the century. Glacial moraine deposited in front of the retreating glaciers has, in some cases, caused lakes to form. It is estimated that there are at least fifty of these lakes in the upper Arun valley alone. Breaching of a moraine dam can occur when:

- heavy rainfall causes the lake level to overtop the moraine dam and scour the outfall
- a sudden rise of lake level occurs, due to an ice fall or landslide into it
- the moraine dam ruptures due to the melting of an ice lens in the core, or an earthquake.

3.18 GLOFs can cause great destruction and modification to river courses for many kilometres downstream. They occur without warning and with no easy means of prediction. Other effects associated with the passage of a GLOF include the transport, deposition and redistribution of large volumes of sediment and the temporary damming of tributary rivers. The passage of a GLOF can also have a long-term influence on suspended sediment concentrations, a four-fold increase being reported in the Tamur River in east Nepal following a GLOF in 1980.

3.19 The potential impact of a GLOF can be devastating to any engineering structures in its path. The impact depends upon the volume of water, the rate at which it is released, the attenuation of its flood wave downstream, the preexisting base flow in the trunk river and the volume of sediment carried and picked up during its flow. Case histories suggest that between 5 million and 20 million m' of water can be released within a period of 1-2 hours, with equivalent volumes of debris.

Box 3.3 Magnitude of some recent GLOFs in Nepal

A flood surge in the River Arun created by a suspected GLOF in 1969 had a maximum stage of up to 10 metres above normal river level in the more confined reaches. It is estimated that as much as 5-10 million m3 of water and a similar volume of debris flowed out in 1-2 hours, leading to a peak discharge of 4,000 cumec: an amount of water approximately equivalent to the runoff generated by a storm with a 100 year return period. A GLOF in the Bhote Kosi and Sun Kosi rivers in 1981 destroyed approximately 251rn of the Lamosangu-Kodari road, much of which was within 10m of the normal river level

Earthquakes

3.20 Earthquakes and ground tremors are common in Nepal, the majority of epicentres (the point or zone on the earth's surface directly above the origin of the earthquake in the crust) being located close to active thrusts or faults. Although it may not be practicable to design mountain road alignments to avoid these active zones, it is advisable to give consideration to the potential effect of seismic shaking when designing high retaining structures, and essential when designing bridges.

3.21 It is often the secondary effects of ground settlement, liquefaction and slope failure that cause the greatest problems for engineering stability. Earthquake-induced landslides, especially rockslides, rock falls and rock avalanches, often pose a greater hazard to life and engineering structures than earthquakes themselves, and can often occur several months after the seismic event, when the perceived risk is over. Rock falls and rockslides often occur when water penetrates comparatively strong rocks that have dilated during an earthquake. The quartzites, limestones and other competent rocks that crop out along the Mahabharat Lekh and the hard granite/ gneisses along the Main Central Thrust appear to be particularly prone to such failures.

3.22 The three most important variables that control the behaviour of saturated cohesionless soils during seismic shaking are:

- relative density (low density materials reduce in volume during shaking, causing deformation in engineering structures)
- confining pressure (low confining pressure allows the material to dilate)
- drainage conditions (pore pressures can build up rapidly during dynamic loading, causing the soil to loose strength or even to liquefy).

Box 3.4 Seismicity in Nepal

Seismicity contours indicate that the highest concentrations of earthquakes in Nepal occur in the foothills between Kathmandu and Dharan, especially along the Main Boundary Thrust. Prehistoric rock fall and rock avalanche deposits are frequently found at higher altitudes in the Middle Himalaya and many of these may have occurred as a result of earthquake shaking and dilation of surface rocks associated with seismicity along the Main Central Thrust.

The impact of the August 1988 earthquake in east Nepal illustrates points in the text. The earthquake epicentre was 65km west of Dharan with a shallow focal depth, a strength of 6.6 on the Richter Scale and a reported intensity of VIII-LX in Dharan on the Modified Mercalli Scale. The last comparable earthquake in Nepal occurred in 1934 and destroyed large areas of Kathmandu and neighbouring towns (which are built upon fine-grained lake sediments). Although in the Dharan-Dhankuta area there is evidence for deep-seated rock relaxation, especially in the vicinity of the Main Boundary Thrust, the majority of failures occurred in the quartzites of the Mahabharat Lekh and on scree slopes where severe shaking caused failure of road retaining walls. However, the event probably resulted in extensive dilation of surface rocks along the Mahabharat Lekh because heavy monsoon rains a few weeks later caused widespread erosion and shallow slope failure. The Dharan-Dhankuta road was blocked in over 30 places, and a total of more than 2,OOOm'/km of slip material had to be cleared from the road surface. Further north, where the rocks are less fractured, the surface effects of the earthquake were minimal, although isolated rock falls occurred on the steeper rocky flanks of the Arun valley in the Middle Himalaya.

Road construction impacts

3.23 Road construction can have significant effects on slope stability, drainage, erosion and sediment supply to drainage networks. Studies from the Himalayas in Nepal and India indicate that cut slope failures after construction can generate an average of 500mYkm/yr of debris, and that up to 2,000m'/km can be generated during single storms with 10-20 year recurrence intervals. Erosion rates in small catchments significantly affected by road construction can be at least 10 times those expected under natural conditions. Environmental impacts of road construction are discussed further in Chapter 6.

4 PLANNING AND DESIGN OVERVIEW

INTRODUCTION

4.1 This chapter describes general matters that relate to planning road construction in mountain areas. The sequence of activities that constitutes the progression of a road project from inception through to operation is summarised below and in Figure 4.1:

- project identification
- feasibility studies, relating to both the basic engineering and an assessment of the route corridor (includes the desk study)
- design of engineering works, usually in two phases (preliminary and detailed)
- commitment of funds, which often takes place in a series of stages, followed by invitations to tender and negotiation with contractors, potential financiers and suppliers
- implementation of construction, typically either by direct labour or by one or a series of construction contracts
- operation, maintenance and monitoring.

4.2 A systematic approach to project assessment and design should be adopted on all roads irrespective of their intended function or standard. However, for reasons of technical complexity, sparseness of geo-technical data, physically difficult communications and physically constrained working, planning procedures cannot be followed so logically as when building in lowland areas. Although the phases of the Project Procedure comprise a logical sequence, in practice, modification may be required to the timing of phases and the extent to which the various studies within each phase are carried out. For instance, feasibility and preliminary design often overlap and combine into a single progressive phase. Similarly, final design is often not concluded until ground conditions are fully revealed during construction. These considerations are illustrated in Table 4.1, which summarises the factors to be taken into consideration during the various phases of the Project Procedure. A relative order of importance or priority is assigned to the inputs within each project phase.

4.3 The most important effect of a flexible approach to design is that final design of the cross-section and retaining structures may be deferred until the time when the ground is opened up in the early stages of road construction; up to that point all designs should be regarded as preliminary. The aim of the feasibility study and design should be to work towards a final alignment and thence to a design, rather than to retain fixed ideas about designs from early on. The engineer should be prepared to modify the design in the light of information as it becomes available. The information-gathering stages are thus the most important part of the design process, because they steer the engineer towards an optimum design in circumstances in which design decisions are often very difficult to make.

4.4 The evolution of a final design is achieved by investigating the site in several phases, at each phase selecting smaller areas to look at in greater detail, using techniques appropriate to each. The phases are:

- desk study and reconnaissance surveys during the feasibility phase to summarise terrain conditions and to identify and compare route corridors, design concepts and costs, and review environmental considerations
- geotechnical mapping, based on geomorphological or landform mapping along route corridors to identify constraints on detailed alignments and to define broad geotechnical parameters for design
- identification of preferred route corridor
- hydrological investigations and hydraulic design that are carried out in progressive detail from regional-specific at the feasibility stage to site and structure-specific during detailed design
- geotechnical investigations at selected locations along the chosen route to further define geotechnical parameters
- design of alignment, cross-section and associated drainage and retaining structures
- geotechnical investigation during road construction, carried out as cuttings and foundations are exposed, from which the final designs are produced.

4.5 In carrying out these investigations, it is important to be aware of the following factors concerning road construction in active fold mountains:

- problems are likely to be encountered whichever route is chosen and the choice of alignment is very often a tradeoff between one constraint and another, or one hazard area and another, and thus it is important to carry out comprehensive feasibility studies to establish hazard and risk assessments for decision-making
- not all hazards and ground problems will be identifiable prior to construction and therefore remedial action and contingency measures should be allowed for

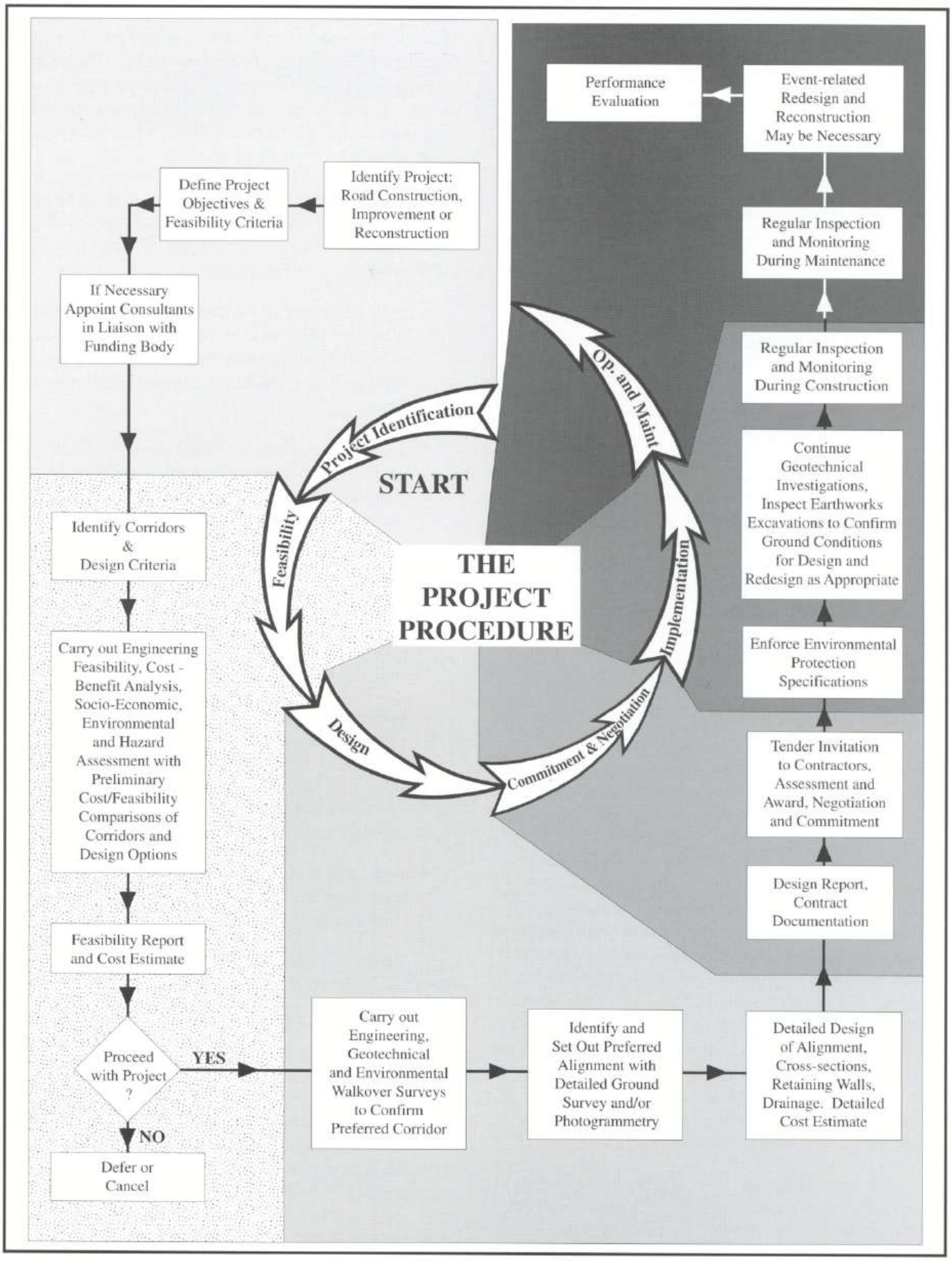


Figure 4.1 The Project Procedure

● Major Input
◍ Moderate Inputs
○ Minor Inputs

DESIGN FACTORS → / ACTIVITIES ↓	CORRIDORS													
	Project Objectives	Route Corridors	Route Length	Engng. Difficulty	Earthworks Vol	Hard Rock Vol	Retng. Wall Vol	Bridges & Spans	Culvert Quantities	Slope Stability	Erosion Potential	Scour Potential	Flood Potential	Materials
IDENTIFICATION														
Scope, funding, mngement.	●													
FEASIBILITY														
Published Literature		○			○	○	○		○	◍	◍	◍	◍	
Topographical Maps		●	●		●		○	◍	○				◍	
Geological Maps		○		◍		◍		◍		◍	○		◍	
Aerial Photographs		●	●	●				●	○	●	●	◍	◍	
Terrain Classification		●			●	○	●			●	●		○	
Hydrological Data								◍	○	◍		◍	●	
Terrain Hazard Maps		●						◍		●	●			
Published Data														
Traffic Surveys														
Economic Assessment		◍												
Engng. Reconnaissance		●	●	●	●	●	●	●	◍					
Engng. Geol. Reconn.		●	●		●	●	●	●		●	●	◍		◍
Environmental Reconn.		◍	◍											
Desk Study Review														
Feasibility/Cost Report	CONFIRM FEASIBILITY OF PROJECT, CHOOSE CORRIDOR AND DESIGN STANDARDS													
DESIGN														
Detailed Engng. Walkover														
Detailed Engng. Geol Surv.														●
Engng. Hydrol. Survey									●					
Environmental Assessment														
Geotech. Investigation														
Route Flagging														
Ground Control														
Photogrammetry									●					
Centre Line Survey														
Land Acquisition Survey														
Environ. Baseline Surveys														
Design/Cost Report	PREPARE DETAILED DESIGN AND COST ESTIMATE													
Contract Documents														
IMPLEMENTATION														
OPERATION														

DESIGN FACTORS → / ACTIVITIES ↓	SOCIO-ECONOMIC							CONSTN. PLANNING							
	Traffic Volumes	Vhcle.Optg. Costs	Design Standards	Agric. Production	Import/Export	Population Profiles	Cost/Benefit	Contractual	Timetable	Rates of Progress	Climate Constraints	Pilot Tracks	Work Fronts	Spoil Disposal	Borrow Areas
IDENTIFICATION															
Scope, funding, mngement.			◍					◍	◍						
FEASIBILITY															
Published Literature			◍								○				
Topographical Maps						◍									
Geological Maps															
Aerial Photographs						○						●		◍	◍
Terrain Classification															
Hydrological Data															
Terrain Hazard Maps															
Published Data	●	●	◍	●	●	●									
Traffic Surveys	●	●													
Economic Assessment	◍			●	●										
Engng. Reconnaissance			◍					◍	◍	◍	●	○	○	○	○
Engng. Geol. Reconn.															
Environmental Reconn.															
Desk Study Review							●								
Feasibility/Cost Report															
DESIGN															
Detailed Engng. Walkover								●	●	●		○	●	●	○
Detailed Engng. Geol Surv.										◍		◍		●	●
Engng. Hydrol. Survey											●	◍		◍	
Environmental Assessment														●	◍
Geotech. Investigation															
Route Flagging															
Ground Control															
Photogrammetry															
Centre Line Survey															
Land Acquisition Survey															
Environ. Baseline Surveys															
Design/Cost Report															
Contract Documents								●	●	●	●	●	●	●	●
IMPLEMENTATION															
OPERATION															

DESIGN FACTORS → / ACTIVITIES ↓	ENVIRON. IMPACT						DESIGN														
	Conservation	Protection	Land Use Impact	Compensation	Erosion Impact	Pollution	Alignment Design	Earthworks Design	Spoil Disposal	Retng. Wall Design	Foundn. Design	Drainage Design	Flood Freeboard	Bridge Design	Construction Mats.	Slope Stabilisation	Erosion Protection	Forest Extension	Agric. Management	Redesign	Reconstruction
IDENTIFICATION																					
Scope, funding, mngement.																					
FEASIBILITY																					
Published Literature																					
Topographical Maps							◍														
Geological Maps																					
Aerial Photographs							◍														
Terrain Classification							○														
Hydrological Data																					
Terrain Hazard Maps							◍														
Published Data																					
Traffic Surveys																					
Economic Assessment																		◍	◍		
Engng. Reconnaissance																					
Engng. Geol. Reconn.																					
Environmental Reconn.	◍		◍																		
Desk Study Review							◍	○	○	○		○	○	○	○	○	○				
Feasibility/Cost Report																					
DESIGN																					
Detailed Engng. Walkover							◍	◍	◍	◍	◍	◍	○	◍	◍		○				
Detailed Engng. Geol Surv.							◍	●	●	◍	●				●	◍	◍				
Engng. Hydrol. Survey							◍					●	●	◍		○	◍				
Environmental Assessment	●	●	●	○	●	●	◍	○	◍												
Geotech. Investigation							○	●		◍	◍			○	●	●	◍				
Route Flagging							●														
Ground Control							◍														
Photogrammetry							●	◍													
Centre Line Survey							○														
Land Acquisition Survey				●													○	◍	◍		
Environ. Baseline Surveys		◍	◍	◍		◍												◍	◍		
Design/Cost Report																					
Contract Documents	○	●			●	●	●	●	●	●	●	●		●	●	◍	◍		○		
IMPLEMENTATION					●			●	●	●	●	●	◍		●	●	●	◍	●	●	◍
OPERATION					●			◍	○	◍	○	○	○		◍	●	●	◍	●	●	●

Table 4.1 Procedural checklist

- road construction across unstable slopes can result in construction costs that are over twice those incurred on stable ground of similar slope angles and materials, and maintenance costs will also be higher
- earthworks and retaining walls together can represent up to 50% of the total construction cost, and both items increase disproportionately with increasing slope steepness and formation width
- the average cost per year of routine maintenance in real terms amounts to approximately 3-5% of construction cost
- it would appear from the limited data available that, every 10 years or so, the occurrence of an earthquake or an intense rainstorm is likely to necessitate substantial road reinstatement works that can cost between 5 and 10% of the original construction cost allowing for inflation.

PROJECT IDENTIFICATION

4.6 The government organisation responsible for road works should take an active and informed part in project decisions, especially in defining the scope of works to be undertaken. It will be necessary, for instance, to define:

- the fundamental objectives of the project
- factors governing the selection of a route corridor
- detailed design standards
- a project implementation strategy.

4.7 It is important to list the specific project objectives in order of perceived priority. These might vary from the provision of rapid access to the site of a planned mine to the construction of all-weather access to villages and towns as part of a broader regional development programme. A clear order of priorities determined at the outset can avoid conflicts of interest that could arise later, and help establish a design standard most appropriate to the intended function of the road and the resources available for its construction and maintenance.

4.8 In developing countries, when a project requires large resources or the provision of specialist expertise, the host government or funding agent usually elects to employ external consulting and contracting organisations. Activities which normally require the appointment of such companies in association with local firms typically include:

- contract strategy studies
- environmental impact studies
- photogrammetric mapping
- feasibility studies
- geotechnical studies
- detailed survey
- detailed design
- construction
- construction supervision
- construction management.

FEASIBILITY STAGE

4.9 The feasibility stage combines a desk study with preliminary walkover and reconnaissance mapping surveys to review the cost/benefit, practicality, stability and environmental consequences of the proposed works, and leads to the identification of a preferred corridor and engineering design approach. This phase is intended to quickly identify all factors that are relevant to design (Table 4.1) and should be multi-disciplinary. It is usually the most crucial stage of a project, when decisions are made that are fundamental to achieving an alignment that can be constructed and maintained with least cost, minimum instability and tolerable environmental impact.

4.10 The feasibility study should also include a wider range of issues such as:

- geometric standard and design life.
- comparative cost estimates
- traffic forecasts (these are outside the scope of this guide)
- socio-economic and environmental assessments
- road safety assessment (this is outside the scope of this guide).

Route corridor assessment

4.11 Broadly, route corridor options can be divided into the following categories:

- ridge top
- compound alignments that cross several types of topography, with vertical climbing sections (hairpin stacks)
- valley floor.

a) Ridge top alignments

4.12 These are often the most stable and least costly, and they are also favoured on socio-economic and environmental grounds, as they usually follow established lines of communication and habitation. Usually, however, steep slopes and abrupt changes in ridge-top elevation dictate that alignments are frequently required to traverse side-long ground beneath ridge tops. When there is a choice as to which side of a ridge an alignment is to be located, it is recommended that the choice be made on the basis of the following, in order of preference:

- select the side on which bedding, foliation or the principal planes of rock weakness dip into the hill-side, so that they are not undercut by excavation. However, in such situations, slopes frequently tend to be steeper and irregular, thus posing practicable problems for road construction. On the other hand, under more gentle slope conditions, deep soils may keep the outcrop of rocks in cuttings to a minimum, reducing the importance of rock structure in influencing earthworks stability

- the leeward slope, sheltered from the prevailing direction of rainfall

- a sunny aspect, so that soils are drier and rocks are less weathered.

4.13 In the Himalayas the last two preferences are often contradictory as the prevailing direction of rainfall is usually from the south, in which case the leeward slope is likely to be more advantageous.

b) Valley side alignments

4.14 Climbing sections of mountain roads can be designed as gradual traverses of side-long ground at a ruling gradient, or as vertical climbing sections (hairpin stacks), or as a combination of the two. The hairpin stack has the following advantages:

- a greater flexibility in route corridor location can normally be achieved

- the crossing of difficult ground, and in particular steep and unstable lower valley sides, can be minimised or avoided altogether

- the use of hairpin stacks to connect lengths of relatively easy ground, such as river terrace or ridge tops, can in some cases lead to a more direct alignment with a saving in overall route length.

4.15 The main problems associated with the hairpin stack approach are:

- on slopes steeper than 30° limited space to construct cut and fill slopes, necessitates either a relaxation in geometric standards or more expensive retaining structures

- lack of suitable spoil disposal sites and access difficulties for plant can pose difficulties during construction

- instability and erosion can easily extend from one loop of the road to another, both upslope and down slope

- storm runoff tends to become concentrated, for which large-capacity drainage structures and erosion protection works are required. The risk (cost) associated with failure of any part of the drainage system is usually high.

4.16 If the topography allows, the problems associated with stacked hairpins can be reduced by creating offset stacks, in which the hairpins are not immediately above one another but are staggered across the slope. This will minimise drainage problems and limit the danger of instability to fewer hairpin loops.

c) Valley floor alignments (see also Chapter 13)

4.17 The advantages of a valley floor alignment are:

- that relatively little climbing and descent are involved, thus making route alignment easier and shorter, with correspondingly lower vehicle operating costs

- a ready supply of construction materials is available (although its quality may not be ideal)

- control of spoil disposal and construction of pilot tracks can be less stringent.

4.13 Against these advantages, routes in the confines of valleys generally have little opportunity, other than by expensive bridging from one side of the river to the other. to avoid problems of awkward topography and instability, and they may be susceptible to river flooding and scour, as well as debris fall from instability above. Despite the attractiveness of a more direct route and low gradients, the construction cost of a valley floor alignment may be significantly higher than a hill route alternative, even though the valley route may be substantially shorter, because of the high costs of bridging, cross-drainage, scour protection and rock excavation in confined areas. If flooding and slope instability on the lower valley sides are recurring hazards, maintenance costs can also be significantly higher. On socio-economic grounds, valley routes may be less favoured if the majority of villages are located on ridge tops.

Design standard and design life

4.19 In mountain areas, selection of geometric standard for low cost roads should take full account of the constraints imposed by the difficulty and stability of the terrain and, if necessary, reduce locally the design standard in order to cope with exceptionally difficult terrain conditions. For instance, to avoid or minimise the crossing of difficult ground, maximum average ruling gradients of 5% for trunk roads and 7%n for feeder roads could be increased to up to 7% and 9%n respectively, with a maximum gradient of up to 9% and 12% over distances of 300m. A formation width of 6.5 metres including the side drain could be reduced to 5.5 metres in particularly steep and unstable areas. On steep ground, a horizontal curve radius of as little as 10m may be unavoidable, even though a minimum of 25m is usually specified. As outlined in Box 4.1, the construction of a high design road will usually result in high repair and maintenance costs later.

4.20 Usually, some elements of design are considered to have an infinite design life, eg the embankment, while others have a finite design life, eg the pavement. Those elements that have a finite design life gradually deteriorate at a known rate under a predicted amount of wear and tear. These finite-life elements have different design lives according to their capital cost and the nature of the stresses imposed on them. Bituminous pavements are designed to last for 15-20 years, and bridges for 50-120 years.

4.21 In mountain areas, elements which are normally thought of as having an infinite life may run a finite risk of being damaged or destroyed by external forces of erosion and slope failure within, say, 25 years. Thus, while drainage structures are designed according to an acceptable design life, retaining structures and earthworks slopes are designed to an acceptable factor of safety based on a comparison of anticipated construction cost, and the cost and risk implications of failure or recurrent maintenance.

Erosion protection

4.22 Under conditions of limited funds, faulty construction practices or inadequate vigilance, any delay in implementing essential protection or precautionary measures can lead to serious problems later, requiring more expensive remedies. Furthermore, the effectiveness of maintenance organisations cannot always be guaranteed, thus emphasising the importance of adequate investment during construction. However, it is usually not cost-beneficial or even technically feasible to construct a road in an unstable mountain area that will be entirely free of instability problems. It is inevitable that damage or loss will occur to parts of the road, and perhaps within only a relatively short period after construction. The only sensible option is to decide upon a level of slope protection that reduces normal rates of damage to an acceptable level. The objective should be to design a road that can be built and maintained at affordable cost, within the technical and operational capabilities of the country, that functions at an acceptable level of safety to the public. If it is not economically possible to maintain long term road stability. then the decision to proceed at all with road construction should be seriously questioned.

Box 4.1 New roads in Nepal damaged by natural disasters

In the Middle Himalaya of Nepal, two years after a mountain feeder road had been completed, a major storm caused the loss of 600 metres of road. The scale of the destruction could not have been foreseen during construction, and even if it had, the cost of the extensive protection works that would have been required would not have been justifiable within the prevailing budget. The total cost of reinstatement works amounted to the equivalent construction cost of ten kilometres of road (approximately 10% of the alignment).

On a comparatively high standard road in the Mahabharat Lekh, earthquake and storm damage in one year resulted in a reinstatement cost that, allowing for inflation, again amounted to approximately 10% of the original road construction cost. Case histories from the Indian Himalaya tell of similar disasters well into the maintenance period. For instance, two storms to Sikkim caused the loss of 18 kilometres of road and ten bridges. Expenditure on disaster repairs may recur every 5-10 years, and is superimposed onto a `normal' maintenance cost that in any case may be 3-5%n per annum of the total construction cost after inflation.

The limited evidence available suggests that costs of maintenance in unstable mountainous regions are, in the long term, proportionate to the costs of construction. In a marginally stable environment, high design standard does not necessarily insure against damage. Some form of damage to the road to future is almost inevitable. Thus, a higher design standard results in higher repair and maintenance costs later. Add to this the fact that a high design standard inevitably disturbs the physical environment to a greater extent than a lower standard. Such disturbance means that adoption of a high design standard can actually increase the risk of failure, and the cost of reinstatement will be the greater also.

Feasibility study report

4.23 The feasibility study report will usually comprise the following:

- project identification, objectives and scope of study
- an account of broader development objectives
- identification of corridor and/or engineering options, with justification of preferred options and conceptual designs
- road safety issues related to design
- review of environmental issues including potential benefits and disbenefits

- review of existing traffic volumes (vehicular, population and livestock)
- import/export and origin-destination surveys, and traffic forecasts
- reductions in vehicle operating costs in the case of road improvement projects
- socio-economic impacts
- economic analysis
- tabulated comparisons of cost, stability and environmental aspects of corridor(s)
- cost and logistical summary of preferred corridor, review of funding requirements, project management requirements, construction planning and maintenance programme
- method of implementation.

DESIGN

4.24 The design stage is often divided into preliminary design and detailed design. Preliminary design is employed in order to avoid too much effort going into detailed design before a project is given the final go-ahead. A preliminary design would be appropriate if, for instance, the funding agency requests a more thorough review, than is usually achieved by the feasibility study alone, of the construction timetable, implementation programme and cost profile before committing financial resources to the project. The preliminary design can be carried out either as a separate exercise or as a logical extension to the feasibility phase. The additional requirements of a preliminary design are to:

- further investigate the engineering and cost implications of terrain hazards and difficult ground conditions
- carry out ground survey and/or photogrammetric mapping from aerial photographs to provide a more accurate ground model for estimation of quantities
- carry out detailed survey and, where possible, geotechnical or geophysical investigations at major bridge sites.

4.25 The above activities should only be undertaken within the preferred corridor and local variations to it, The various stages of the design phase are discussed below, although it must be reiterated that design rarely ends when construction commences, but rather evolves with experience as construction proceeds.

Detailed survey and alignment design

4.26 With the route corridor confirmed, the alignment engineer, preferably accompanied by an engineering geologist or similar specialist, with a survey team, will flag the approximate centre-line. Considerable savings in time can be achieved if an approximate alignment is drawn onto photogrammetrically plotted contour maps (see below) and enlarged prints of aerial photographs in the office prior to embarking on detailed fieldwork.

4.27 If slope stability is critical to the alignment, then geotechmcal mapping surveys should be undertaken at scales of between 1:1,000 and 1:5,000. It will be easier for personnel to locate themselves with the required accuracy if an approximate centre-line has been set out, but the engineer should be prepared to modify the location of the centre-line in the light of the geotechnical survey. In very difficult ground, these surveys should ideally be carried out prior to the centre-line flagging exercise using aerial photograph enlargements or compass traverse as a means of location positioning.

Box 4.2 Photogrammetry

The processes of detailed survey, alignment design and setting out are time consuming, especially if changes to the alignment are made later owing to unforeseen ground conditions or changing design criteria. The use of photogrammetry can speed up these procedures and provide the flexibility to allow additional off-site engineering works such as access to borrow pits, spoil disposal sites and slope drainage works, to be designed at a later date.

As an example photogrammetry from aerial photographs of 1:25,000 scale can yield uncontrolled contour mapping at a maximum scale of 1:5,000, with contours at 5 metre intervals. It is advisable to correct the contour model by establishing two ground control points in each stereo pair, by tying points on the photographs either to the national or local grid, or by GPS. The main problems associated with the use of photogrammetry relate to the lack of ground definition in areas of shade, cloud or dense forest cover.

4.28 With the alignment confirmed, detailed design of all subsequent works can proceed. Design of the detailed vertical and horizontal alignments will require topographical mapping at a scale of 1:1,000 with contour intervals at a maximum of 2m, using ground survey, photogrammetry or a combination of the two. Ground survey may be preferable at this stage due to the greater survey accuracy required. The use of photogrammetry will require the establishment of a base line traverse and the commissioning of air photography at a scale of between 1:5,000 and 1:10,000. Plan and profile drawings, and schedules of earthwork and retaining wall designs and quantities can then be produced for contract documentation.

4.2 One of the most important aspects during detailed design is to ensure that the right-of-way is defined adequately, so that the road can be constructed along the optimum alignment. Further detailed topographic survey can then be carried out after site clearance but before construction in order to optimise road geometry and minimise quantities. A design capability should therefore be provided on site during construction, in order to attend to the needs of the detailed engineering and geotechnical assessment.

COST ESTIMATION

4.30 Cost comparisons should be based on the expected costs of design, erosion protection, construction, maintenance and environmental mitigation. At the preliminary design stage, construction costs are usually estimated by sub-dividing the alignment corridor into segments according to slope angle, preferred cross-section and estimated quantities of soil and hard rock in excavations. The placement of fill on slopes steeper than about 30° requires the construction of an embankment retaining wall, the size and cost of which is estimated per centre-line metre for each combination of cross-section and slope angle. At the detailed design stage, the site-specific selection of cross-section, slope design and drainage and retaining structures is combined with the ground model and the designed alignment to yield quantities for cost estimation.

4.31 Table 4.2 gives approximate quantity estimates for the various combinations of design cross-section and slope angle. These data were derived from simple trigonometrical calculations based on the various assumptions given in the table, and are included here for approximate comparisons only. Clearly the quantities are also a function of formation width and for a road design in Nepal, reducing the formation width from 5.5 to 4.5m, led to a reduction of 20% in pavement and retaining wall quantities, and 15% in cut and fill volumes. Non-continuous quantities such as culverts and slope protection works can be estimated by reconnaissance mapping or on a pro-rata basis from other roads constructed under similar slope conditions. In many projects, it is often found that cost estimates increase by 10% or more between successive phases of the estimating process and there have been occasions when the estimate at the feasibility stage has been less than 50% of the detailed design cost. These underestimates are particularly prevalent in mountainous areas and usually stem from oversimplification of topography, changes to alignment during design and inadequate geotechnical assessment.

4.32 A broad indication of road construction costs in Nepal, and their geographical variation is given in Box 4.3.

Box 4.3 Road construction cost comparisons front Nepal

At 1996 prices, the construction cost per km for 5.5m formation width sealed roads in Nepal varies between US$0.1-0.3 million. If construction is to be carried out by an international contractor using machine-intensive methods, these construction costs can be more than doubled. In this case, the contractor's establishment cost can be 30% of the total billed items. There is a geographical pattern to the range of construction costs given above:

- construction costs in the Low Himalaya can be almost double those for the same design specification in the Middle Himalaya, due to generally steeper ground and significantly greater drainage and instability problems encountered

- depending upon topography, middle valley side alignments can be between 30% and almost 100% more costly to construct than ridge top alignments. Lower valley side and valley floor alignments can be up to 300% more expensive than those located on gently undulating ridge tops

- from the limited data available, construction to a 6.5m formation width can result in up to 70% increases in per-km costs over a 5.5m formation to all geographical cases.

CONTRACTS

Contract conditions and specifications

4.33 Any provisions peculiar to the site will be allowed for in the special conditions of contract and the special specification. In Nepal, and in mountainous areas generally, the following topics are likely to require attention:

- the need for the contractor to regularly clear silt washed down from newly-completed construction operations, from drains and drainage structures, as part of his maintenance operations

- the possible need for the contractor to work alongside traffic

- limitations on the use of pilot tracks and the requirement to protect adjacent slopes from erosion control on the dumping of spoil material

- erosion control and other environmental protection measures associated with land clearance, earthworks, drainage works, borrow areas, temporary stockpiles, the construction of work camps and the use of forest products

TERRAIN TYPE	SLOPE ANGLE (degree)	CUT ANGLE	CROSS - SECTION TYPE AND QUANTITIES (m^3/ m run)																							
			FULL CUT			2/3 CUT – 1/3 FILL						1/2 CUT – 1/2 FILL						1/3 CUT – 2/3 FILL						FULL FILL		
			Hard rock	Soft rock	Soil	Hard rock	Soft rock	Soil	Fill	Retaining wall type	Retaining wall volume	Hard rock	Soft rock	Soil	Fill	Retaining wall type	Retaining wall volume	Hard rock	Soft rock	Soil	Fill	Retaining wall type	Retaining wall volume	Fill	Retaining wall type	Retaining wall volume
HILLY	10	1:1	–	–	2.7	–	–	1.2	0.3	–	–	–	–	0.7	0.6	–	–	–	–	0.3	1.3	–	–	3.0	–	–
	20	1:1	–	1.2	5.9	–	–	3.1	1.1	–	–	–	–	1.8	2.5	–	–	–	–	0.8	4.5	–	–	10.0	–	–
	30	1·5:1	0.5	3.8	7.5	–	0.8	4.4	0.6	G	2.5	–	0.1	2.8	1.2	G	2.5	–	–	1.3	2.1	G	4.5	5.4	G	7.0
	40	1:5:1	8.9	11.3	3.5	1.8	6.4	2.3	1.0	G	4.5	0.3	4.0	1.7	1.9	G	4.5	–	1.5	1.1	2.3	G	7.0	6.2	G	9.0
	50	2:1	20.2	10.8	–	6.0	10.3	–	1.2	M	4.7	2.1	7.1	–	2.5	M	7.0	0.4	3.7	–	4.1	M	8.7	–	–	–
	60	10:1	26.2	–	–	5.5	–	–	1.6	M	7.8	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	70	10:1	47.4	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

NOTES:

1. Figures are illustrative only for a 5.0m formation width.
2. Cut angle = vertical:horizontal.
3. Hard rock = blasting.
4. Soft rock = rippable/excavated by dozer.
5. G = Gabion
6. M = Masonry
7. Slope angle (degree) | Assumed weathering profile

Slope angle (degree)	Assumed weathering profile
10	2m soil, 1.5m soft rock, hard rock
20	1m soil, 1.5m soft rock, hard rock
30	1m soil, 1m soft rock, hard rock
40	0.25m soil, 1m soft rock, hard rock
50	1m soft rock, hard rock
60	Hard rock
70	Hard rock

8. Retaining wall excavation not included. Retaining walls assumed to have a 10:1 sloping front face and be located immediately adjacent to the road (see Figures 11.1 and 11.2).
9. For retaining wall volume calculations, base of retaining wall assumed stepped when hard rock encountered.
10. Cut slope angle idealised and likely to vary with cut height. In mountainous terrain underlain by strong rock, cut slopes of 10:1 are likely to be feasible on 50° side slopes.
11. All fill slope angles assumed to be 1:1.5.
12. Cross section type:

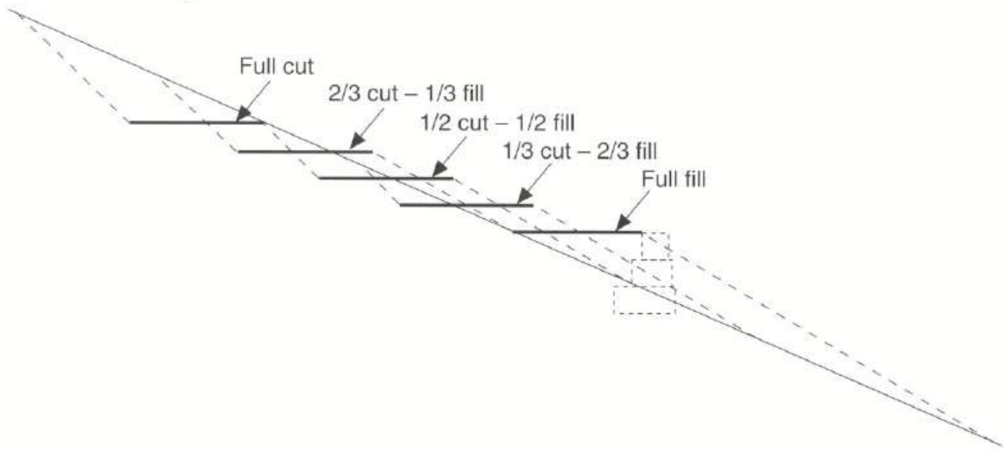


Table 4.2
Earthworks and retaining wall quantities according to cross-section type and slope angle

- the establishment of nurseries to provide plants for slope revegetation
- requirements to provide accommodation and provisioning for labour
- requirements for programming of site surveys following the provision of design information.

Contract administration

4.34 At an early stage of the project it is necessary to decide how to proceed with constructing the works. The main issues are:

- direct labour or contracted work
    - single large contract
    - multiple small contracts
- contract risk and claims
- labour intensive or mechanical methods of construction
- the extent to which allowance should be made for the development of design during the construction stage
- scheduling of construction contracts in relation to the wet season.

a) Direct labour versus contracted work

4.35 In the case of direct labour, the engineer, whether it be the government agency concerned or an appointed consultant, generally has greater control over methods of construction and more flexibility to make design changes. Modifications to design can be made during construction without incurring contractual claims. However, the larger the project is, the less likely it is that the client will be willing to undertake construction using his own resources, giving preference to engaging the services of contractors.

b) Contract risk and claims

4.36 Having decided to employ contractors, the most appropriate contract strategy should be identified. This will determine, for instance, whether the works will be let to a single contractor or to several contractors in a series of smaller contracts. Different forms of contract apportion the risks involved in a construction project differently between the contractor, the employer and, to some extent, the insurers. The amount of risk that a bidder is required to carry can have a considerable effect on his bid price. Given that the risks attached to engineering works in unstable mountain areas tend to be high, any action that can be taken by the employer or engineer to reduce the risks is likely to be reflected in a reduction in price. Problems that are likely to present significant risk for a contractor include:

- the reliability of the contractor's own estimate given the extreme nature of the terrain
- the remoteness of the site, particularly if the supplying sea port is in another country, with the consequent length of time required to import plant, spare parts and materials
- difficult access to and within the site area
- extremes of weather
- uncertainties over the supply of locally produced commodities such as fuel and cement
- geotechnical uncertainties over the extent of unstable ground and foundation conditions at the sites of major structures
- uncertainties over the availability of construction materials and haulage requirements
- severe time and operational constraints imposed under the contract
- the effectiveness of escalation clauses
- the scale of liquidation damages.

4.37 Some of these risks can be reduced or transferred from the contractor to the employer by the adoption of an appropriate form of contract. A representation of the allocation of total project risks apportioned to the employer and the contractor by using different forms of contract is given in Table 4.3.

4.38 In a price-based contract the contractor is required to carry the risks, and he will include a contingency in his bid price which the employer will pay whether or not the risk event transpires. The price may not bear much relation to the actual cost of construction. In a cost-based contract the

Table 4.3 Relative degrees of risk associated with different forms of contract

Employer's risk	Type of contract	Contractor's risk
0% ↓	Turnkey	100% ↑
	Lump sum fixed price	
	Lump sum: escalation	
	Admeasurement	
	Target	
	Cost-reimbursable	
100%	Directly employed labour	0%

employer pays the actual cost of constructing the works. While the employer may initially be alarmed by the prospect of carrying the risks, the clear advantage to him is that he only pays for risk events that actually occur. Cost-based forms of contract can cause difficulties for funding agencies, particularly in respect of the procurement of goods.

c) Labour-intensive methods *versus* mechanical methods of construction

4.39 In theory, it is possible to carry out most of the activities involved in road-building by manual methods, and in a labour-abundant and foreign capital-scarce economy, it makes economic and social sense to do so. However, mass haul of spoil and construction materials, compaction and road surfacing are usually best undertaken by mechanical means.

4.40 It is important to anticipate the extent to which labour-intensive construction will be necessary on a project. Such operations may require several thousands of labourers, both skilled and unskilled and, particularly in remote areas, it is likely that labour will have to be imported, possibly placing severe strains on local resources. Clear provision should be made under the contract for the supply of accommodation, fuel, water and sanitation facilities as well as the importation of goods, particularly foodstuffs.

4.41 By combining mechanical and labour-intensive methods, certain advantages can be gained. Operations such as the construction of retaining walls and cross-culverting can be undertaken using materials that can be carried in by porter, and can therefore be started in advance of the main earthworks. Operations such as earthworks and pavement works are better carried out by machine, working at an advancing road head. Designs suitable for porterage include the use of bolted structural steelwork for bridge substructures and superstructures, and corrugated metal piping for culverts.

d) Development of design during construction

4.42 It is usual, particularly in the case of conventional admeasure contracts, for the design to be well developed before contracts are let. However, when working in difficult ground conditions there may be good reason to develop the design as far as an alignment and standard details, and then complete the detailing of the final design during construction. Advantages include:

- the designer is more free to modify designs due to unforeseen ground conditions, as often arises
- the design can be completed in a better knowledge of the relative costs and performance of alternative forms of construction
- the designer can draw maximum benefit from the contractor's strengths.

e) Scheduling to take account of the wet season

4.43 The significance of the timing of letting construction contracts in relation to the wet season is often underestimated. For instance, the effective working season in the Low and Middle Himalayas is between eight and nine months long and it is important that the time available to the contractor should contain as many complete dry seasons as possible. Ideally, the letting of a construction contract should allow sufficient time for full mobilisation before the commencement of the dry season. The mobilisation period will vary according to the size of the contract, the location of the site and the home base location of the contractor, but for a major international contractor it could be as long as six months. At higher altitudes, the occurrence of frost and snow will also reduce rates of construction.

CONSTRUCTION ASPECTS

4.44 There are a number of construction issues which should be addressed during the design phase, because they affect issues of construction management and environment. They are:

- rock blasting methods
- construction materials
- the choice between bituminous surface treatment or gravel running surface
- rate of construction and construction sequence
- temporary access for construction (pilot tracks)
- quality control and supervision
- disposal of spoil.

Rock blasting

4.45 Uncontrolled rock blasting is atypical feature of road construction in Nepal and other developing countries where labour-intensive methods of excavation rely on a totally fragmented rock mass for easy removal. However, bulk blasting, as it is termed, usually results in significant overbreak and the creation of a highly fractured rock mass. The angle of the dressed slope is invariably lower than the optimum, and the slope usually weathers and ravels back to an even shallower angle within a short period of time. Pre-split (pre-shear) and cushion blasting remove smaller quantities of rock along predetermined planes of weakness so that overbreak is minimised and the remaining rock mass is left intact, and hence is more stable. However, these methods require a greater design expertise and a large number of closely-spaced drill holes to be accurately orientated.

Construction materials and water

4.46 The steepness of mountain slopes tends to limit opportunities for the accumulation of significant deposits of naturally occurring aggregates and gravels. River beds and river terraces offer a ready source of aggregate, although materials are often inconsistent and haulage uphill can represent a significant project cost.

4.47 For pavement design in areas where suitable materials are scarce, the following options should be considered:

- open a central quarry of good quality aggregate and budget for increased haulage costs
- blend materials to achieve a material that is within specification
- use materials that are outside the normal specification and plan to replace them more frequently
- if no source of surfacing aggregate is economically available, surface the road with gravel and maintain frequently.

4.48 Mica is present in a large proportion of the higher grade schists and gneisses of the Himalayas and to the natural aggregates derived from them. The presence of mica in fine aggregates can adversely affect the workability of concrete, and the addition of water to increase workability will only lead to a reduction in strength. The quantity of mica is controlled by careful selection of the source material. However, it is rarely possible to find naturally occurring fine aggregate completely free of mica and it is better, therefore, to avoid specifying the use of high-strength concretes. Where high strength concrete is a requirement, for example to bridge works and for culvert pipes, then fine aggregate can be manufactured by crushing rock.

4.49 The availability of water for construction purposes can become a major consideration if haulage uphill is required in the dry season to satisfy compaction requirements.

Gravel or sealed surface

4.50 The choice between a gravel and a sealed road surface is not simply one of traffic level. The general paucity of aggregates applies especially to good quality wearing-course gravels to such an extent that this may become an important factor in the choice between providing a road with a gravel or a bituminous surface. Good wearing course gravels may be difficult to obtain due to poor rock, deep soils, the difficulty of quarrying on steep slopes and long haulage distances from river beds. Wearing-course gravels require not only a good grading but also a limited plasticity, and it is often this last element that is difficult to fulfil. Without some plasticity, gravel wearing-courses erode more quickly, perhaps at a rate of up to 50mm per year, requiring replacement more frequently, thus aggravating the gravel supply problem still further.

4.51 There are other disadvantages to the use of gravel surfacing in mountains:

- the use of gravel surfacing, with its poor resistance to skidding, in conjunction with tortuous alignments is potentially dangerous. Also, vehicles cannot climb steep inclines on a loose surface
- gravel is easily dislodged by tyre scuffing, and washed away by rain. Rapid gravel loss leads to high maintenance costs, and attrition of the poorer gravels will generate large quantities of dust, posing an environmental and safety hazard.

4.52 One of the main advantages with gravel road construction relates to the fact that embankment fill can settle significantly in the early years after construction. Such movements can easily be accommodated by a gravel surface which can be replaced later by a more durable pavement.

Rate of construction and construction sequence

4.53 Actual rates of construction vary widely according to the relative proportion of labour and machine-intensive activities, volumes of earthworks, frequency and size of culverts and retaining walls, and the ability to construct pilot tracks (see below). Adopting a labour-based approach will provide easier access to remote workfronts and therefore the rate of construction will depend upon the size of the labour force that can be managed and sustained, and on the maximum rate at which construction can be supervised by the engineer, although factors such as availability of materials, spoil disposal and the contractor's working method are also relevant. Dry season labour-intensive construction rates might be of the order of 1km/month.

4.54 In the case of predominantly machine-intensive construction, the rate of progress of advancing workfronts might be between 2 and 3km/month, depending on formation width, access and contractor's working method. 0.5km/ month is probably the maximum progress likely to be achieved in the wet season. Attention to drainage and erosion control is crucial to any activities carried out in the wet season and it is usually advisable to limit most construction activities to the dry seasons and ensure that the works are adequately detailed and protected from erosion during the intervening wet season.

4.55 The recommended sequence of cross-section construction is shown in Figure 4.2.

Pilot tracks

4.56 The speed with which a road can be constructed or upgraded depends to a large extent on the number of work fronts that can be established. The provision of temporary construction access tracks can enable plant and materials to be supplied to remote work fronts and allow a faster rate of

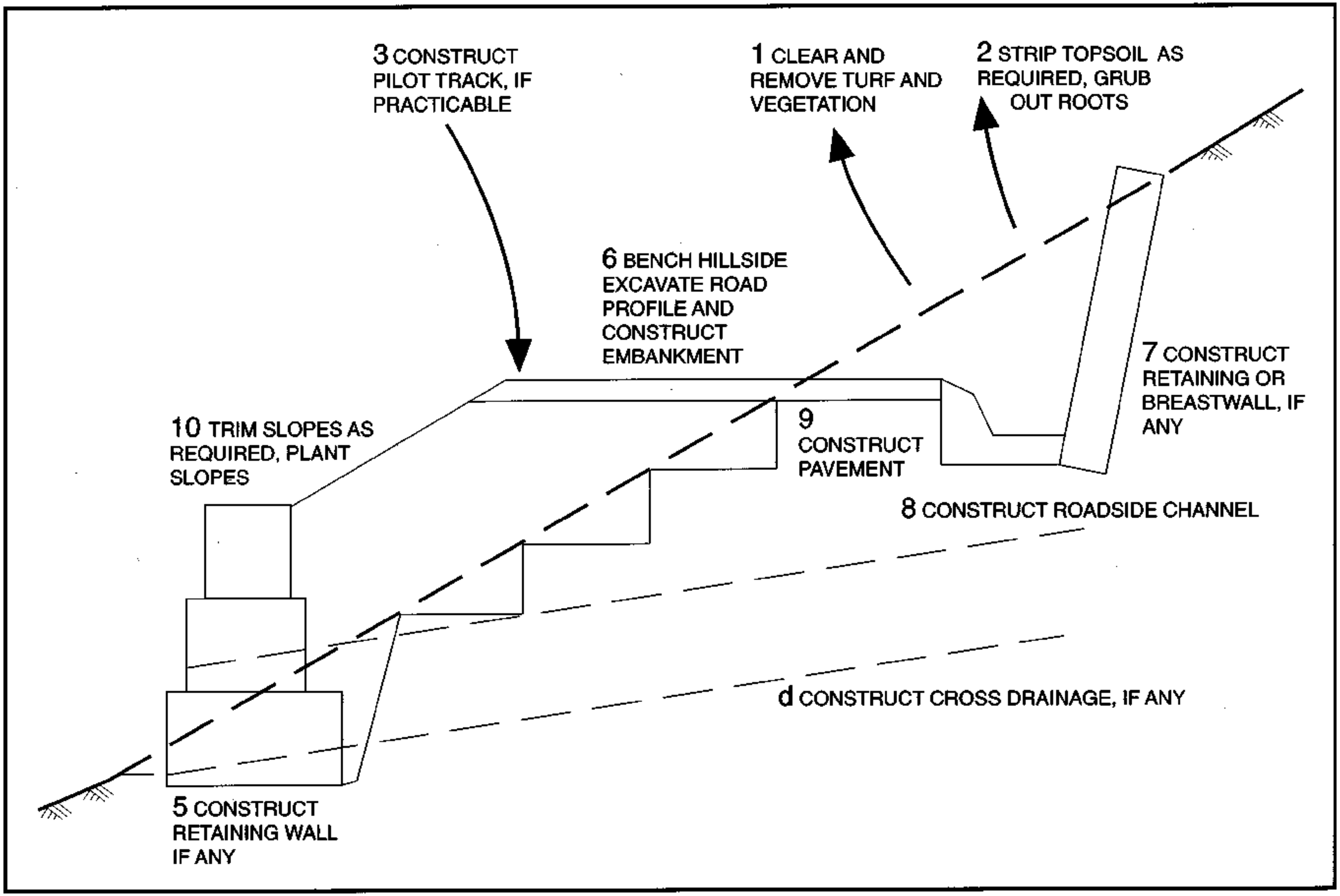


Figure 4.2 Typical sequence of cross-section construction

progress. However, the steepness of the terrain and its vulnerability to erosion often make it inadvisable or impossible to force through a motorable construction access track. Exceptions where advance access can be put in safely are:

- along the approximate centre-line of the alignment in advance of the main work front, provided that drainage and erosion protection are incorporated as soon as the track is opened up, and completed prior to the onset of the wet season. If this is not achieved, then very severe erosion can occur, proving both difficult and expensive to rectify

- on rock slopes where the potential for instability and erosion is judged to be low

- along valley floors. The location and permanency of valley floor pilot tracks during the construction period will depend upon flood levels during the wet season. Access tracks along valley floors require either temporary or permanent bridging of tributaries. The location of access tracks in such situations should be designed to minimise disturbance to cultivated land and neighbouring communities.

4.57 Where rapid construction progress is required, aerial support by helicopter or aircraft may be an alternative though expensive means of establishing remote work fronts and construction camps. The more remote these sites are, the longer will be the period of aerial support required to supply and maintain them.

Quality control and supervision

4.58 Although the general and special specifications may contain sufficient stipulations and safeguards to ensure, on paper at least, that construction will be carried out to required standards with minimum and controlled environmental impact, it is the task of the engineer to supervise and oversee that these standards are applied in practice. In the case of multiple work front construction especially, failure to adequately supervise all aspects of work can have severe consequences when the road comes under stress from rainstorms or floods. This is particularly true of `hidden' work such as the preparation of foundations, the compaction of soil and backfill, and the filling of gabion boxes and construction of masonry walls behind the front face, which if not carried out to specification, cart lead to failure of the whole structure.

Improvement and upgrading

4.59 In gentle terrain, there is often scope for upgrading the standard of a road by stage construction, if predicted growth in its use is uncertain, or if construction funding is limited. When the time for reconstruction is reached, its geometry and design can be reappraised. In mountains the possibilities for upgrading through widening or improvement to geometric standard are limited, as it is impracticable to make minor modifications to the geometry without having to undertake significant earthworks and retaining wall construction, involving high costs and the risk of reactivating slope instability and erosion.

4.60 Disregarding patching and re-sealing operations as being strictly maintenance works, road improvement and upgrading may consist of one or more of the following:

- drainage rehabilitation

- pavement overlay

- pavement reconstruction

- road widening and minor improvements to road geometry

- complete or partial reconstruction.

4.61 While access to the site will not be a problem on a road upgrading scheme, the fact that there is an existing road will mean that it will be necessary to accommodate the passage of public traffic throughout the construction period. In more gentle terrain this is normally achieved by providing diversion tracks, but in mountainous areas this is often impracticable, so that special provisions may be necessary to allow traffic to pass, which in turn places severe restrictions on the way some operations can be undertaken. For example, excavations for the installation of new culverts across the main carriageway cannot be undertaken as a single operation but will have to be performed in halves in order to keep the road open. Similarly, pavement layer construction cannot be undertaken as a full-width operation.

4.62 In developing countries, roads attract human settlement and in mountainous areas, where level ground is at a premium, homes are often built close to the roadside. This process is difficult to control and it is frequently difficult to gain access to sufficient width of land to carry out road widening. Also, land acquisition can be legally complicated and land prices may be inflated.

4.63 Geotechnical studies are made easier in existing road cuttings due to improved access for drilling rigs and the fact that soil horizons and geological strata are exposed. The assessment of foundation conditions for retaining walls, particularly those below the existing road, may be made significantly more difficult by the need to distinguish natural materials from construction spoil, a task that can be more difficult than it sounds.

4.64 It is often the case that earlier road construction will have exploited the most easily accessible sources of gravel and stone so that, unless the supplies are regularly replenished river deposits, good construction materials may be either difficult to locate or difficult to transport to site.

4.65 In steep ground there are often significant cost advantages in restricting road widening operations to one side only of the road cross-section. At any given point it is often better to encroach either wholly on the cut-side or wholly on the fill-side, particularly if retaining walls are involved. This will limit the extent of ground disturbance to one side of the road and it will make traffic control easier. However, this can make the achievement of a mass balance more difficult so that if borrow and spoil are problematic, then switching between cut and fill at regular intervals along the road may be necessary. If borrow is available, and the disposal of spoil is not a problem, the decision to achieve road widening predominantly by cut or by fill will depend upon slope conditions, the potential for cut slope instability and the stability of slopes below the road. It should be noted, however, that cut slopes often take a considerable time to weather back to a quasi-stable angle and further excavation is likely to trigger a new cycle of failure, erosion and readjustment. Although some cutting is usually unavoidable, a tendency towards fill rather than cut is preferable if foundation stability below the road permits.

Maintenance practice

4.66 Although this note does not deal specifically with road maintenance, it is important in any road scheme to plan for sensible and timely maintenance in order to avoid creating unstable situations through either neglect or ill-advised maintenance practice. Some of the more common maintenance practices that create or aggravate instability problems are listed below:

- uncontrolled tipping of spoil materials arising from cut slope failures. This causes erosion and siltation below the road

- diversion of road drainage water around slip material on the carriageway causes uncontrolled runoff over the road edge

- failure to repair eroded side drain inverts leads to deepening of the side drain and erosion of adjacent materials

- failure to clear drains of debris leads to overtopping of the drain and erosion of adjacent slopes

- quarrying into cut slopes for road maintenance materials, and inadequate quarry restoration, can create slope instability and erosion problems

- often, where the slope below is eroding and undermining the road, a hillside is progressively cut back in order to maintain passage. This de-stabilises the hillside. (It is usually always advisable to address the problem below the road as quickly as possible in order to prevent an unmanageable slope situation developing, and to avoid further cutting of the hillside above)

- if, through lack of funds or insufficient time in advance of a wet season, it is not possible to implement pavement protection or stabilisation works, then a drainage or slope problem can usually be kept under control by constructing temporary works

- proper design of walls and drainage works and good construction supervision are just as important during maintenance as during the initial construction period

- removal of construction materials (especially theft of stone from gabion boxes) and disturbance to road drainage systems in village areas and irrigated farmland by diversion of the water are problems that require rectification and monitoring.

5 DESK STUDY AND RECONNAISSANCE SURVEY

INTRODUCTION

5.1 Desk study and reconnaissance survey are fundamental to the success of any feasibility study. The availability of desk study information, together with its scale and level of detail, will determine the extent to which desk studies can advance feasibility studies prior to embarking on detailed fieldwork. The importance of good preparation in a thorough desk study cannot be over-emphasised, although its scope will depend on the availability and quality of maps, aerial photographs and other published materials. Where sufficient data exist, the desk study can be a time and labour-saving process that allows the identification and analysis of options for route corridors and design concepts, and serves to focus attention on the most relevant factors to be considered during site reconnaissance. The more comprehensive the desk study is, the fewer problems may be expected later throughout the duration of the project.

DATA SOURCES

5.2 The desk study comprises a review of published and unpublished information concerning the physical, economic and environmental characteristics of a study area. Although office-based, it is usual to combine the desk study with reconnaissance survey in order to reduce the uncertainties arising out of interpretations of often rather scanty published data.

5.3 Because of the difficulty of access and the frequent historical lack of economic interest in tropical mountains, background information about them tends to be sketchy and often inaccurate. At best, it may be limited to small scale topographical and geological mapping, aerial photography and satellite imagery. At worst, none of these data sources may be available. Some countries, in which tropical mountain ranges occur, do not keep stocks of maps and publications about these remote areas, and it may be easier to obtain information from sources in Europe and North America than in the countries themselves.

5.4 Desk studies are usually based on the following data sources:

- published literature covering a range of topics including road construction and maintenance case histories and geological, economic and environmental reviews. The scientific literature is often a good source of information, though the areas studied tend to be small
- topographical maps
- geological maps, agricultural soil maps and other natural resource maps
- remotely sensed imagery - aerial photography and satellite imagery
- climatological records (Chapter 8).

MAPS

Topographical maps

5.5 Topographical maps are usually obtainable from the local survey department, or sometimes from specialist stockists in Europe. Their accuracy is sometimes questionable, and the scale may be too small to allow other than an orientation in the field. Contour heights may not be reliable, and maps may be out of date in respect of settlements and features that are prone to change, such as river courses. Their main advantage is in their consistent scale and clarity of representation, which enables areas, lengths and directions to be measured for purposes of feasibility study with a reasonable degree of accuracy.

Geological maps

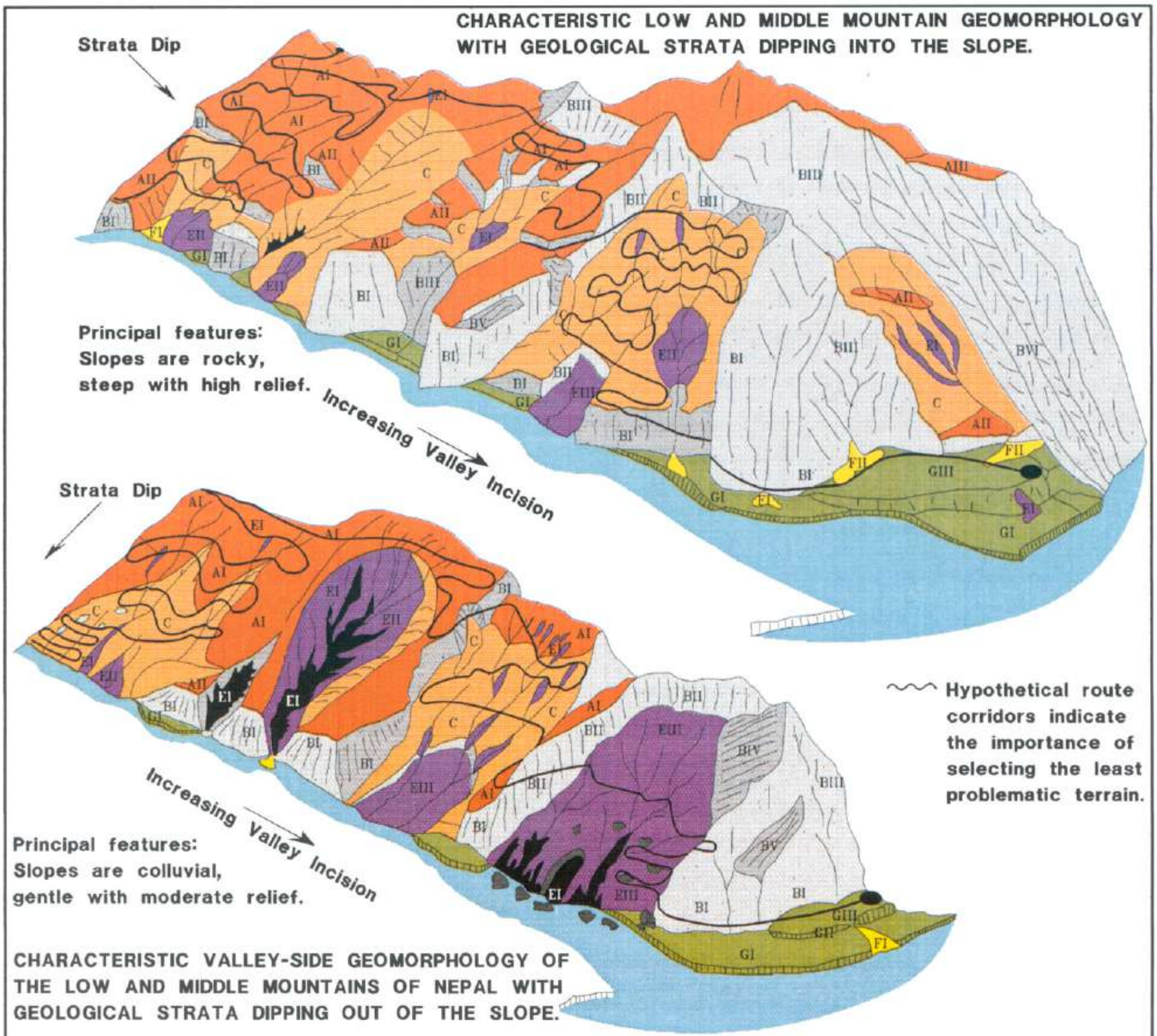
5.6 Geological maps at small scale are the most common type of thematic map found in both developed and developing countries. Most countries have a national geological map at 1:1,000,000 scale or smaller, although few have complete or even substantial cover at large scale. The main use of small scale geological maps is to provide both an overview of the broad distribution of rock types for construction purposes, and a background to interpretations of other forms of data, such as topography, terrain classification and drainage. Geological maps often do not show surface deposits overlying the solid bedrock, although major features such as river terraces and sand banks may be shown.

Soils maps

5.7 Soils maps are usually produced only for areas of potential agricultural interest, so their availability in mountainous areas is patchy. Soil mapping is useful to verify or extend the information given in geological maps, with emphasis on materials occurring within the top one to two metres. Agricultural soil classifications can contain elements that are of interest to engineering and it is worth reading carefully the descriptions of the soil units to identify criteria of engineering significance.

Terrain classification maps

5.8 Terrain classification maps have been compiled for various parts of the world as general-purpose planning tools. They are based upon the classification of landform combined with soils, drainage characteristics and parent materials. The landform component can be a detailed geomorphological interpretation, or more loosely defined in terms of general topographical properties (slope angle, slope shape, etc) and appearance in aerial photographs (air



LANDFORM TYPE	LANDFORM SUB-TYPE	
A Rounded - flat, residual soil slope, either ridge or spur top or mid - slope bench. Expect erosion rather than slope failure. Soil depths will range from 1m to 10m on weathered rock.	AI	Rounded ridge and spur summits, and gently sloping ground with residual soil cover.
	AII	Mid slope benches with residual soil cover.
	AIII	Structurally controlled dip slopes with residual soil cover.
B Rock slopes	BI	Miscellaneous rock cliffs, either structurally controlled or steep lower valley sides (greater than 45 degrees).
	BII	Rock cliffs formed along margins or in scars of mass movement.
	BIII	Irregular steep slopes (up to 45 degrees).
	BIV	Bedding, foliation or joint plane surfaces usually exposed as slip surface of deep seated failure.
	BV	Structural bench.
	BVI	Structurally controlled ridge and ravine topography.
C Slopes covered by colluvium but generally stable	C	Colluvium and weathered rock fall deposits.
E Areas of unstable ground in surface soils or deep-seated ancient slope failures.	EI	Mudslides, debris slides and rockslides on steep and frequently wet lower valley slopes.
	EII	Mudslides and debris slides associated with local erosion and soil saturation.
	EIII	Large, deep-seated slope failures usually with adverse structural control.
F Fan deposits	FI	Active fan surfaces, usually on flood plains.
	FII	High level fan deposits, usually associated with high level terraces or deposited by catastrophic landslides.
G Terrace gravels	GI	Flood plain terrace (periodically inundated).
	GII	Terrace flank.
	GIII	High level terrace.

Figure 5.1 Terrain classification for corridor selection purposes in east Nepal

photo pattern). The main advantages of this kind of mapping are its ability to summarise large areas of terrain and its applicability to a range of land use and development projects.

5.9 Published terrain classification has been used very effectively in geologically old landscapes with relatively dry climates, where the drainage and material assemblages have become ordered into clear patterns of terrain units. Its usefulness in active fold mountains, where ground conditions change rapidly and unpredictably over short distances, is much diminished. Pre-existing terrain classification maps are therefore unlikely to be available for use in route feasibility studies in these areas.

5.10 The objective of any route alignment feasibility study is to collect and analyse sufficient data to enable the broad corridor details, design concept and approximate cost associated with the preferred option to be identified, without recourse to costly and time-consuming field investigation over large areas. Therefore, if terrain classification can be carried out cheaply and reliably, it will prove useful in gaining an understanding of the terrain from which to plan a strategy and recording system for later reconnaissance surveys. The classification system adopted, and its extrapolation over the area of interest, should be based on the interpretation of aerial photographs combined with topographical maps and ground verification. The sophistication of the terrain classification system devised should be sufficient to allow route corridor alternatives to be identified and divided into manageable units within which terrain and engineering parameters are considered likely to remain approximately constant. These terrain units could be based on a simple combination of the terrain zones given in Figure 2.4 with a four-fold slope angle classification, and further differentiation between soil and rock slope categories. Alternatively, they could be based on a more sophisticated terrain classification system, such as that illustrated in Figure 5.1. Under the usual constraints of limited time and lack of sub-surface information, the simpler this system can be made the better.

5.11 The design of the classification will normally depend upon:

- the geographical extent and reliability of available data sources required to carry out the classification, and the time and budget available to derive primary data, often over large areas, if existing data sources are insufficient
- the extent to which recognisable landforms are repeated in the landscape, and the confidence with which assumptions regarding materials, processes, drainage and stability can be applied to them.

5.12 A classification could be drawn up with the following points in mind:

- develop a classification that is not too rigorous - the simpler the better
- identify and record actual landforms, materials and processes by preliminary walkover and then devise a classification scheme that reflects these
- avoid using a classification scheme that has been produced previously for another terrain type. Mould the classification to apply strictly to the area under study, even to the extent of using more than one classification in a region where strongly-contrasting terrain groups exist
- abandon the hierarchical concept, upon which most terrain classifications are based, unless there is strong evidence that one does exist, and that the concept will be of value for design.

5.13 Table 5.1 describes some of the common landforms found in mountain terrain and illustrates the fact that contrasting processes and materials can give rise to the same landform type, thus reinforcing the need for ground verification.

AERIAL PHOTOGRAPHY

- Aerial photograph interpretation (commonly shortened to air photo interpretation or abbreviated API) is not only a highly important technique for terrain classification and desk study evaluation of topography and ground conditions, but it also provides vital data for many other analytical and design activities, including:
- landslide hazard mapping
- route corridor identification and selection
- drainage assessment and river catchment mapping
- identification of potential bridge sites
- identification of potential materials
- photogrammetry for contour mapping
- land use classification
- mapping of buildings and property for land acquisition
- environmental impact assessment and environmental monitoring
- construction planning.

5.15 However, the view of the ground can be obscured completely by forest, cloud cover or shade, especially in steep and irregular terrain. Under these conditions, recourse to ground survey is usually the only viable option.

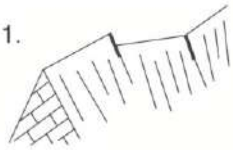
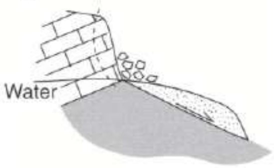
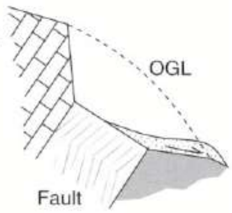
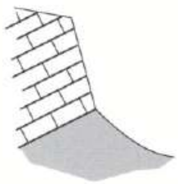
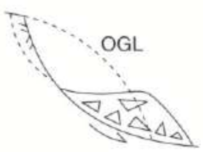
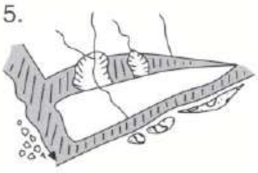
LANDFORM	ORIGIN AND STRUCTURE	DESIGN IMPLICATIONS
Steep-sided knife-edged ridge (Zone 2)		
1. 	1. Very strong rocks cropping out along ridge-line. or	Sound rock for excavation and road retaining wall foundation. Very steep slopes.
2. 	2. Deep-seated rock failures with back scars forming ridge crest.	Do not unduly undercut dip or joint planes. Road retaining wall preferable.
Linear and Regular Cliff-Line in Rock (Zone 2, 3, 4)		
1. 	1. Cliff retreat: contact between strong rocks forming cliff and weaker rocks forming eroding and unstable low ground. or	Possible rock falls, topples and cambers from cliff face. If aligning at the top of the cliff allow clearance for stress relief cracks. If aligning at the toe of the cliff allow clearance for rock fall. At foot, talus deposits overlie weaker lithologies with possible undrained loading effects.
2. 	2. Differential erosion: fault structure with faulted materials forming low ground below. or	Cliff probably stable, though check for location and orientation of faults or associated structures in cliff. Erosion and instability in faulted materials.
3. 	3. Differential erosion: contact between massive (unjointed) rocks forming cliff and closely jointed rocks forming low ground below. or	Cliff probably stable. Expect instability in weaker materials.
4. 	4. Back scar of rock slide. or	Back scar stable if in competent rock. Check for tension cracks at cliff top.
5. 	5. Ancient flank to river terrace.	Probably weathered with some stress relief cracks, deep drainage ravines and weathered failure scars.
	Refer to Figure 2.4 for Zone Classification	OGL – Original Ground Level

Table 5.1.
Landforms in the Nepal Himalaya and their implications for road design

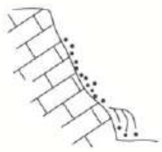
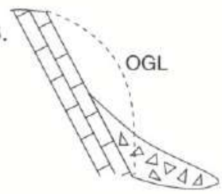
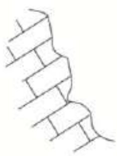
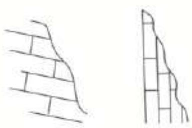
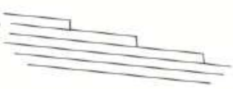
LANDFORM	ORIGIN AND STRUCTURE	DESIGN IMPLICATIONS
Steep uniform slope (Zone 3, 4)		
1. 	1. Dip or joint-controlled surface. or	Potential failure along discontinuity planes. Some excavation and good foundations possible on strong rocks. Avoid deep excavations in weak or weathered rocks.
2. 	2. Progressive ravelling/rock fall to equilibrium slope above river or in back scar of active landslide where slope is being periodically undercut. or	Avoid wherever possible. Dip or joint surfaces may be only locally significant. Road retaining wall rather than cut, though check for stability below.
3. 	3. Back scar of deep-seated joint-controlled rock slide.	Avoid wherever possible. Rock may withstand only small unsupported cut. Road retaining wall may be best option.
Steep irregular slope (Zone 3, 4)		
1. 	1. No structural control. or	Irregular topography. Steep cuts possible, good foundation for road retaining wall.
2. 	2. Dip is out of slope but at much too shallow or steep an angle to control topography.	Irregular topography. Possible failures in cut along steeply dipping joint planes, though any failures will probably be small. Good foundation for road retaining wall.
Benched or stepped slope (Zone 3, 4)		
1. 	1. Cyclical downcutting and pause in drainage incision. or	Stable. Deep soils and terrace deposits likely on benches.
2. 	2. Horizontally-near horizontally dipping strata or strata dipping gently out of hillside.	Good excavation and foundation conditions.

Table 5.1
Landforms in the Nepal Himalaya and their implications for road design (continued)

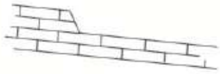
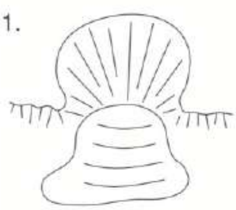
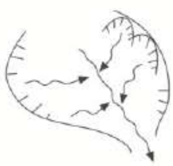
LANDFORM	ORIGIN AND STRUCTURE	DESIGN IMPLICATIONS
Mid-slope bench (Zone 3, 4)		
1. 	1. Gently sloping persistent joint or dip of strata exerts structural control.	Shallow residual soils, strong rocks exposed in cuttings, stable slopes and foundations.
	or	
2. 	2. Flat surface of rotated or slipped block. Should be accompanied by steep back scar with irregular or convex slopes below. Rare.	Avoid if possible or cross back scar with road retaining wall on solid rock foundation.
	or	
3. 	3. Pause in valley incision.	Residual soils. Stable slopes.
Deep semi-circular depression in hillside (Zone 3, 4)		
1. 	1. Back scar of deep-seated rotational slip (relative stability can be determined from extent of bare ground in back scar and presence of disturbed ground in slipped mass).	Full cut or road retaining wall across back scar. Avoid loading slipped mass or excavating toe.
	or	
2. 	2. Erosion bowl (relative stability can be determined from extent of bare ground and freshness of erosion features).	Cross gentler slopes on embankment with good cross-drainage for run-off and debris. Cutting will make problem worse.
	or	
3. 	3. Ancient meander bend associated with higher river base level.	Rear cliff may contain old slope failures and be jointed through stress relief, cross on retaining wall if unavoidable.

Table 5.1
Landforms in the Nepal Himalaya and their implications for road design (continued)

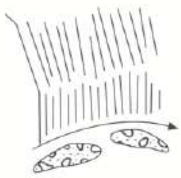
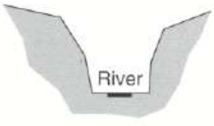
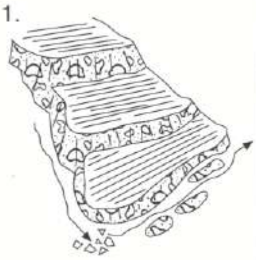
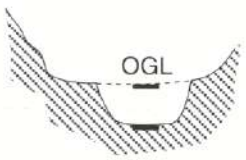
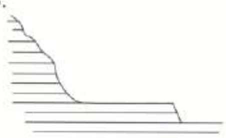
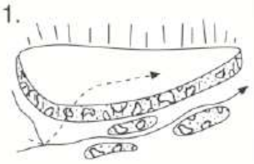
LANDFORM	ORIGIN AND STRUCTURE	DESIGN IMPLICATIONS
Steep lower valley side slope (Zone 4)		
1. 	1. Undercutting and slope steepening on outside of valley meander bend. or	Possible site of toe erosion and further instability. Avoid or cross in full cut. Adverse dip or joint orientations will be problematic.
2. 	2. Recent/current phase of valley incision.	Ditto.
Lower slope bench (Zone 4)		
1. 	1. Higher level river terraces or fan surfaces. or	Surface should be stable. Stability of slopes above and below and the configuration of drainage crossings may control detailed location of alignment.
2. 	2. Erosion surface in rock or soil associated with ancient higher river level. or	Ditto except for drainage crossings.
3. 	3. Structurally-controlled bench.	Ditto except for drainage crossings.
Uncultivated flat land (Zone 4,5)		
1. 	1. Flood plain or flood plain terrace. or	Avoid, or expect regular inundation and high road maintenance costs.
2. 	2. Gently sloping bedrock surface.	Suitable for road embankment. Strong rock in excavation.

Table 5.1.
Landforms in the Nepal Himalaya and their implications for road design (continued)

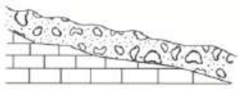
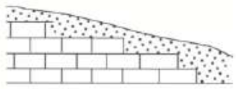
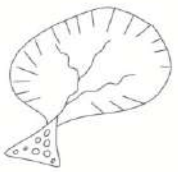
LANDFORM	ORIGIN AND STRUCTURE	DESIGN IMPLICATIONS
Gently to moderately sloping ground (Zone 3, occasionally 4)		
1. 	1. Weathered and reworked colluvium.	Potential for shallow failure in excavations.
	or	
2. 	2. Residual soil slope.	Potential for erosion in excavations.
Cone-shaped slope surrounded by steeper ground (Zone 2, 4, 5)		
1. 	1. Steep: scree or talus deposit from cliffs above (stability indicated by extent of vegetation or cultivation).	Cross in cut/fill section or road retaining wall. May need cut slope retaining or revêtment wall.
	or	
2. 	2. Gently sloping: fan from eroding slope or catchment (stability indicated by extent of vegetation or cultivation).	a) Active. Cross at fan apex. b) Stable. Cross on embankment as high as is convenient. Match culvert and bridge design and location to debris load and flow pattern.

Table 5.1.
Landforms in the Nepal Himalaya and their implications for road design (continued)

5.16 Air photo interpretation should always be undertaken by a person who is familiar with the landforms, processes and materials likely to be encountered. As this familiarity can only be gained properly by reconnaissance survey, air photo interpretation and ground survey should form an iterative process. Some of the possible misrepresentations arising from air photo interpretation without ground verification are illustrated in Table 5.1. An example of air photo interpretation undertaken for route alignment in Nepal is shown in Figure 5.3 (pages 41 and 42).

5.17 A factor to recognise in hilly and mountainous terrain is the exaggeration of relief when viewing aerial photographs stereoscopically. Relief exaggeration causes slopes within the stereoscopic model to appear much steeper than they really are. The greater the exaggeration of relief, the more difficult it is to make an accurate estimate of the true slope angle. The degree of relief exaggeration is governed by the distance, or air base, between successive photographs, in relation to the flying height. Doubling the air base doubles the relief in the stereoscopic model (Figure 5.4. The air base in the upper stereoscopic pair is half that in the lower pair). In extreme cases, the image of a mountainside can appear so steep that it is impossible to make any effective interpretation of ground conditions. By contrast, low relief features such as river terraces, fans and landslide deposits are more readily identifiable by the exaggerated relief.

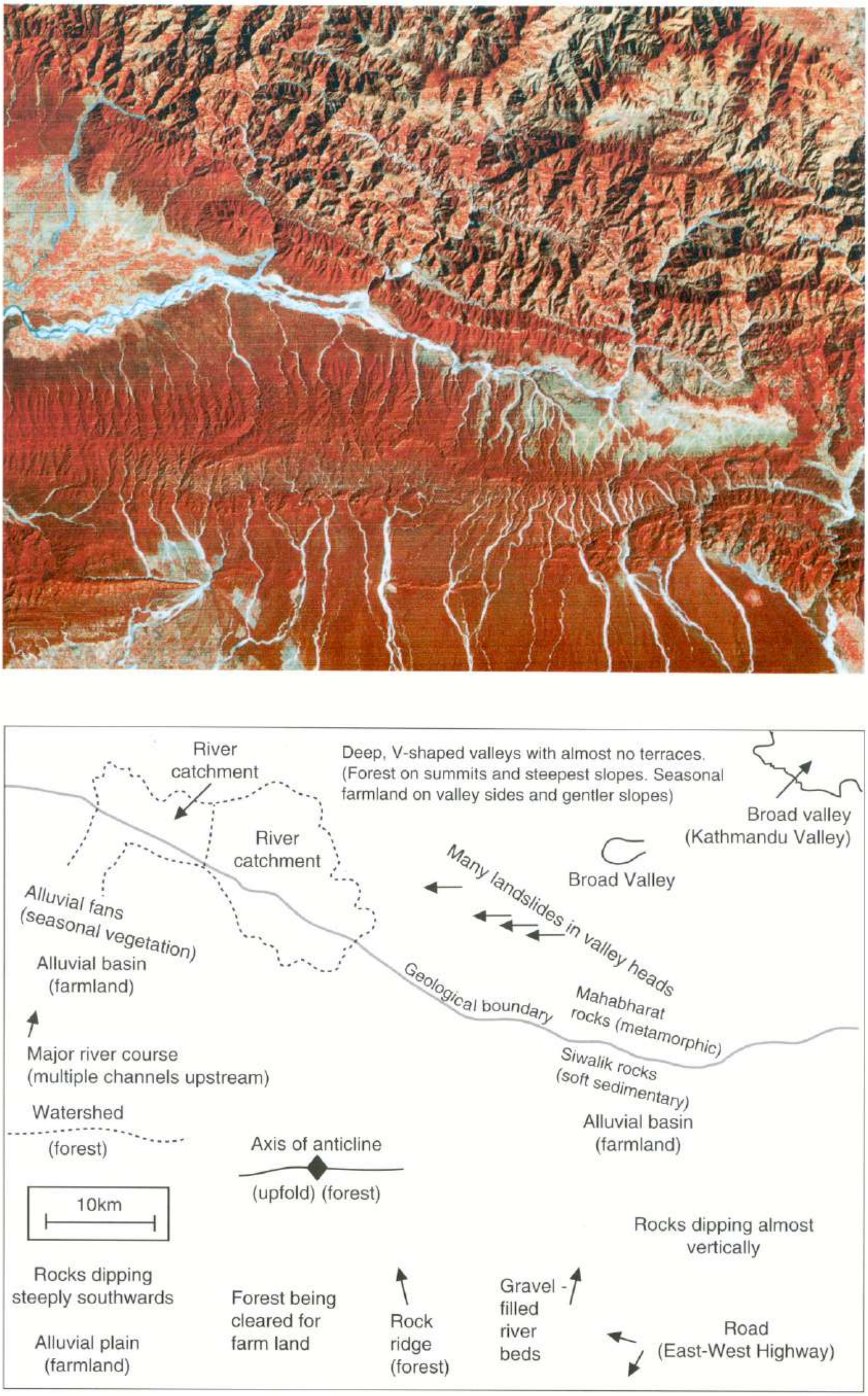


Figure 5.2 Landsat satellite image of part of central Nepal

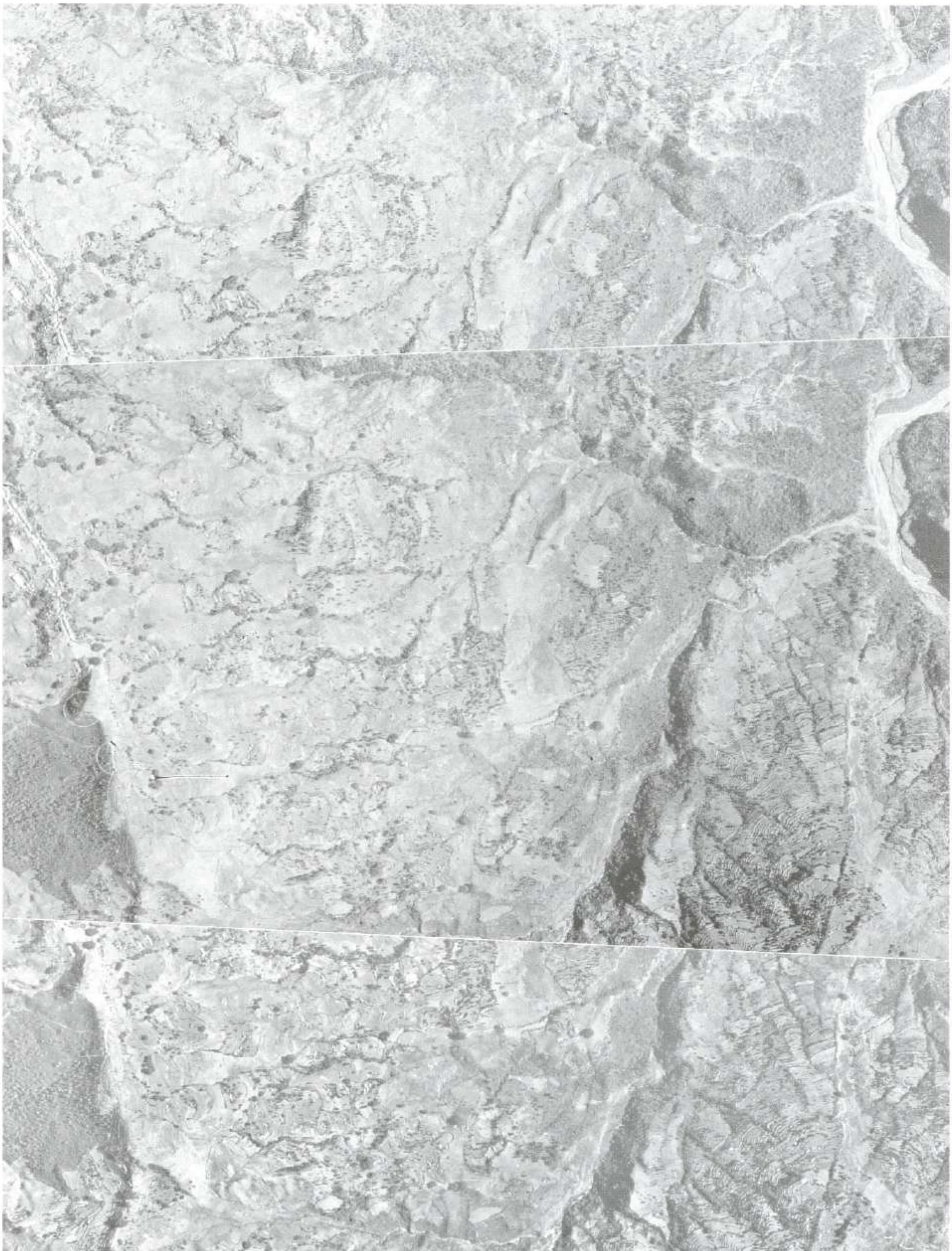


**Figure 5.3 Aerial photograph interpretation for road alignment purpose-
Steroscopic pair of aerial photographs**

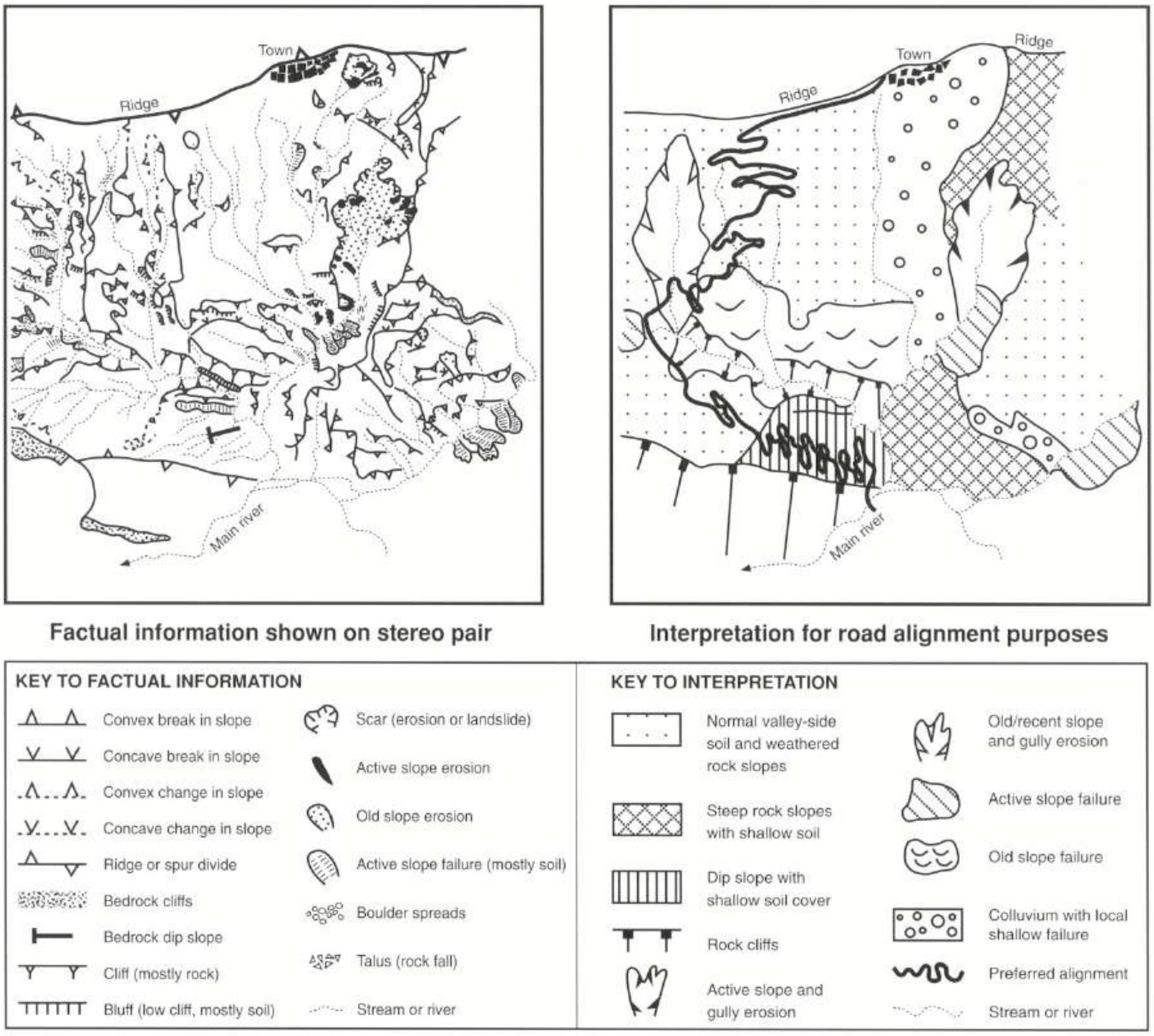


Figure 5.3 Aerial photograph interpretation for road alignment purposes (continued) - map and legend

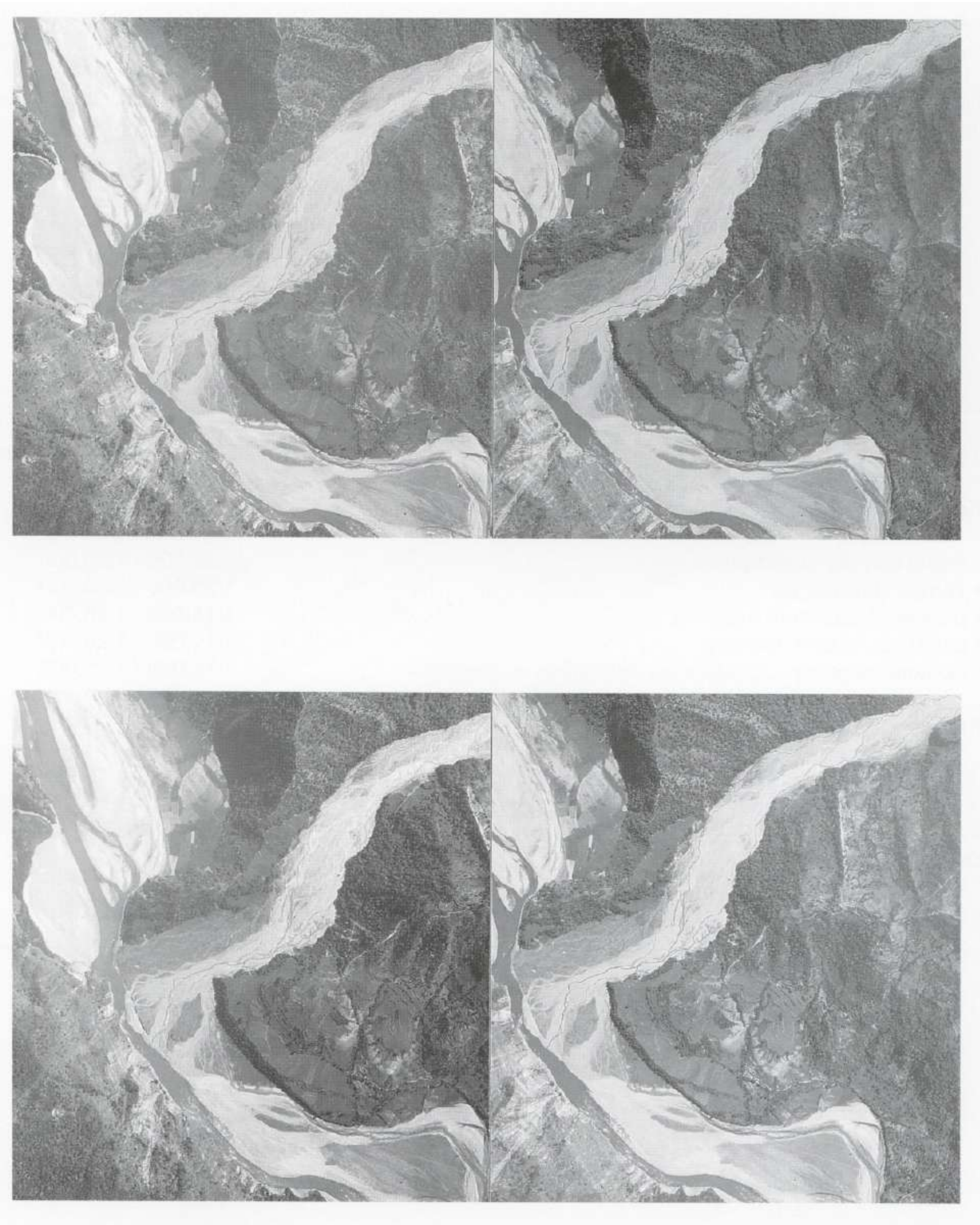


Figure 5 4 Relief exaggeration through increased air base between successive photographs

5.18 For those in a position to commission their own, it may be worthwhile taking photography with an 80% photograph overlap in order to be able to select an appropriate air base for the amount of exaggeration required. For interpretation in areas of high relief the viewer would use every photograph in the run (80% overlap, giving a short air base and minimum relief exaggeration). For areas of low relief the viewer could select every other photograph, to double the relief exaggeration. In extreme circumstances of very flat ground the viewer could select every third or even every fourth photograph, doubling the relief exaggeration each time.

5.19 The scale of photography is also an important factor to consider in the reliability and ground resolution of the interpretation. Table 5.2 indicates the optimum scales of photography required to perform various desk study and design tasks.

5.20 Table 5.3 summarises topographical and geomorphological indicators relevant to the identification of geological, stability and drainage features on aerial photographs. These guidelines are only indicative and each site will require its own evaluation of factors and processes.

Table 5.2 Air photo scales for various project tasks

Task Activity	Optimum Air Photo Scale
Feasibility Study:	
• Route corridor identification	1:20,000 - 1:30,000
• Terrain classification	1:20,000 - 1:30,000
• Drainage/catchment mapping	1:15,000 - 1:25,000
• Landslide hazard mapping	1:10,000 - 1:20,000
• Contour mapping for preliminary estimation of quantities	1:15,000 - 1:25,000
Preliminary Design:	
• Detailed interpretation of chosen corridor(s) for geotechnical purposes	1:10,000 - 1:15,000
• Ground (contour) model for preliminary alignment design and quantities	1:10,000 - 1:15,000
Detailed Design:	
• Ground (contour) model for detailed alignment design and quantities	1:5,000 - 1:10,000

Table 5.3 Appearance of terrain features in aerial photographs

Feature	Appearance in aerial photographs
SOIL *Depth*	Shallow: High percentage of rock outcrop and marked structural control to slope morphology; slopes steeper than 40° are almost certainly to be formed in rock. Deep: Concavo-convex slope profile with lobate and gently rounded lower slopes. Dendritic drainage pattern. Presence of visible large boulders.
Type Residual soil	Red and reddish-brown colour on colour aerial photos, commonly occupying rounded ridge and spur summits and flat or gently sloping benches. Often intensely cultivated. Can become severely eroded on sloping ground where vegetation is removed.
Rockfall/ Rockslide colluvium	Deposits of boulders below rock cliffs due to rock fall or rock slide. Range in size from small isolated deposits to deposits extending over large areas. Identification: rock back scar or source area, chaotic jumble of boulders, uneven slope, immature drainage pattern with jungle and patchy cultivation.
Undifferentiated colluvium (transported weathered slope materials	Gradual slope failures and shallow creep give rise to more uniform slopes at limiting stability, often with larger boulders towards the toe. Usually intensely cultivated, with shallow failure depressions, immature drainage systems and often wet ground at the toe. Lower slope may be regularly or permanently unstable.
GEOLOGY *Rock outcrop*	Steep slopes, structural control on topography, high light reflectance from bare rock surfaces, repeated pattern of structural surfaces, cliffs and waterfalls.
Strong rock	Steep and rugged topography with steep-sided V-shaped gullies and valleys, knife-edge ridges, cliffs and fresh structural control to landscape. Rockfalls; rockslides.
Weak rock	Gentle slopes, rounded spurs and ridges, low relative relief, no visible rock outcrop, concave slopes, shallow slope failures (soil). Less marked or no structural control on topography.
Rock Type	
Mudstone, shale	Slopes as for weak rocks; immature and shallowly incised drainage system, surface ponding and wet, heavy soils, frequent mudslides.
Limestone	Small, relatively deep depressions on ridge tops and structural benches, internal drainage and lack of regular surface drainage, discontinuous ridges, castellated tor-like high points along ridges, outcrop in structurally-controlled cliffs, often dip slope/scarp slope landforms, frequently forested, poor soils reflected in sparse cultivation.
Quartzite	Narrow, jagged, irregular cliff-lines often with little structural control. Thin soils, sparse vegetation and high reflectance from rock surfaces.
Phyllite/Schist	Intermediate between soft rock and hard rock landscape and landforms. Linear structures are often evident due to a pronounced foliation.
Gneiss	Low *elevations:* subdued and weathered landforms with frequently deep residual soils. *High elevations:* steeper, rocky landscapes with high relative relief and structural control over topography.

Table 5.3 Appearance of terrain features in aerial photographs (continued)

Table 5.3 Appearance of terrain features in aerial photographs (continued)

Feature	Appearance in aerial photographs
Linear structures (Dip/joint orientations, faults, etc)	Lineations of ridges, cliffs and streams, often crossing other topographical features of different orientations. Fault lines are often areas of concentrated erosion and instability. Deep gullies may form where faults run downslope. Lithological (rock type) boundaries may be marked by a fault.
Dip slopes Hard rock	Planar and regular slope surfaces tending to occupy large portions of valley sides in steep terrain. Topographical lineations are parallel to strike. Streams are straight, often approximately equi-spaced and shallowly incised into bedrock.
Soft rock	Roughly planar surfaces showing translational slope failures, shallow failure deposits and deeper drainage lines.
Scarp slopes (Dip/foliation into hillside)	Slope profile is steep and irregular with short cliffs and benches. Gently-dipping strata give a stepped slope profile in steep terrain. Hard rock strata form ribs across a slope, parallel to strike, frequently with trellis drainage pattern.
SLOPE FAILURE *Active*	Fresh failure scar and slipped mass. High reflectance from unvegetated back scar. Landform uncharacteristic of surrounding topography. Rock spalls around margin of deep failure. Immature vegetation or bare ground in slipped mass. Springs, ponds and wet ground at toe and in slipped mass. Slope tension and compression (hummocky profile) within slide area. Toe of failure protrudes into river flow; river erosion removing toe. Lack of cultivation or disturbed cultivation/irrigation pattern.
Old	Subdued/weathered landform described above, mature vegetation, established cultivation.
Mechanism	
Progressive soil creep	Immature/uncharacteristic vegetation. Disturbed/hummocky ground surface; micro-relief ridges perpendicular to direction of movement. Discontinuous/uneven irrigation and cultivation pattern.
Mudslides (translational failures in fine- grained soils)	Slow moving ground with distinct lateral boundaries and shallow, narrow, planar track. Vegetation/cultivation is often maintained though disturbed in pattern. Usually on slopes of less than 28°. Weathered rock or fresh soil exposed in back scar. Toe may be lobate. Usually on lower valley slopes where groundwater is higher. Failing slope angle often similar to adjacent unfailed slopes.
Debris slide (translational slides in coarse- grained soils)	Debris slides usually occur as an initial sudden failure with or without irregular movements thereafter. Well-defined back scars and side scars and an irregular slope surface. Vegetation is often destroyed. Usually on slopes between 30 and 40° in angle. High reflectance from back scar and slip material if recent. Boulders may be identifiable in deposit. Failed slope or debris cone usually shallower in angle than adjacent unfailed slope. Springs often emerge from base of back scar and along toe of slipped debris.

Feature	Appearance in aerial photographs
Debris flow (high velocity flows of debris and mud)	Source area of slope failure or erosion on steep ground and depositional track or fan of debris on concave slope below. Common in unstable and eroding catchments.
Progressive rock creep ('sackung')	Mostly in fractured rock masses occupying high, steep slopes. Difficult to identify and confirm on aerial photographs. Ridges and trenches orientated normal to direction of maximum slope may form as a result of plastic flow within rock mass. Active creep causes rock spalls on margins of failure zone.
Rock fall	Rock fall scar (topple, wedge or plane) and a talus deposit below. Progressive rock fall gives slightly concave, uniform, talus slope of 33-38°; larger particles accumulate towards toe.
Rock slide: Rapid (instantaneous)	Distinct margins of failure with cliffs. Planar failure surface may be exposed below the back scar with higher reflectance if recent. May be seepage, surface ponding and irregular slope drainage pattern within slipped mass. Failed mass of chaotic boulders and rafts of rock. May be a single block or a number of blocks, stability of each controlled by movements of block below. Slope angle of failed debris significantly less than that of adjacent unfailed slopes. Large valley-side failures may partially block river below or cover river terraces. Large blocks may persist in river.
Rock slide: Slow moving (progressive)	Hummocky and furrowed slopes in head of failure with or without compression ridges in toe. Slope rupture and small vertical displacement along margins.
Rotational slides: Soil	Arcuate back scar in plan. Concavo-convex slope profile in section from back scar to lobe at toe of failure. Well defined slip margins. Debris toe forms a spreading lobe on slope below. Rotated slipped mass may form a reverse slope below back scar, with possible ponds. Toe lobe may be eroded due to ground water released during failure or adjacent river scour. Lobate form of failed mass may create anomalous drainage paths diverging down slope along margins of slip. Springs may continue to emerge from base of back scar. Multiple back scars indicate either progressive failure upslope above original back scar or secondary failures within original slipped mass. Second case may give stepped slope profile exploited as farm terraces.

Table 5.3 Appearance of terrain features in aerial photographs (continued)

Feature	Appearance in aerial photographs
Rotational slides: Rock	As for soil failures in principle, although: Slip margins may comprise a number of segments that approximate to arcuate form. Surface of failed mass usually of boulders. Irregular profile, possibly less lobate, probably no surface drainage; Secondary failure unlikely (unless undercut by river erosion below). The reduction in slope between the back scar and the failed debris is usually much more marked in the case of rock failures. Comments above for differentiation of rapid and slow moving rock slides apply to rotational slides.
Rock avalanche (catastrophic rock fall)	Original rock fall scar may not be apparent due to subsequent weathering. Failed mass is long, thin tongue of boulders on slope below, in valley floor or even high on opposite valley side. Flow lines and pressure ridges parallel to flow. Rock avalanche may alter drainage pattern. Failure volumes up to 100 million m3 can occur.
DRAINAGE *Slope Drainage* Sheet wash	 Generally not identifiable on aerial photos.
Channel flow	Shallow parallel erosion channels on bare slopes, becoming gullies below.
Pipes (shallow concentrated subsurface drainage and erosion)	Generally not identifiable on aerial photos until collapse occurs. Settlement along a pipe line may cause linearity in surface drainage. Subsurface flow may lead to wetter surface soils. Both conditions may cause lines of vegetation.
Springs	At base of steep slope (strong pervious rock/weak impervious rock boundary). At junction between permeable and overlying impermeable layers (colluvium/residual soil; rock strata). At base of slope failure back scar. Wet ground, often with dense vegetation. At heads of stream channels.
Gully heads	Steep-sided erosion bowl at head of a gully.
Eroding gullies	Irregular channel in plan and width shows variable erodibility of channel materials. High reflectance from bare surfaces. Obstructions such as fallen trees and boulders in channel bed. Sediment deposited downstream in channels and on fan surfaces.
Flood plain and valley floor Active fans/high sediment yield	Very low stream banks and evidence of sediment overspilling banks. Fan may have higher elevation than surrounding slopes. Lines of sediment orientated parallel to stream. Eroding source area for debris. Lack of vegetation on channel bed. Tributary fan encroaches into main river. Preserved recent depositional bed forms.

Table 5.3 Appearance of terrain features in aerial photographs (continued)

Feature	Appearance in aerial photographs
Flood plain	Uniform or braided sediment-laden river bed.
	Unvegetated except for isolated islands of flood plain terrace.
Flood plain terrace	Immature or no tree cover.
	Usually uncultivated.
	Sediment deposited onto banks from recent flow.
	Recent erosion of terrace bank.
Higher level terrace	Usually either covered in mature trees or cultivated.
	Bordered by flood plain terrace or directly scoured by river.
Active terrace bank erosion	Terrace edge cuts across field boundaries.
	Boulders too large to be removed by erosion mark previous bank positions.
	Vertical banks with no vegetation: high reflectance.
	Successive aerial photos may show rates of bank retreat.
Flood level	Usually only identifiable on large scale air photos if taken within a year of flood.
	Strand-lines of vegetation, sediment and slope erosion.
	High flood in main valley may cause deposition of sediment at this level in tributaries.
Lower valley-side scour by main river	Fresh exposures of rock (high reflectance).
	Absence of vegetation.
	Instability on the lower valley-side caused by undercutting.
	Absence of terrace deposits.
	Large boulders deposited at the foot of the slope by river flow.
High level fan	Cone-shaped deposit at mouth of tributary valley above level of current main river action.
	No active or recent sediment on fan surface.
	Cultivated fan surface with signs of semi-permanent habitation.
	If fan surface is tree-covered rather than cultivated, surface flow may still occur intermittently.
	Fan may contain active channel incised to level of present-day drainage system.

Table 5.4 Comparison between the Landsat MSS, Landsat TM and SPOT satellites

Technical characteristic	Landsat Multi-Spectral scanner	Landsat Thematic mapper	SPOT
Country of origin	USA	USA	France
Date of operation	1972 to present	1982 to present	1986 to present
Satellite altitude	913km[1] or 700km[2]	700km	832km
Time of crossing Equator	9.30 am approx.	9.30 am approx.	10.30 am
Revisit frequency	18[1] days or 16[2] days	16 days	26 days[3]
Number of spectral bands	4	7	3, plus 1 panchromatic
Image size on ground	185 x 185km	180 x 180km	60 x 80 or 60 x 60km[4]

NOTES
1. Landsats 1 and 2
2. Landsat 3
3. The satellite is capable of taking images of the same area at intervals of one to several days by directing its sensors right or left along preceding or future orbit paths.
4. According to whether satellite is looking vertically or obliquely (see Note 3).

Table 5.5 Advantages and disadvantages of the three main types of satellite imagery

Satellite	Advantages	Disadvantages
Landsat Multi-Spectral Scanner	• World coverage on archive • Coverage dates from 1972 • Large image size (185 x 185km) • Least expensive (archive)	• Rather low spatial resolution (80m nominal) • Only 4 spectral bands
Landsat Thematic Mapper (TM)	• World coverage on archive • 7 spectral bands available • Resolution better than MSS (30m nominal) • Large image size (185 x 185km)	• Requires a lot of computer processing power to explore all 7 bands
SPOT	• Best spatial resolution (multispectral 20m: panchromatic 10m. Nominal) • Stereoscopic capability	• Image size is only 60x6Okm • Only 4 spectral bands • Not full world coverage (archive) • Archive stereoscopic imagery

SATELLITE IMAGE INTERPRETATION

5.21 There are three principal types of satellite imagery, namely Landsat Multispectral Scanner (MSS), Landsat Thematic Mapper (TM) and SPOT. The technical characteristics of the satellites are listed in Table 5.4, but the choice of imagery often depends on what is available. The advantages and disadvantages of each type of imagery are given in Table 5.5. SPOT panchromatic imagery can be acquired in stereo-pairs from which a digital terrain model with a horizontal accuracy of about 10m and a vertical accuracy of about 20m can be derived.

5.22 An example of a satellite image of mountainous terrain and its interpretation is reproduced in Figure 5.2 (page 40). Satellite images are compared with aerial photographs in Table 5.6, but the principal advantages of satellite imagery over aerial photographs include the ability to portray large areas of ground surface showing large scale geological structures, drainage patterns, river catchment areas and broad land use patterns. The preparation of an aerial photograph mosaic to cover an equivalent area would be time-consuming and very expensive, and would in any case suffer significant distortion and changes of scale in areas of high relief.

5.23 The main applications of satellite image interpretation to road feasibility studies in mountain areas relate to:

- measurement of distances and areas, especially catchment areas
- broad classification of terrain types
- assistance in the identification of route corridors
- coarse differentiation between rock and soil slopes, broad soil and vegetation types, areas of high groundwater and large scale slope instability
- identification of erosion (even small areas, near the resolution limit of the image, are visible if they are of high contrast with their surroundings)
- identification of major geological structures and their influence on topography and regional slope stability
- identification and mapping of large water bodies that could give rise to Glacier Lake Outburst Floods (GLOFs)
- identification of broad changes in the course of rivers, land use and snow cover over time if sufficient archive data exist.

Furthermore, satellite imagery can be used as a means of monitoring some aspects of environmental impact, and especially land use change during and after project implementation.

Table 5.6 Comparison between aerial photography and satellite imagery

Image type	Advantages	Disadvantages
Aerial photography	Low unit cost, though total cost depends on size of area covered	National cover often incomplete
	Widely available through survey departments	Access can be restricted in sensitive border areas
	Very good image detail on black and white photographs	No colour information
	Colour enables drainage, land use and soils to be interpreted more readily	Slightly poorer image detail on colour photographs
	Stereoscopic image	Scale not constant. Cloud cover sometimes a problem
	Specialist training in use is minimal	Digital analysis not possible
	Familiarity as a conventional source of information	Large numbers of photographs needed at desk study stage
Satellite imagery	World-wide coverage (Landsat)	High unit cost, although cost depends on size of area in image
	Access to imagery not normally restricted	Relatively low spatial resolution
	Purchased by mail order from remote sensing centres	Stereoscopic imagery rarely available (though likely to increase)
	Colour information	Cloud cover sometimes a problem
	Digital image analysis possible	Specialist equipment necessary for digital analysis
	Constant scale	

5.24 Satellite imagery can be purchased in digital form (on tape or disc) or as photographic products. These two forms are compared in Table 5.7. Despite the potential applications offered by satellite imagery, if essentially cloud free, stereo aerial photography exists at a scale of 1:30,000 or larger, then the only practicable advantage in procuring satellite imagery for route feasibility and design purposes lies in the production of an essentially distortion-free photographic map of a given area that can be used for visual orientation and as a project planning base document. In the absence of topographical mapping it can also represent a useful means of measuring distances and areas.

5.25 Before a decision is made to purchase satellite imagery it is advisable to be aware of the following factors:

- if satellite imagery is not available from archive sources, it can be commissioned, although this procedure is more expensive and takes longer for delivery of images
- a higher spatial resolution is generally preferable
- all three types of imagery listed in Table 5.4 are impaired by atmospheric moisture and, in tropical areas especially, cloud cover can be a mayor constraint
- Imagery obtained in the dry season is more likely to be cloud-free and generally portrays a clearer definition of soil and rock conditions
- the percentage of cloud cover given in computer listings of archived imagery can be incorrectly stated, especially in areas where high ground is covered with snow
- colour prints of satellite images cost between 2 and 6 times more than black and white
- if photographic products are required it may be preferable to purchase film negative rather than film positive or paper print, so that extra copies can be made cheaply
- digital data are required for digital image processing and Geographical Information System (GIS) applications
- geometrically-corrected imagery will eliminate distortion, although the accuracy of the final image can only be as good as the Ground Control Points in the underlying co-ordinate system and the maps or other means by which they are identified
- as with aerial photographs, the reliability of the interpretation must always be verified by other forms of evidence.

5.26 Radar imagery is unaffected by cloud cover or atmospheric moisture, and therefore provides an alternative to conventional photographic satellite imagery in areas where

Table 5.7 Comparison between photographic and digital satellite image products

	Advantages	Disadvantages
Photographic satellite images	Prints are easy to annotate. Images are geometrically correct. Images can be enlarged to a scale of 1:100,000 without serious degradation. A full scene can be reproduced at maximum resolution of the photograph. If a negative is purchased, multiple copies are cheap to produce.	Specialist photographic expertise is required to process or manipulate bands. Dynamic (colour) range is limited. A good colour balance for all parts of the image is difficult to achieve.
Digital satellite images	Colour can be optimised for differentiation of any type of feature. Resolution down to individual pixels can be achieved. Different combinations of spectral band can be displayed quickly. Processing and manipulation of the image is very quick. Mathematical and statistical processing can be carried out on the spectral data. Images can be fitted to any cartographic projection.	Image processing equipment is required for display and output. Hard copy has to be made from photographs of the display screen, colour printers or specialist film writing equipment. Only a fraction of the image can be displayed at full resolution on the screen.

these are limiting factors. The main disadvantages with radar imagery are a limited (but growing) availability and difficulty of interpretation.

RECONNAISSANCE SURVEY

5.27 When potential route corridors have been identified from the desk study analysis, then reconnaissance survey is usually employed to verify interpretations, to help determine the preferred corridor and identify factors that will influence the feasibility design concept and cost comparisons. Even at the reconnaissance stage, data collection should be detailed enough to identify the most significant sources of construction difficulty in the landscape. Allowing sufficient time and resources to identify and evaluate these problems during reconnaissance survey could avoid costly readjustments to the design concept or the route corridor later on. The team employed to carry out the reconnaissance survey should include adequate geotechnical expertise, familiar with the problems expected to be encountered. Hydrological and environmental assessments will also form important elements of reconnaissance survey.

5.28 In most cases, reconnaissance survey will significantly modify the desk study interpretations. In the absence of any worthwhile desk study data, or where dense forest cover prevents the successful interpretation of aerial photographs, reconnaissance survey will have to be relied upon as the sole means of route corridor identification and evaluation.

5.29 Reconnaissance survey data can either be recorded onto topographical maps or enlarged paper prints of aerial photographs. The latter can form a particularly useful means of recording data in the field, especially in areas where forest cover is relatively low. Minor ground features, such as houses, field boundaries, cliffs and water courses, can be readily identified. However, the scale of aerial photograph enlargements is highly inconsistent and alignment length should be approximated from topographical maps and then transferred onto the enlarged photographs. GPS can be an extremely useful means of determining coordinates for specific locations.

5.30 During reconnaissance survey, the following factors should be considered:

- terrain classification units
- the location of topographical constraints, such as cliffs, gorges, ravines and any other features not identified by the desk study
- slope steepness and limiting slope angles identified from natural and artificial slopes (cuttings for paths, agricultural terraces and existing roads in the region)
- slope stability and the location of pre-existing land-slides
- preferred cross-sections according to terrain units
- rock types, geological structures, dip orientations, rock strength and rippability
- percentage of rock in excavations
- materials sources
- soil types and depths (a simple classification between residual soil and colluvium is useful at this stage)
- soil erosion and soil erodibility
- slope drainage and groundwater conditions
- drainage stability and the location of shifting channels and bank erosion
- land use and its likely effect on drainage, especially through irrigation
- likely foundation conditions for major structures
- approximate bridge spans and the sizing and frequency of culverts
- flood levels and river training/protection requirements
- environmental considerations, including forest resources, land use impacts and socio-economic considerations (Chapter 6).

5.31 Data collection should be designed according to the terrain classification units discussed in paragraph 5.8-5.13 and recorded directly onto maps and, where time permits, onto proformas according to the terrain classification system adopted. An example of a reconnaissance survey proforma is shown in Figure 5.5. Usually anything more detailed than this, or the use of multiple proformas, is time-consuming to compile and therefore not justifiable in terms of improved corridor assessment.

LANDSLIDE HAZARD MAPPING

Introduction

5.32 The aim of Landslide Hazard Mapping (LHM) is to portray the geographical variation in the susceptibility of slopes to failure. LHM is based on the premise that the relative potential for slope failure can be assessed by reference to the existing distribution of identifiable failures and by measuring and comparing the distribution of landslide-controlling factors. From the point of view of route location, LHM may be able to provide an early appreciation of relative slope stability over a comparatively large area at the feasibility stage, without going to the expense of detailed and extensive geotechnical ground mapping. However, if air photo interpretation, terrain classification and

Airphoto No: 53 | **Map No:** 12 | **Route / Sheet No:** 1

Terrain Classification Unit: All | **Slope Angle:** 10 - 20° | **Land Use:** Irrigated farmland

Chainage / Length: From: Km 50 To: Km 55 | **Date:** 6/12/95

Soil Type: Dark brown to reddish brown, slightly clayey silt and silty gravelly boulder colluvium

% Residual: 50

% Colluvium: 50

Soil Depth: 1 - 3m

% Soil in E/W: 80

Geomorphology: Undulating rounded spurs and colluvial slopes beneath rock cliffs. Slopes steepen into valley below

Rock Type and Structure: Moderately weathered weak GNEISS dipping 30/NE, ie. into slope

% Outcrop: 10

% Rock in E/W: 10

Materials Availability: Poor, river crossed at km 45 is most suitable source

Slope Drainage and Groundwater: Irrigation dominates slope drainage. Natural gullies are relatively undeveloped. No major seepages noted

Design of Earthworks: Mostly 0.5 cut 0.5 fill with full cut though spurs. 1:1 soil cuts, 2:1 rock cuts

Culvert Sizes

	1m Ø	2m Ø	3m Ø
NUMBER	40	5	

Bridge Nos & Spans: None

Scour Protection: Nominal gabion aprons to culvert outlets

Slope Stability: No problems identified, though old failure and erosion scars noted in fractured rock on steep slopes below alignment

Length of Alignment Unstable: 0

Remedial Works: None required, though control of irrigation water and farming practices in general above cuttings will require detailed consideration

Further Comments: Cliffs above alignment may require inspection to identify potential rockfall sites, though none of the debris at the base of the cliff appears particularly fresh

Figure 5.5 Field data sheet for comparison of alignment feasibility

reconnaissance survey can be satisfactorily combined, or if a small number of feasible route corridor options can be easily recognised in the field, LHM may not prove to be a particularly cost-effective use of resources if carried out specifically for an individual road project. Furthermore, if large volumes of primary data have to be collected to complement insufficient desk study sources, then the speed with which LHM can be carried out will be significantly reduced.

Common methods of landslide hazard mapping

5.33 LHM employs both direct and indirect methods of assessment. For direct mapping, a study area is zoned according to the location or density of recorded landslides, on the assumption that future landslides are more likely to occur in areas with similar slope conditions to those where landslides have occurred already. Indirect mapping relies on the evaluation of factors that are considered to be significant in the initiation of slope failure. Figure 5.6 lists some of the factors that contribute to the stability or failure of slopes in the Himalayas. Clearly, through lack of data, only a small proportion of these factors can be evaluated during route corridor feasibility study. The aim of LHM for corridor comparison should be to identify landslide locations and to compare this database with whatever terrain and factor classifications can be provided from topographical maps, geological maps, aerial photographs and reconnaissance survey, thus combining direct and indirect mapping methods.

5.34 Figure 5.7 presents a recommended procedure for carrying out LHM. Box 5.1 outlines the procedural stages that should be followed. A simple statistical comparison between the density of existing landslides and terrain classification units may be all that is required if a significant relationship is obtained. The availability of aerial photographs is the key to the speed and success with which this exercise can be performed. If aerial photographs are unavailable, then it may be more cost-effective to revert to more detailed reconnaissance survey and geotechnical mapping of selected corridors, rather than to spend a great deal of time in the field generating primary data for LHM over large areas.

5.35 Wherever possible, differentiation should be made between landslide failure mechanisms, particularly the distinction between landslides and areas of erosion, and deep-seated and shallow failures. This is an important consideration because these various distinctions pose different risk implications for road construction and maintenance, and their occurrence will usually be affected by different factors.

5.36 The main disadvantages with using direct LHM in isolation relates to the fact that it does not permit the potential for `first-time' failures to be evaluated, or for ground conditions to be compared in terms of broad susceptibility to disturbance during construction. For instance, a weak rock type may show no visible signs of failure because slope angles are too shallow to initiate movement. During construction. however, slope excavation for road-works may give rise to widespread failure. Furthermore, aerial photograph interpretation, and even ground reconnaissance survey, may fail to recognise the more subtle failures, and therefore it is worthwhile evaluating contributory factors that can be readily identified from available sources.

Box 5.1 Recommended outline procedure for Landslide Hazard Mapping

- determine the distribution of existing slope failures in a given study area, through a combination of air photo interpretation and reconnaissance survey
- identify the most significant factors that control or trigger slope failure in the study area
- divide the study area into slope units using terrain classification or geomorphological criteria
- systematically measure landslide controlling factors for each terrain unit and assign values to class intervals according to natural breaks or observed relationships in the landscape. For instance, slopes on rock type A may be known to fail when slope angles exceed 25°, while slopes on rock type B may remain stable at slopes of up to 45° if foliation or bedding orientations are unfavourable to stability, and up to 60° where they are favourable
- assign a hazard rank to each factor class according to its correlation with the observed distribution of landslides (if the distribution of landslides cannot be reliably represented then the hazard rant: assigned to the various factor classes will have to be made on the basis of field observation and judgement)
- test the statistical validity of the final hazard classification against the distribution of landslides and by any other independent means, including ground verification
- the final hazard map should depict no more than five levels of hazard and should be accompanied by the landslide distribution map and an explanatory text.

Applications of landslide hazard mapping

5.37 LHM has been successfully applied to land use planning in mountain areas where economic and engineering development criteria have required geographical comparisons of landslide potential within the context of a regional development or land capability/land use management programme. Intensive data collection, rigorous modelling and a thorough knowledge of local landslide - controlling factors can then give LHM a reliable analytical basis, and the relatively high cost involved is justified.

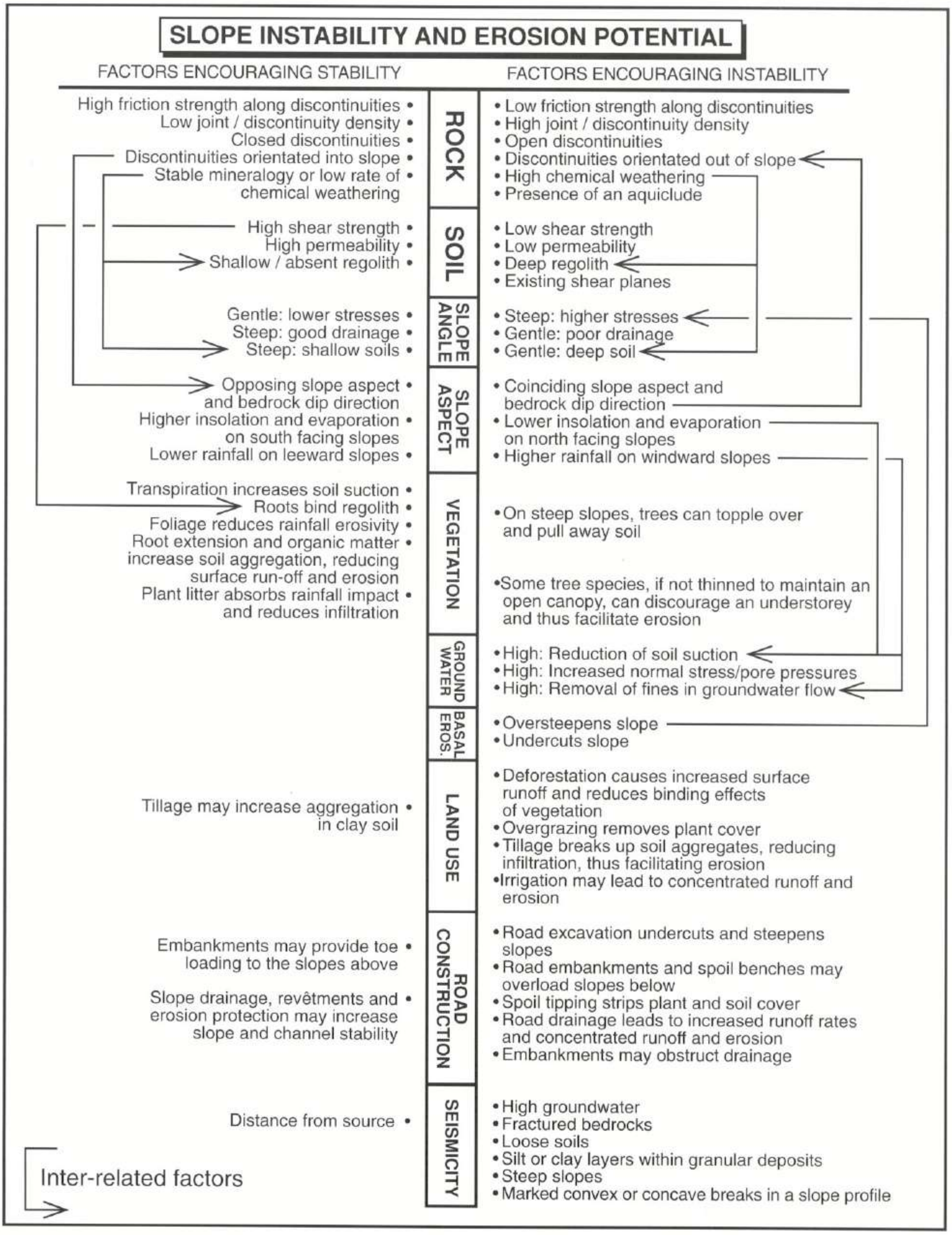


Figure 5.6 Factors influencing slope instability and erosion potential in the Himalayas

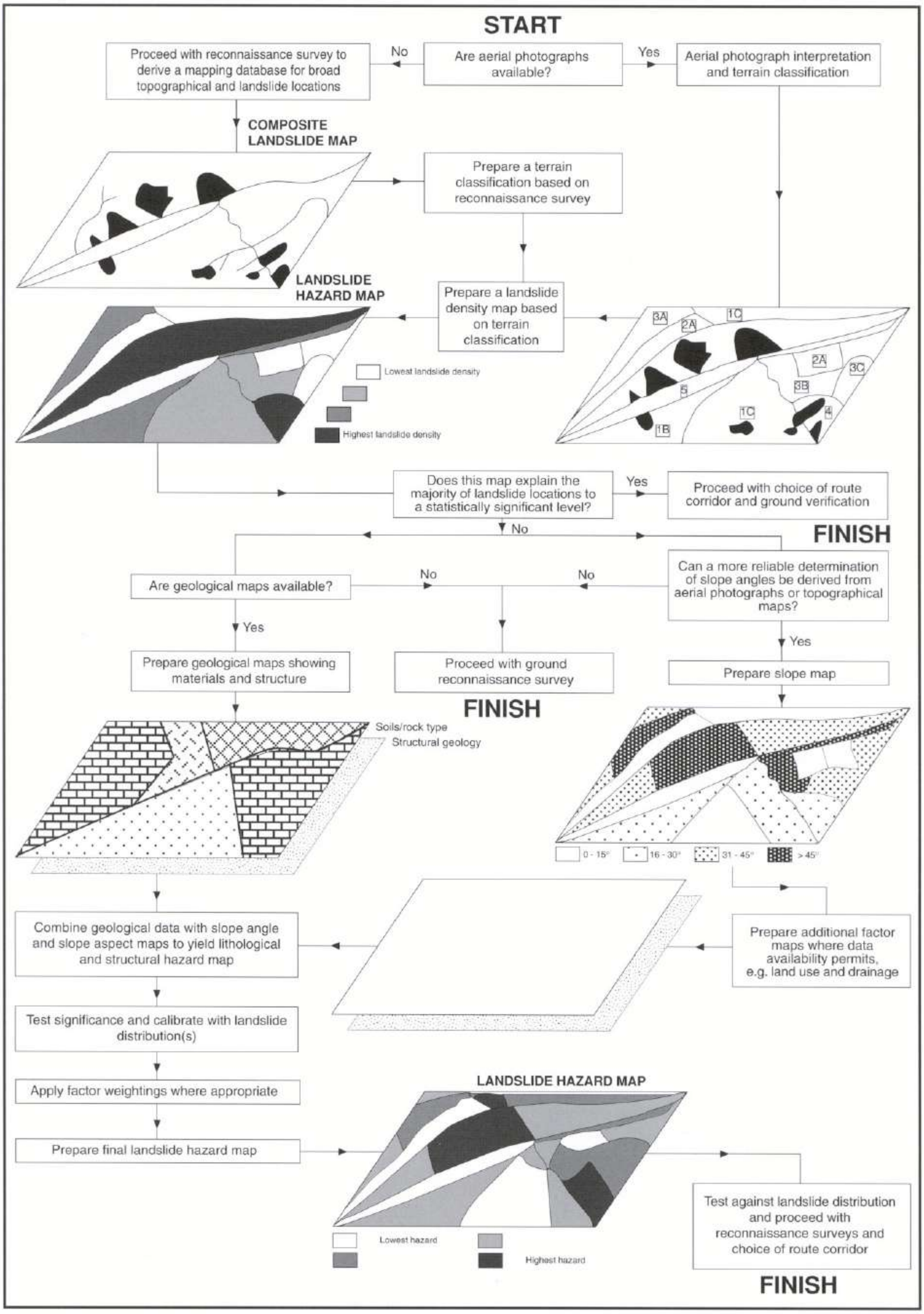


Figure 5.7
Recommended procedure for landslide hazard mapping in remote mountain terrain

5.38 In the case of new road construction through mountain areas, the pre-existence of a LHM database would provide a useful point of reference for route corridor identification. However, it is usual to fund that such a database is unavailable, and that any given road project in remote mountain areas will have an extremely limited source of information at its disposal. Under these conditions, there are several reasons why, in practice. LHM may not provide the objective and reliable estimate of relative stability that is often claimed or implied (Box 5.2).

5.39 Despite these reservations, LHM may prove a useful means of imposing the need to evaluate a range of terrain characteristics in a methodical way, thus ensuring that all available information is taken into objective consideration. In general, the development and adoption of systematic guidelines for collecting, analysing and interpreting LHM data could improve the quality and reliability of the end-product. Furthermore, LHM and risk assessment can often be employed more effectively on a site-specific basis in evaluating hazard potential and runout distances on individual slopes, or in logging and classifying the stability of earthworks slopes during road construction and maintenance. Some illustrations of LHM applications are provided in the following sections and in Chapter 7.

Box 5.3 Distinction between `hazard' and `risk'

The terms 'hazard' and 'risk' have often been used synonymously in the published literature, creating a source of confusion. The term 'hazard' defines the physical attributes of a potentially damaging event (landslide, flood or earthquake) in terms of type, mechanism, volume and frequency. 'Risk' is governed by hazard type, size and probability of occurrence, the value of the property, structure or population at risk and the vulnerability to the hazard, should it occur. In other words, 'hazard' describes the event and `risk' describes the probability and total consequences of the event occurring.

Although risk can rarely be fully evaluated, some studies have attempted to assess the probability of landslides of given sizes and potential impacts, by reference to the return period of triggering events such as earthquakes and rainstorms. Even then. the relationship between seismicity and the timing and distribution of slope failure is largely unknown, while antecedent rainfall and seasonally fluctuating groundwater levels are often just as important in triggering deep-seated slope failures as 24 hour storm rainfall or maximum rainfall intensity.

Box 5.2 Landslide hazard mapping (LHM) for route alignment purposes

The following points should be borne in mind when considering whether or not to embark on an LHM exercise for road alignment in remote mountain areas:

- there are no standardised guidelines for carrying out LHM
- LHM does not calculate an absolute value of slope stability, only a relative one. Unless rigorously applied. LHM may be incomplete in its assessment of hazard as defined by failure potential, depth of failure and failure mechanisms. Also, it rarely enables risk, or the absolute sensitivity of a slope to disturbance by road construction, to be determined
- landslide-controlling factors are difficult to measure or estimate accurately. Many are underground and some are time-dependent, eg groundwater fluctuations
- unstable mountain slopes are often characterised by multiple hazards of various mechanisms, and it may be difficult to represent them all on a single map with a common hazard ranking system. Furthermore, each will have a different risk implication for road engineering
- the problem of lack of data can, to a certain extent, be resolved by reconnaissance survey, but from a mountain road design point of view, it would be generally too costly and take too long to reconnoitre a large area in order to make a landslide hazard map merely for use as a means of identifying route corridors. A large proportion of the data would become redundant once the preferred corridor was identified
- leading on from the above, decisions regarding alignment options are usually required to be made long before LHM based on ground survey can be completed. Conventional air photo interpretation with reconnaissance survey is the most efficient way of identifying and selecting alignment options
- in young fold mountains, seismic shaking is an important trigger mechanism for slope failure and yet there are very few examples where thus factor has been incorporated into LHM. The main reasons for this are the indeterminate nature of earthquake intensity and the uncertainties regarding the manner in which slope materials will react to shaking.

Landslide hazard mapping for route alignment in Nepal

5.40 This study of relative slope stability for route corridor comparison in one of the most unstable parts of east Nepal was carried out as a separate research exercise after road construction to compare LHM with the original geotechnical reconnaissance survey. The latter did not have the benefit of good quality aerial photography or reliable topographical mapping, both of which were obtained later in the project.

5.41 The study area for the LHM comprised three main drainage basins covering a total area of $120km^2$. Black and white 1:25,000 scale aerial photographs, 1:50,000 scale topographical maps derived from the aerial photographs and 1:200,000 scale geological mapping, were used to derive a series of factor maps. Separate analyses were carried out for each of the three catchments in order to minimise the effects of extreme geographical variations in rainfall that could not be reliably quantified. The observed densities of landslides occurring within each factor category were compared with those expected had there been no control on the distribution of slope failure (a random distribution). Rock type, physiography and slope aspect were found to be significant factors, and the various factor classes were weighted according to their observed relative control on landslide distribution. Land use was found to be a significant controlling factor in two of the three catchments, while slope angles measured by parallax on the aerial photographs over grid cells of 230 x 230m proved too coarse to be significantly correlated with landslide distribution, even when analysed according to rock type. Figure 5.8 shows the sequential factor mapping undertaken for one of the catchments and the final hazard classification with respect to the constructed alignment. The hazard classification vindicated the chosen route and was found to be significantly correlated with the distribution of additional landslides mapped from aerial photographs taken seven years later, thus supporting its application as a predictive tool.

5.42 The LHM procedure described here took two man-months to complete and yielded essentially the same conclusions with respect to route corridors as those reached by the reconnaissance survey, which was accomplished in approximately the same mantime duration. However, the reconnaissance survey yielded site-specific data that was incorporated into design later on and therefore represented a more cost-effective use of mantime, with or without the availability of desk study data sources. Although the landslide distribution map differentiated between hazard mechanism, to the extent that this is possible from air photo interpretation, deep-seated slope failures and the factors that promote them were not adequately represented in the analysis. Shallow failures are much greater in number and usually far easier to identify.

Rock avalanche hazard mapping in Papua New Guinea

5.43 The objective was to provide an assessment of landslide and avalanche runout potential from a steep mountainside located above a mining township. Slopes are covered by dense rainforest, and consequently all data, covering an area of over 3km=, had to be dewed by geotechmcal ground survey using compass traverse at a scale of 1:2,500.

5.44 Fieldwork took approximately 5 man weeks to complete and was followed by a similar period in the office carrying out analysis, interpretation and reporting. Figure 5.9 illustrates the various stages of the mapping and analytical procedure. The geotechnical survey was used to:

- record evidence of active and previous slope failures
- divide the study area into a total of 494 terrain units
- derive a suitable database for landslide hazard mapping.

5.45 Tables 5.8 and 5.9 list the various rock and soil factors used in the LHM, the categories identified and the weightings employed. Separate analyses were carried out for rock and soil slopes. Each factor was assessed systematically for every slope unit and assigned a category value which was then multiplied by the weighting for that factor to yield the hazard score. The hazard scores were then summed for each slope unit and assigned to one of four hazard classes. The distribution of landslides identified by the geotechnical survey was used to test the validity of the hazard classifications. For both rock and soil slopes, it was found to be significant at the 95% level.

5.46 The source areas and failure deposits belonging to previous landslides were also recorded during the geotechnical survey. By categorising each landslide according to estimated volume and mechanism, and by measuring slope angle and slope length in the failure zone and slope angle and runout length in the depositional zone, a set of regression equations was established for predictive purposes. Estimation of the likely volumes of potential failures, and the relevant slope angles and slope lengths in the high hazard areas, allowed potential runout lengths to be predicted from these equations and precautionary measures to be recommended (Figure 5.9e).

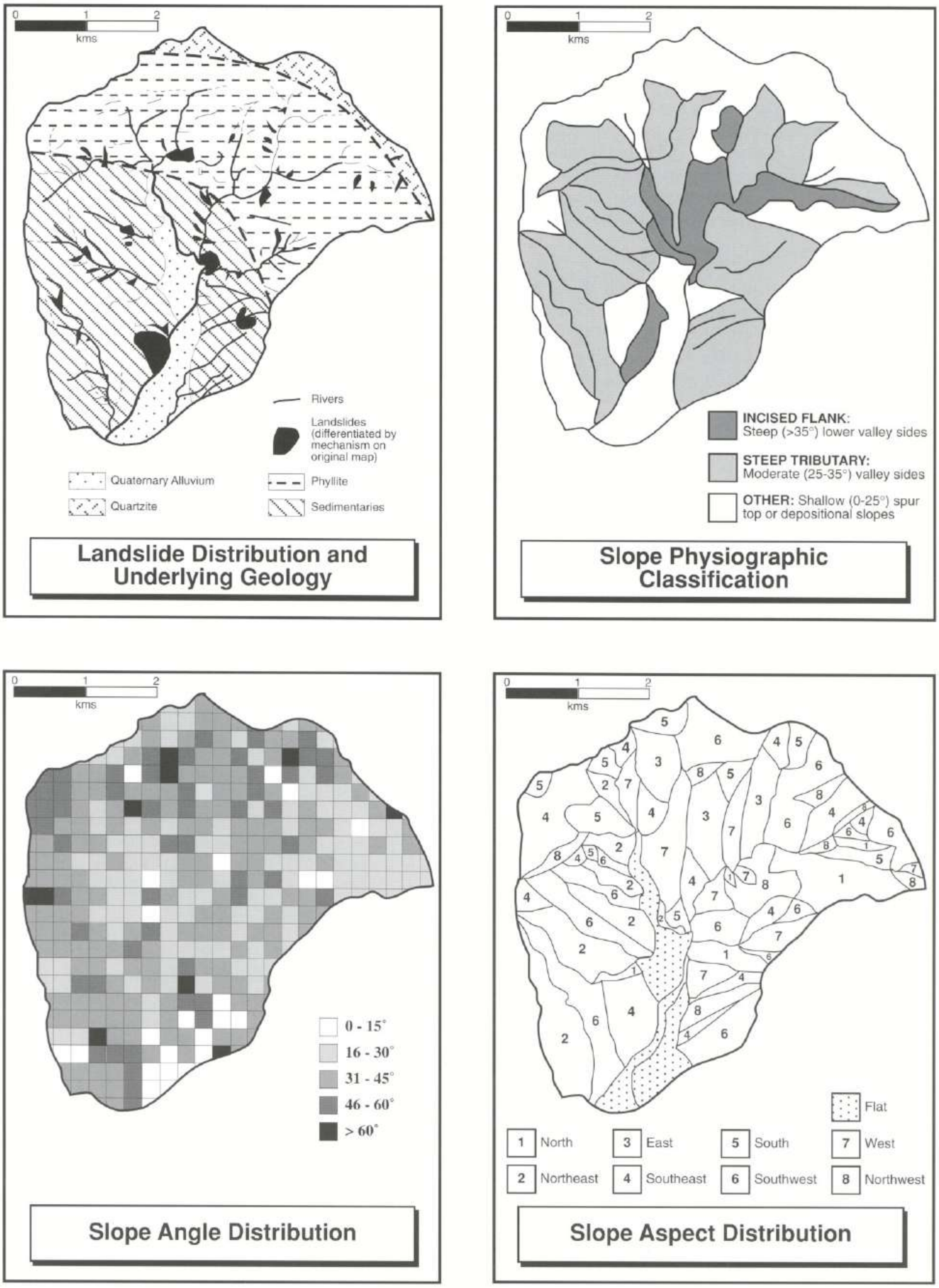
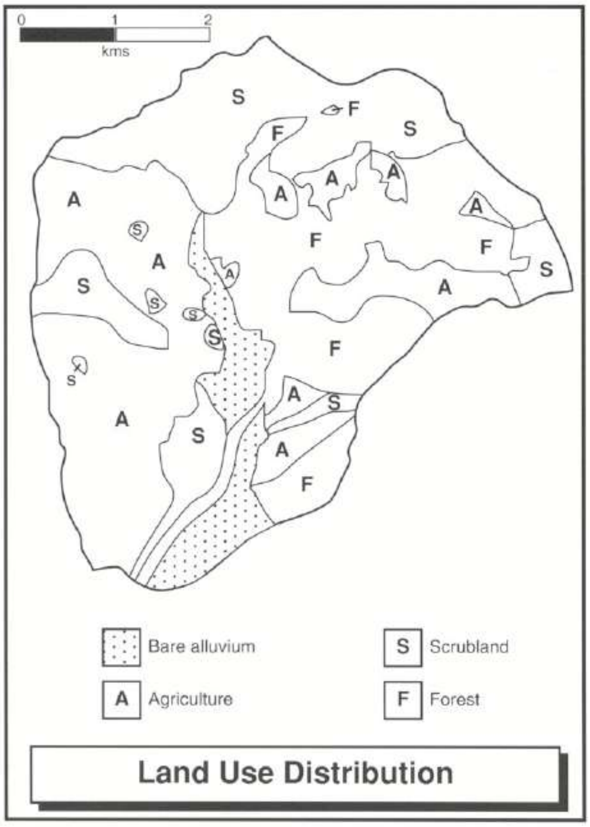
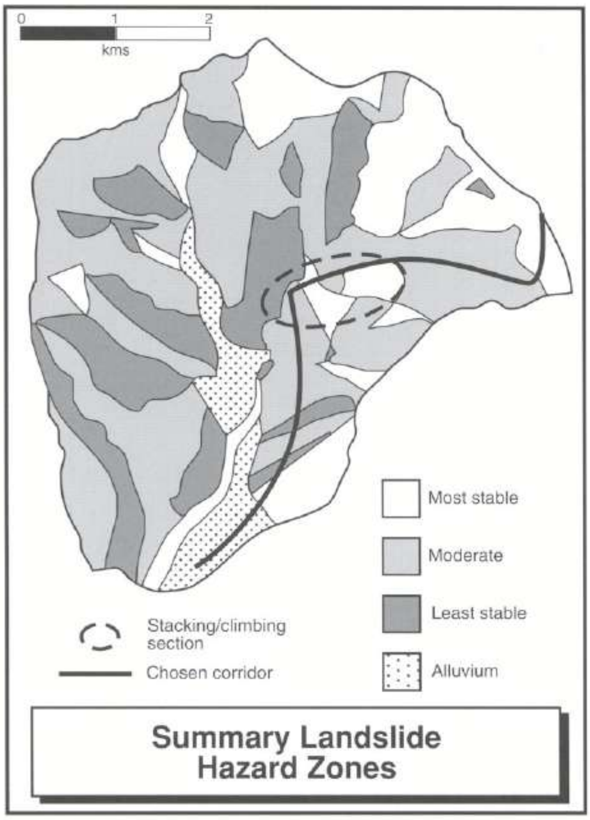


Figure 5.8
Landslide hazard mapping for route alignment through an unstable river basin in east Nepal





FACTOR	CATEGORY	O / E	HAZARD RANK
ROCK TYPE	Sedimentaries	1.2	3
	Phyllite	0.8	2
	Quartzite	0.3	1
SLOPE ASPECT	North	0.7	1
	South	2.0	2.5
	East	2.2	3
	West	0.7	1
	Northwest	0.6	0
	Northeast	0.9	2
	Southeast	0.9	2
	Southwest	0.5	0
PHYSIO-GRAPHY	Incised flank	2.9	3
	Steep tributary	1.1	2
	Other	0.3	1
LAND USE	Scrub	1.0	NOT SIGNIFICANT
	Agriculture	1.1	
	Forest	0.9	
SLOPE ANGLE	0 - 15°	0.7	
	16 - 30°	1.0	
	31 - 45°	1.3	
	46 - 60°	0.8	
	> 60°	0.4	
CHANNEL PROXIMITY	Stream rank:		
	First Order	0.9	
	Second Order	1.1	
	Third Order	1.5	
	Fourth Order	0.9	
	Fifth Order	1.0	

Build up of hazard rank for the illustrated catchment. Hazard ranks for each factor category are summed for every terrain unit and assigned to one of three hazard classes. 3 is the most unstable condition.

N.B. The expected number of landslides (E) is calculated on the basis of the percentage study area coverage of each factor category multiplied by the total number of observed landslides (O) in the study area. Thus, higher O/E ratios indicate a greater occurrence of instability than would be expected from a random distribution.

Figure 5.8
Landslide hazard mapping for route alignment through an unstable river basin in east Nepal (Continued)

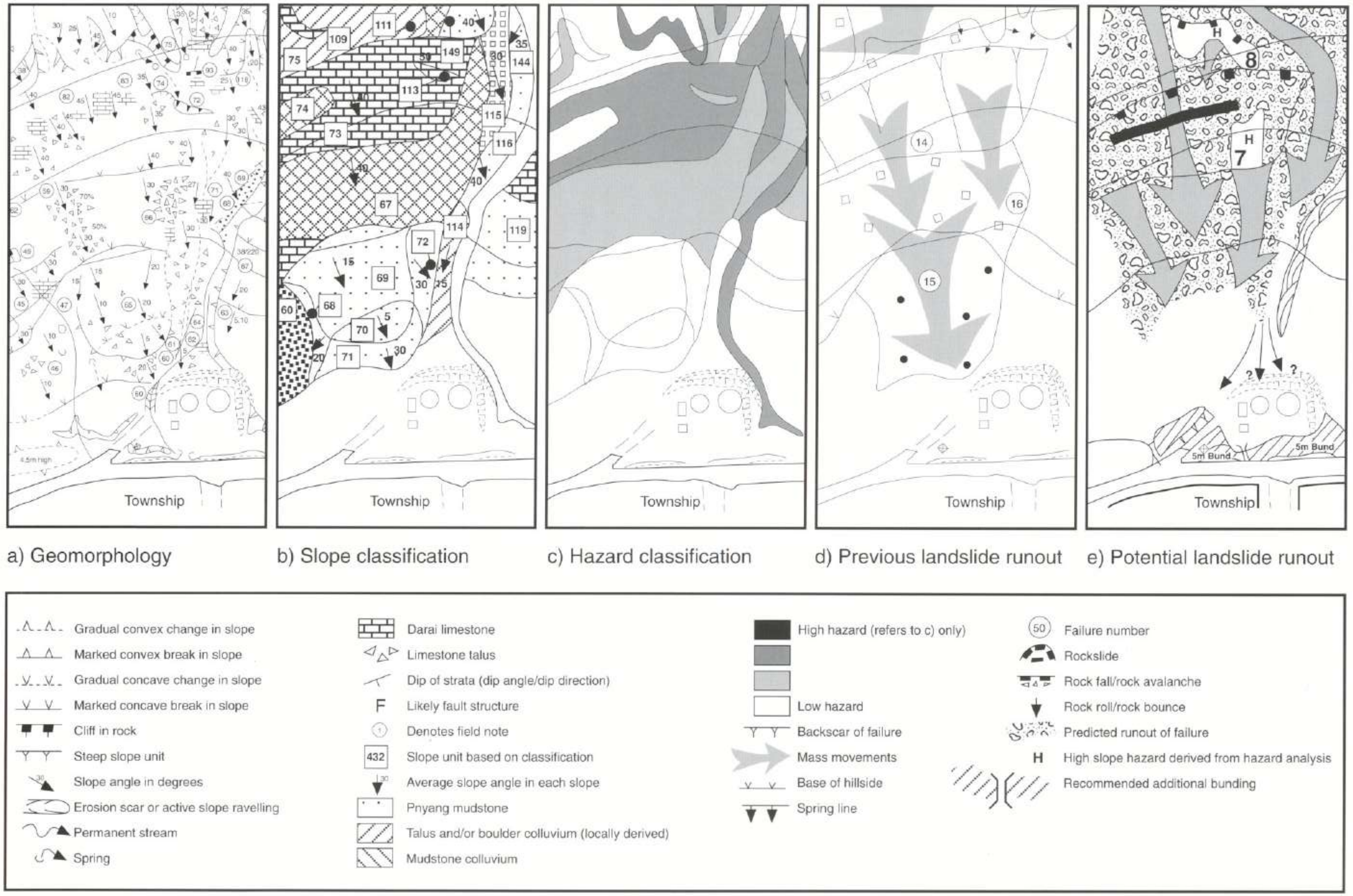


Figure 5.9 Landslide hazard and runout mapping in Papua New Guinea

Table 5.8 Rock slope hazard rating for a site in Papua New Guinea (on scale of H = 0 (least hazard) to H=3 (greatest hazard)

	H
Factor 1: Slope angle irrespective of lithological dip	
Limestone slopes (based on graphical database)	0
Angles, <30°	
Angles, 31-44°	1
Angles, 45-60°	2
Angles, >60°	3
Weighting factor, 3	
Mudstone slopes (based on graphical database)	
Angles, <25°	0
Angles, 26-30°	1
Angles, 31-35°	2
Angles, >35°	3
Weighting factor, 3	
Factor 2: Slope angle with respect to angle of unfavourable lithological dip	
Limestone and mudstone slopes	
Angle is at least 10° less than dip angle	0
Angle is at least 5° less than dip angle	1
Angle is 10° greater than dip angle	2
Angle is > 10° greater than dip angle	3
Weighting factor: 3 where dip direction is directly out of slope and 2 where dip direction is obliquely out of slope	
Factor 3: Adverse bedrock sequences (permeable overlying less permeable lithologies and/or strong overlying weak lithologies)	
None	0
Repetitive sequence of thin beds	1
Single sequence with thin reservoir or strong rock stratum	2
Single sequence with thick reservoir or strong rock stratum	3
Weighting factor, 2	
Factor 4: Faulted or sheared rock masses (only one of Factors 4 and 5 may be included in rating)	
None	0
Yes, but with favourable structural orientation	1
Yes, with unfavourable structural orientation	2
Weighting factor, 2	
Factor 5: Rock-mass dilatation (only one of Factors 4 and 5 may be included in rating)	
Massive with closed joints	0
Massive with open joints or moderately fractured with closed joints	1
Highly fractured with open joints	2
Crushed and dilated (as in H=1 or H=2 for Factor 4)	3
Weighting factor, 2	
Factor 6: Drainage	
Well-defined channelled runoff and dry slopes	0
No evidence of surface runoff, sinks or seepages	1
Sinkholes, internal drainage or ponding	2
Active springs and seepages	3
Weighting factor, 1 for limestone slopes and 2 for mudstone slopes	
Factor 7: Slope unloading by toe erosion or failure from below	
None	0
Localised and infrequent or due to ancient slope failure	1
Periodic or due to recent slope failure	2
Active	3
Weighting factor, 2	

Table 5.9 Soil slope hazard rating for a site in Papua New Guinea (on scale of H = 0 (least hazard) to H=3 (greatest hazard)

	H
Factor 1: Slope angle (based on graphical database)	
Cohesive soils	
Angles, 0-15°	0
Angles, 16-25°	1
Angles, 26-35°	2
Angles, >35°	3
Weighting factor, 4	
Granular soils	
Angles, 0-20°	0
Angles, 21-30°	1
Angles, 31-40°	2
Angles, >40°	3
Weighting factor, 4	
Factor 2: Drainage	
Well-defined channelled runoff and dry slopes	0
No evidence of runoff, sinks or seepages	1
Sinkholes, internal drainage, tufa deposits and dispersed runoff	2
Active springs, seepages and waterlogged ground	3
Weighting factor, 3	
Factor 3: Soil type	
Large boulders	1
Granular soils	2
Cohesive soils	3
Weighting factor, 1	
Factor 4: Vegetation	
Mature tree stand with understorey	0
Mature tree stand with no understorey	1
Secondary growth	2
Cleared	3
Weighting factor, 1	
Factor 5: Rockhead variability	
Normal weathering profile	1
Colluvium on irregular rockhead surface	2
Colluvium with rockhead regular and parallel to slope	3
Colluvium on existing rockhead failure surface	4
Weighting factor, 2	
Factor 6: Toe Erosion	
None	0
Localised and infrequent	1
Periodic	2
Active	3
Weighting factor, 2	

6 ENVIRONMENTAL IMPACT ASSESSMENT

INTRODUCTION

6.1 Environmental impact assessment (E1A) is a formalised procedure for investigating, analysing and presenting the environmental implications of a proposed development and identifying mitigation measures required to reduce environmental impact to acceptable levels. The advantage of such a procedure is that environmental impacts and potential benefits are brought to light prior to implementation in an objective way, for discussion between concerned organisations such as the funding agency, the client, natural resource experts, community representatives and the engineering design management. Despite its apparent judicial role, EIA is intended to act in the best interests of all parties by integrating a project smoothly into the environment within the context of the wider conservation and regional development issues.

6.2 The issues addressed by an EIA may already be incorporated into a normal feasibility study, and many road schemes in the past have been designed and constructed without the benefit of EIA and with no apparent adverse environmental effects. Others, of course, have been less sympathetic. Whether or not a formal EIA is carried out it should be stressed that sound judgements in engineering, hydrology and engineering geology will always remain among the most important factors in successfully integrating design, construction and environmental compatibility in hilly or mountainous areas. It is the engineer's responsibility to ensure that, notwithstanding the need to accede to political and economic circumstances, this principle is adhered to, and that physical damage to the environment is minimised.

6.3 Most road projects funded under foreign aid are now required to have an EIA carried out as part of the aid agreement. Because EIA covers such a wide range of issues, it is normal to hire the services of a specialist consultant to carry out the study; this also helps to ensure impartiality. The environmental consultant is normally expected to liaise with the relevant governmental and non-governmental organisations in order to:

- establish or strengthen environmental legislation
- establish, confirm or improve upon recommended guidelines for EIA
- where the environmental consultant is from an overseas country, undertake training of national personnel in EIA and environmental management
- make maximum use of local man-power and technical resources, although this is usually a requirement of development programmes independently of environmental considerations.

6.4 Regional and national conservation and development issues, as well legislative and institutional aspects, are usually considered by government planning departments as part of project identification. They would not normally form part of an EIA for an individual road construction project, except where an environmental consultant is asked to advise on institutional strengthening and legislative policies.

6.5 The relationship between man and his environment is of course complex. However, the real issue behind many environmental concerns in remote (especially mountainous) regions is that separate social groups are brought into contact for the first time as a result of the development project and one can affect the other detrimentally, thus generating conflict between the groups involved. Frequently, one group exploits the natural environment in a way that degrades the habitat of the other, impairing the other's way of life.

IMPACT CATEGORIES

6.6 It is useful to classify a mountain road project EIA into the following environmental impact categories:

- geomorphology
- ecology
- land use
- socio-economic factors.

6.7 Impacts under these categories are expanded in Table 6.1, and their inter-relationships are shown in Figure 6.1. The discussions below illustrate how the impacts interact and show that no impact category can be examined independently of the others.

Geomorphology

6.8 The impacts of a road on landscape and geomorphology are among the primary concerns and responsibilities of the engineer. The geomorphological impacts of road construction relate to the effects of earthworks, spoil disposal and road drainage on slope and channel erosion and slope stability. Most geomorphological impacts of road construction are usually addressed as part of the design, as their effects are immediate and they have a direct bearing on engineering performance. The effects can extend for considerable distances outside the right of way, and can be devastating. Compensation is often either insufficient or non-existent, and thus these impacts can be among the most severe for rural farmers.

Ecology and natural resources

6.9 Ecological impacts can be classified into four groups:

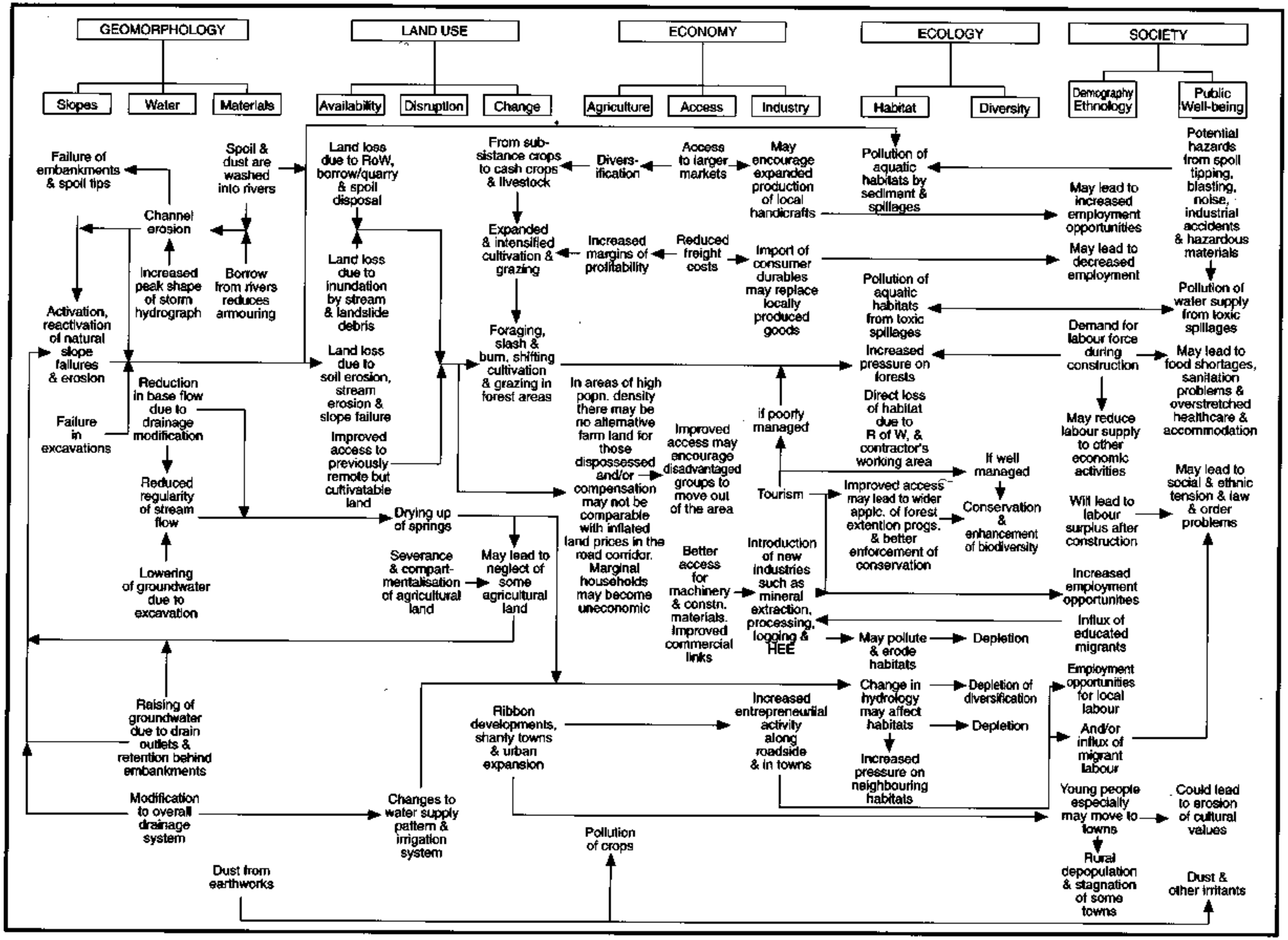


Figure 6.1

Cause-effect interrelationships between environmental impacts along mountain roads serving rural communities

- loss, depletion or disturbance of faunal or floral species (animals and plants) that are rare or protected and are considered to be of regional, national or international importance. In some circumstances, ecological factors such as rare species and diverse forest or aquatic habitats may be sufficiently important to influence the detailed alignment or even the choice of route corridor

- loss, depletion or irreparable damage to rare or vulnerable habitats. Destruction of a habitat reduces a region's biodiversity (the number and variety of plants and animals living within the habitat). Biodiversity is important in maintaining the stability and resilience of the plant and animal populations in the habitat

- reduction in biodiversity (genetic, species and habitat) due to the above effects

- management of natural resources.

6.10 The pollution of lakes and water courses and the depletion of forest habitats are often the most important ecological concerns in sub-tropical mountainous areas. As Figure 6.1 shows, forest habitats can be adversely affected by a number of factors, therefore it is important to assess habitat stability under natural conditions before road-related impacts are evaluated.

6.11 Forest extension schemes attached to a road project may lead to an overall improvement in the management of forest resources, and road access may allow more effective policing of conservation areas. However, road construction usually results in an increased demand for forest products that can put the habitat under considerable pressure.

6.12 Natural resources are those components of the physical environment that are exploited by man. They are important to both ecology and socio-economics. Natural resources can be renewable (such as timber) or non-renewable (such as mineral deposits). Good management of renewal natural resources can enhance the health and productivity of the environment, including the non-resource environment (ie, those aspects of the environment that are not utilised by man). Bad management can deplete both the resource and the non-resource environment.

Land use

6.13 Road alignments tend to occupy less steep ground that is usually of highest agricultural value. The livelihood of subsistence farmers can be severely affected by the loss of cultivable land. Financial compensation for acquired land is usually paid on the basis of cadastral survey and expected yields during the design life of the scheme. The current value of agricultural land is comparatively easy to determine, though future yields may be difficult to assess. An expropriation period equivalent to the design life, or any other time span, is difficult to justify because it is inconceivable that a right of way would be handed back to agricultural use once exceeded.

6.14 The compensation issue is complicated by the fact that the price of land often increases substantially within the route corridor (up to five-fold increases have been reported, and in the vicinity of rapidly expanding urban areas, hundred-fold increases may be common). In these circumstances, farmers who have been compensated for their land at the `normal' rate may not be able to afford the price of replacement land in the vicinity. Rehabilitation and reemployment programmes directed at these farmers and households can be proposed, along with maximum reclamation of the contractor's temporary working areas when construction is complete.

6.15 It is important to respect any existing agreements on the use of water supply for household use and irrigation. Road drainage works, especially hairpin stacks, can significantly affect water pathways.

Socio-economic factors

6.16 These can include:

- traffic and infrastructural aspects, (this factor is directly the responsibility of the engineer)

- regional and national conservation and development issues

- economics, and changes in economic structure

- social, cultural, religious and ethnic factors

- demographic factors

- tourism

- mineral extraction (uncontrolled mineral extraction and waste dumping can be especially damaging)

- legislative and institutional aspects

- pollution (the pollution of water supplies is a particularly sensitive issue).

6.17 Positive socio-economic benefits will accrue in remote areas only if a proposed road scheme is accompanied by a well-planned and well-managed development strategy. The benefits of increased demand for foodstuffs, and hence higher prices, during construction may lead to a short-term improvement in farm incomes. Those who benefit most are farmers or landlords who own large areas of farmland, and those who are able to bring marginal land into agricultural production. Tenant farmers who survive at subsistence levels may experience no economic benefit.

6.18 In the long term the agricultural economy can be adversely affected if farmers become unable to compete with cheaper imported produce. Diversification is a possible solution to this, but it may cause the community to become less self-sufficient and therefore susceptible to external fluctuations in commodity price and availability. In addition, the full economic potential for diversification may not be realised when regions of similar agricultural or industrial output are connected. Under conditions where road access does not result in the import of cheaper food commodities, a rapid rise in population density alongside the road, combined with a general trend towards rural depopulation, may lead to food shortages and increased poverty. This may result eventually in significant migration of people out of the area.

6.19 Demographic issues include population growth, density, distribution and migration, and composition of ethnic, religious and age groups. Often, a development project such as a road scheme brings benefit to certain sections of the population and not others. according to caste or ethnic background, age, sex or education. Thus, disadvantaged groups may become increasingly marginalised and may be forced to move away from the road corridor, or out of the area entirely. A road scheme can actually increase inequality among socio-economic groups.

6.20 The young and the unemployed will tend to move into the road corridor to take advantage of potential employment opportunities, and this may lead to a breakdown in the sustainability of rural communities and a gradual erosion of traditions and cultural values. A further consequence is a rapid expansion of urban areas and shanty developments along the route corridor, within which basic living, sanitation and health facilities may be grossly inadequate. Similar outcomes may arise with the influx of migrant workers who form part of the temporary construction labour force, and this may lead to cultural and ethnic mixing and the potential for social unrest.

ENVIRONMENTAL IMPACT ASSESSMENT PROCEDURE

6.21 The main donor agencies have published their own guidelines on EIA procedure and, nowadays, most projects cannot be implemented without environmental considerations having first been satisfied. The various guidelines correspond approximately to the following schedule of EIA activity. Those in bold type relate directly to engineering:

- **description of proposed project**
- **systematic comparison of alternative schemes, sites, technologies and designs**
- **identification of appropriate mitigation and compensation measures, together with an assessment of their likely success and cost, where possible**
- description of existing environmental base-line conditions, including the quantification of natural trends, such as land stability and rates of erosion, distribution of wealth and poverty, population growth, urban expansion, rural depopulation, deforestation and land use change
- identification and quantification of potential environmental impacts, both direct and indirect, including opportunities for environmental improvement
- recommendations for environmental management and monitoring of mitigation performance
- training of local personnel and institutional strengthening, where appropriate.

6.22 The various stages of EIA procedure are briefly described below. The EIA report should be accompanied by a non-technical summary, known as an Environmental Statement, that is set out according to the following format. The Environmental Statement should outline the manner in which the project will be managed within the wider context of the study area environment.

Project description

6.23 An overview of the proposed (or existing) scheme is required, in terms of its location, size, technical design and construction method, construction time-frame, estimated cost, labour force requirements and operational procedures.

Base-line study

6.24 The base-line study summarises the existing conditions against which the environmental effects of a proposed project are evaluated. Its objectives are to:

- describe the existing environment in terms of ecological zones, vulnerable and protected species and habitats, patterns of human occupation, land use types, and cultural and religious aspects
- quantify, wherever possible, the natural rates of environmental change in order to identify any pre-existing trends and provide the basis for assessing the potential impact of the proposed scheme. Projection of natural trends allows predictions to be made of environmental interactions in future, and the means of protecting and improving the environment to be identified.

Comparison of alternatives

6.25 Investment strategies, construction and operation technologies, design standards and route corridor options will be assessed as part of the engineering feasibility and preliminary design studies, but if EIA principles are incor-

Table 6.1 Positive and negative environmental impacts a) during construction

Impact category	Positive impacts	Negative impacts
During construction		
Geomorphology		Accelerated erosion and slope instability from: - newly-opened earthworks and clearing operations - cultivation of marginal land to meet increased demand and to provide income. Siltation of water courses from spoil and eroded materials. Changing patterns of hydrogeology and surface water runoff.
Ecology and habitat	Rare ecologies endangered under 'natural' conditions can be identified during base-line survey and protection measures implemented or enforced.	Increased demand on resources: hunting, grazing, shifting cultiva tion and harvesting of forest products, with corresponding pressure on forest habitats.
Land use		Loss of agricultural and forest land within right of way. Temporary loss of land due to contractor's working areas, stockpile areas, spoil disposal areas and borrow pits. Depletion/redirection of water from existing irrigation.
Socio-economic factors	Employment opportunities for both migrant and local labour force. Enlarged market for local produce from agriculture and craft Industry. Problems of 'natural' environmental decline in the region of the construction project, such as deforestation, bad agricultural practices, public health and sanitation problems, can be tackled.	Disruption of agricultural practices due to changes in water supply, irrigation schemes and compartmentalisation of land. Influx of migrant workers may lead to: - deficit in food supplies, accommodation and health care facilities. - social unrest and law and order problems. Demand for a construction labour force may adversely affect economics of other labour-intensive activities in the area. Food prices will inflate if agricultural production cannot match increased demand. Expropriation of land may lead to increased poverty among disadvantaged groups, and cause their migration out of the area. Public safety may be at risk from spoil tipping and rock blasting. Pollution: deposition of spoil and eroded materials in water courses may inundate agricultural land and pollute aquatic habitats. Dust and noise from construction operations. Spillage of petroleum products and other hazardous materials may contaminate water supplies and ecological habitats.

Table 6.1 (continued) Positive and negative environmental impacts b) after construction

Impact category	Positive impacts	Negative impacts
After construction		
Geomorphology		Slope failure in cuttings, disposal of spoil from routine clearance operations and concentrated road drainage will continue to aggravate erosion problems. Accelerated erosion due to abandonment of agricultural land cultivated during construction.
Ecology and habitat	Agricultural and forestry extension schemes may enhance sustainable development of natural resources and ensure that habitats remain protected.	Increased prosperity from road operation may place additional pressures on forest products and other natural resources. Certain types of industrial development spin-offs may lead to pollution and loss of ecological habitats.
Land use	Increased demand for agricultural produce due to population growth may provide incentive for farmers to increase productivity and diversify and bring previously abandoned land back into cultivation.	Lowered ground water table adjacent to cuttings - crops suffer from drought. Continued depletion/redirection of water from pre-existing irrigation systems.
Socio-economic factors	Reduced costs of imported food. Reduced costs of transport and increased market potential. Opportunity to diversify agricultural production into cash crops and livestock farming to serve urban markets in other regions. Possible expansion of feeder road system and growth of urban centres. Possibility to improve sanitation, water supply, education, health care and industrial training. Entrepreneurial skills introduced due to commercial opportunities and improved communications. Industrial development may take place as a result of parallel investment, mineral resources and hydro-electric power supply. Tourism will lead to employment opportunities, infrastructural developments and greater emphasis on conservation.	Increased population and possible unsustainable demand on food, forest and water resources. Reduction in food prices due to cheaper imports from other regions may reduce farm incomes and self-sufficiency in basic foodstuffs. Drop in employment opportunity if migrant workers remain in the project area. Import of manufactured goods may adversely affect local handicraft industries. Migration of young people to urban areas both within and outside the road corridor. Decline of rural populations and urban centres in the road catchment area. Loss of cultural and community values. Pollution: certain types of industrial development, such as mineral extraction, processing and logging, may lead to water pollution.

porated into the design process, an environmentally compatible design will be produced. Comparison of alternatives is usually achieved using inventories of the various impact categories identified. Some categories, such as number of buildings demolished and land use classification within the right of way, can be quantified and compared relatively easily, while others, such as cultural and ecological effects, are less amenable to objective and tangible comparison.

6.26 Some impact categories will be considered more important than others, and the allocation of a weighting scheme helps to clarify the total impact associated with each alternative. However, a weighting scheme is subjective and may provoke emotive debate. The choice between route corridors is not always clear cut, and which of the environmental factors should take precedence is often a matter of opinion. Short term economic and political reasons for favouring a route that clearly contradicts engineering judgement should be strongly resisted.

Mitigation and compensation measures

6.27 Once a route has been confirmed, mitigation and compensation measures should be identified to further enhance the integration of the scheme with its environment. The decision as to whether to proceed with construction will require that these mitigation measures are advanced at least to a preliminary design stage in order to establish their feasibility, overall benefit and likely costs. Mitigation measures that form part of the environmental protection programme but which are incorporated into the engineering design may need to be brought to the attention of the client in order to substantiate the apparent extra cost and head off possible accusations of `overdesign'.

Environmental management and monitoring

6.28 Environmental monitoring is the process of measuring changes in landscape, land use or cultural activity that arise as a result of the implementation of a development project. The Environmental Statement should outline the manner in which the project will be managed within the wider context of the study area environment, enabling maximum benefits to be derived from the scheme and allowing the performance of engineering design, construction, maintenance and mitigation measures to be monitored.

6.29 Environmental monitoring may form part of an environmental impact assessment, and indeed should do so in order to verify estimates of environmental impact made during the planning stages of a project. However, in practice this rarely happens. For projects where monitoring is seen as a practicable and desirable undertaking, rates of slope movement by failure, slope lowering by erosion and channel enlargement by scour and sediment transport can be measured by simple and low cost techniques.

6.30 Data collection in areas outside the sphere of influence of the scheme may allow firmer conclusions to be drawn regarding its environmental effects. However, with the possible exception of drainage and slope stability effects, the sphere of influence may be difficult to define, and comparisons may not be entirely valid. Monitoring will usually allow undesirable effects, not anticipated during project feasibility and design, to be identified, evaluated and mitigated as the project proceeds.

7 GEOTECHNICAL ASSESSMENT

INTRODUCTION

7.1 Geotechnical assessment comprises mapping, investigation and analysis and is central to the design and evaluation of the following:

- slope stability assessment, slope stabilisation and road erosion control
- cross-section and cut and fill slope angles
- sub-grade conditions
- stability and bearing capacity for retaining walls and foundation conditions at sites of major engineering structures
- materials suitability as fill, masonry, aggregate and road surfacing
- the identification, characterisation and quality specifications for material source areas
- slope drainage
- detailed horizontal and vertical alignment (as influenced by the above considerations).

GEOTECHNICAL MAPPING

7.2 The team `geotechnical mapping' is used here to cover engineering geological and geomorphological mapping procedures for recording and interpreting ground conditions for engineering purposes. Detailed geotechnical mapping is usually carried out only along a chosen route corridor identified from reconnaissance survey. Mapping scales of 1:2,500-1:10,000 are most common, although larger scales (1:500 - 1:1,000) are sometimes used at sites of major engineering structures or at higher risk landslide sites. The mapping should be accompanied by a report that summarises design and construction considerations.

7.3 The aim of geotechnical mapping is to convey information on surface and sub-surface ground conditions (so far as they can be ascertained) sufficient to satisfy the requirements for either preliminary or detailed design, while avoiding expensive ground investigation. It is usually most cost-effective to combine geological and geomorphological data on the same map, as slope stability assessments normally require both kinds of data. The range of factors usually considered is listed below and illustrated in Figure 7.1.

- Rocks:
  - rock type
  - outcrop pattern
  - location and trend of major geological structures
  - dip of lithology
  - dip of foliation
  - joint orientations
  - joint spacing
  - joint openness
  - weathering grade
  - permeability
  - estimated rock strength.
- Soils:
  - origin
  - colour
  - horizons
  - depth
  - grading
  - plasticity
  - density
  - strength
  - water, including inflow.

7.4 Other aspects to be considered include:

Factual information

- drainage features, such as springs and see pages, sinks, stream channels and areas of wet ground
- natural slope angles
- main topographical features, such as cliffs, terraces etc
- land use and details of irrigation
- the stability of slopes, with comments on mechanism, depth, extent and rate of movement of recorded failures, where known
- construction material sources including field descriptions and the locations of any samples taken for laboratory analysis
- the location and results of any field investigation and testing sites, such as trial pits, Mackintosh probe or Dynamic Cone Penetrometer (DCP)

Interpretation

- recommendations for stabilisation and erosion protection, where appropriate
- design and construction considerations, including foundation conditions in general, preferred cross-sections, bridge sites and abutment/pier foundation and stability, drainage design, river protection, construction access, pilot tracks and spoil disposal.

7.5 Teats containing standard rock and soil engineering descriptions and classifications are listed in Chapter 14

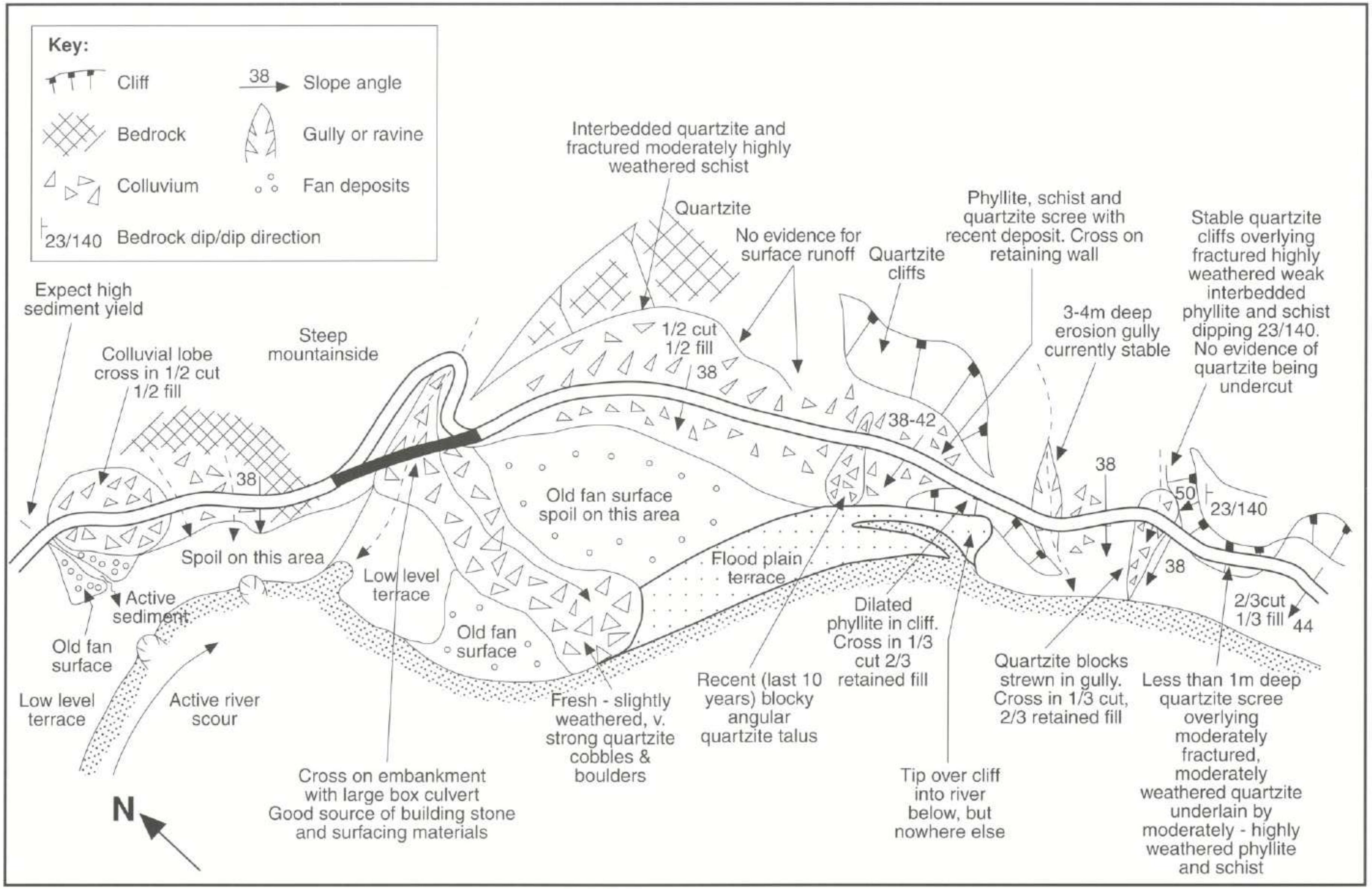


Figure 7.1 Geotechnical map for part of a proposed road alignment in Nepal

(Bibliography). No single classification system is applicable to all circumstances, and it may be that none is entirely relevant to a particular case without some modification. The classification adopted should be based on the range of conditions found, with modifications to the standard if necessary.

7.6 During construction and maintenance, geotechnical mapping can be combined with inventory surveys to further classify ground conditions for purposes of hazard assessment and slope management. An example from Papua New Guinea is given in Box 7.1.

GEOTECHNICAL INVESTIGATION

7.7 On low cost mountain roads, geotechnical investigations are usually limited to shallow sub-surface exploration using manual excavation and hand-portable equipment. Specialist drilling investigations are usually only cost-effective and practicable at major bridge sites, although they may have some application to high risk landslides along existing roads where access is comparatively straight-forward. The practical depth of hand-dug trial pits is limited by the increase in ground strength with depth, normally to around 2m. In some soils, 5m can be achieved, although shoring will be required to keep the pit open and maintain the safety of the pitting crew. Care must be taken in locating and digging deep pits in colluvium and unstable ground, particularly in the wet season or if the pit encounters ground water.

7.8 In addition to geological and geotechnical logging, the following data should be recorded during trial pitting:

- the location of the pit, sufficient to relate to the centre-line if known, by grid co-ordinates and elevation, or by position from a temporary bench mark or permanent ground marker

- the date and weather conditions prior to and during excavation

- the depth and horizon from which soil samples are taken

7.9 Hand-operated probing equipment, such as the DCP and the Mackintosh probe, can be used to obtain estimates of soil strength near the surface based on experience with the soil being investigated. The DCP is a useful means of measuring in situ CBR, while the Mackintosh probe can be correlated to Standard Penetration Test (SPT) N values to assess relative densities in the same soil types. The DCP can only be used to a depth of 1 m, unless it is advanced by trial pitting, while in soft ground the Mackintosh probe can be used to a depth of 6 or 7m. However, both probes are of limited use in stony ground typical of most colluvial soils that mantle mountain slopes. If they are to be used in these areas, it is advisable to excavate some trial pits to confirm that the probe readings relate to actual ground conditions. For example, an apparent increase in soil strength may be due to the presence of a boulder, or an increase in the stone content with depth, rather than a genuine increase in soil density.

7.10 In clayey soils (uncommon in mountain areas, except under localised residual weathering conditions), peak and residual undrained shear strength can be determined directly from vane shear apparatus and hand penetrometer, although, as with the probes mentioned above, the presence of coarse particles in the soil will significantly affect the results.

Trial earthworks

7.11 There is often advantage in excavating small trial cuttings which can be left open for one or more wet season(s). This technique is particularly valuable in areas of complex geology and weathering profiles, and allows deeper investigation from a working platform than from the confined area of a trial pit. The depth of a trial cutting should be similar to the expected design depth of cutting, and it should be at least as long as it is deep. The cutting face is cut to an angle considered appropriate for the material exposed. The stability of the cut slope is then logged and monitored. It is important to ensure that the trial cutting and generated spoil do not themselves lead to instability or erosion. Trial cuttings are probably best undertaken when the contractor is on site and is available to contain any such occurrence.

7.12 Similarly, trial embankments and trial compaction tests can be undertaken with control on materials, layer thicknesses, water content and compaction levels in order to compare performance with laboratory test data, and to better define specification standards to suit the conditions found on site. However, these trials require specific contractual arrangements that may be unavailable within a low cost road construction programme.

Sampling and laboratory testing

7.13 The standard procedures for soil and rock sampling and laboratory testing are well known and do not require elaboration here. However, the observations listed below are especially relevant.

7.14 For routine geotechnical design it is usual to cant' out index and classification tests on disturbed samples. Where material is to be considered for use as construction material, then compaction and strength (CBR) testing may also be required. CBR samples should be tested in a soaked condition for areas of high rainfall and/or high groundwater.

7.15 Rock exposed at outcrop or in shallow trial pits is generally more weathered than rock at depth. Thus, an assessment of rock for masonry and crushed aggregate

BOX 7.1 Geotechnical inventory and slope hazard / risk classification during and maintenance: an example from Papua Guinea

LOCATION Final section Confusion to Parrots Beak turnoff		**REFERENCE NO**	R125
		DATE	19/1/91
GEOMORPHOLOGICAL SLOPE SITUATION Limestone and siltstone slopes above landslide / erosion embayment of Ok Ningi.		**WEATHER**	Dry, sunny
		OPERATOR	G Hearn
DRAINAGE GROUNDWATER ETC. Horizontal drains installed in toe of slope, dry at time of survey.		**LANDUSE**	Access road

	SLOPE (m) LENGTH	AVE (°) ANGLE	MAX ANGLE (°) & HEIGHT (m)	ASPECT (°)	SLOPE (m) HEIGHT	% VEG	% ROCK OUTCROP
CUT SLOPE	15	50	-	080	11	50	30
NAT SLOPE	200	35	-	080	220	80	?

ROCK TYPE: Highly fractured loose siltstone exposed in cut. General FS = 0.05m. Conglomerate on upper slopes and local outcrop of relatively massive limestone.

SOIL TYPE / DEPTH: Loose clayey silt with subangular siltstone gravel. 0.5m thick zone of grey and brown soft-firm silty clay developed as weathering product.

DISCONTINUITIES	1	2
DIP ANGLE	60	23
DIP DIRECTION	140	010
PERSISTENCE	0.1	Bedding
WIDTH	Closed	0.002
SPACING	0.4	0.05
ROUGHNESS	3	3

WEATHERING GRADE III

ROCK STRENGTH 5

STRUCTURES / BEDROCK ASSOCIATIONS

Fault crops out in cut at northern end of section 57/150.

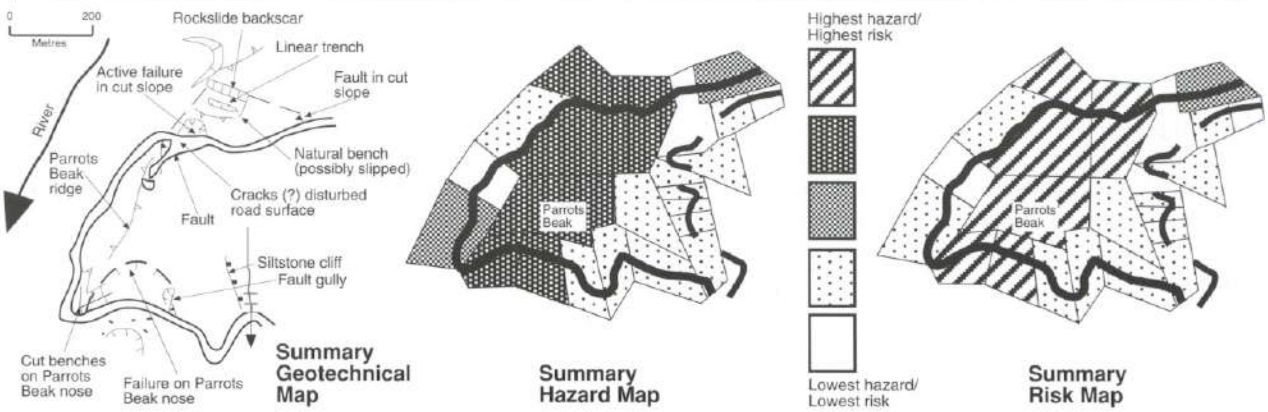
CONDITIONS DOWNSLOPE: 33 - 35° slopes broadly concave towards the river below local seepage gullies developing at approx 50m below road. At 60 - 100m below road approx N - S line of erosion scars with active gullying in loose colluvium (siltstone and limestone in clay matrix) over an area of 80X60X4m deep.

GEOMORPHOLOGICAL FEATURES: When viewed from Parrots Beak, slopes above cut appear to form an ill-defined bench - structural or landslide (?). Ground beginning to crack open on 35° slopes above road. At approximately 60m above road slopes flatten to 20° on top of bench with a trench oriented N - S, 2-3m deep, and 3-4m wide, 20m long. Approx 30m above the trench is a 10-20m high 60-70° landslide scar formed in highly fractured siltstone oriented N - S. Immediately above the scar is a narrow spur separating the back scar to the east (described above) from a steep landslide embayment to the southwest.

STABILITY: Large rockslide in siltstone above road. Total anticipated failure volume from cut slope 100X70X30m (including R126). Potential deep seated rock failure involving R125, R126, R127 and leading eventually to road loss.

HAZARD TYPE	SIZE (MxM)	DEPTH (M)	STABILITY	CAUSE	CURRENT IMPACT	POSSIBLE FUTURE IMPACT	INFERRED RATE
a) Deep (>2m) slope erosion (below road)	80 X 60	4	Active	Seepage and loose materials	None	Might eventually erode back to, or undercut road	1m / yr
b) Debris slide (cut slope)	50 X 50	5	Creep	Possible shallow failure within rock slide mass	Small failures in R126 cut slope	Temporary road blockage	mm / week
c) Rock slide (whole slope)	100 X 70	30	Creep	Unknown	Small failures in R126 cut slope	Could be linked to Parrots Beak failure and could dislocate or remove road	mm / week

Performance of existing stabilisation / protection measures	Comments / Recommendations	Operator Site Hazard and Risk Classification Hazard (H) = size (S) X probability (P) Risk (R) = H X vulnerability (V_u) X value (V_a)
Horizontal drains in cutslope	Review every 6 months. Geotechnical review of Parrots Beak area.	a) S = 2 P =2 V_u = 1 V_a = 3 (access road) b) S = 2 P =2 V_u = 2 V_a = 3 (access road) c) S = 2 P =2 V_u = 3 V_a = 3 (access road)



Summary Geotechnical Map

Summary Hazard Map

Summary Risk Map

made during walk-over survey may underestimate the strength and durability of material at formation level or when excavated from a quarry.

7.16 The requirement to carry out undisturbed sampling from trial pits is usually limited to the rare occasions when the result of the analysis is likely to cause a modification of the design. The engineer must have confidence in the sampling procedure, the test method and the capability of the laboratory to carry out the test properly. The practical problems in obtaining undisturbed samples from coarse-grained colluvial soils or partially saturated residual soils, in remote areas especially, are usually prohibitive. Undisturbed sampling from borehole investigations allows a much wider range of tests to be carried out, particularly with regard to shear strength and compressibility. However, core recovery in soil or weathered or fractured rock will be low or nil, unless sophisticated methods are employed. Undisturbed rock samples are generally only required for foundation design at major bridge sites or at the sites of high risk slope failure.

7.17 If good core recovery is obtained, it can provide a wealth of information on drainage, rock strength and weathering, fracture spacing and orientation, faults, crushed rock zones and shear zones. However, careful logging by geotechnical personnel is usually required in order to differentiate between different soil and rock types in areas of complex geology, including the identification of thrust materials, landslide colluvium, rock avalanche materials and crushed bedrock.

Geophysics

7.18 Geophysics is used to detect variations in seismic velocity and other physical conditions brought about by changes in strata or groundwater and the presence of underground cavities. The principal advantage of a geophysical survey is that it can record these conditions from the ground surface at relatively low cost compared with conventional ground investigation. Its main disadvantage is the difficulty associated with the interpretation of complex ground conditions, especially where subtle changes may go unrecorded. Geophysical data should be supported by borehole information to confirm the results and assist in their interpretation.

7.19 Seismic refraction is generally the most frequently used geophysical technique for engineering applications but it will be reliable only if:

- seismic velocity is constant, or increases with depth. This is not always the case in landslide materials, where there may be a low velocity along the slip surface
- there is a distinct boundary and a marked difference in velocity between the layers being investigated
- boulders are absent from the profile. The presence of large boulders complicates the interpretation of results.

7.20 Although, geophysical survey generally has little to offer low cost road design in mountain areas, there may be some potential for its uncorroborated use in situations such as:

- the detection of the boundary between comparatively fine-grained colluvium and an unweathered rock head surface
- the definition of distinct sequences of river deposits
- the detection of underground cavities and other karst features developed in calcareous rocks and deposits.

Logging of excavations as construction proceeds

7.21 The properties of materials can only be properly evaluated during construction. As excavation proceeds, observed ground conditions should be compared with those assumed in the design, and any necessary modifications implemented. For instance, where a soil profile is more weathered than originally thought, or where materials are found to be colluvial rather than residual, it may be necessary to reduce the cut slope angle, deepen a foundation excavation or widen a wall cross-section to accommodate the weaker materials.

7.22 Some geotechnical parameters that may not be fully evaluated until construction is underway are listed in Box 7.2.

GEOTECHNICAL ANALYSIS AND DESIGN

Introduction

7.23 In the context of mountain road design, geotechnical analysis is used principally in evaluating slope stability for the design of earthworks and slope stabilisation schemes, and in the assessment of stability and bearing capacity for retaining wall foundations. This section deals mostly with slope stability aspects of geotechnical analysis; foundations for retaining walls are discussed in Chapter 11.

Assessment of soil strength parameters

7.24 In order to carry out any analysis, it is necessary both to assess the strength and depth of the materials forming the slope and the most likely worst-case groundwater and soil moisture conditions. Differentiation between colluvium and residual soil is of major importance owing to the differing strength characteristics of these two soil types.

7.25 Colluvial soils are usually coarse-grained, and their approximate strength and other geotechnical characteristics can be assessed either from the results of SPT or other

Box 7.2 Geotechnical evaluation cannot normally be finalised until construction

Geotechnical factors that are usually not fully evaluated until construction is under way include:

- soil depth and weathering grades at the soil/rock head interface
- the location of weaker and more weathered zones
- mineralogy (the presence of mica, chlorite and graphite, for instance, may increase the potential for failure)
- the distribution and orientation of rock discontinuities
- a classification of rock fabric microstructure and rock mass macrostructure
- the presence of pre-existing shear surfaces (although these may be difficult to identify even in excavations)
- soil moisture and drainage conditions
- bearing capacity and mobilised shear strength of in-situ soils
- sub-surface drainage conditions and groundwater fluctuations
- suitability of excavated materials as aggregate or masonry
- susceptibility of soils to erosion following disturbance by earthworks and artificial slope drainage
- susceptibility of marginally stable slopes to disturbance by construction.
- precise drainage and land management requirements where alignments pass through areas of irrigated terracing
- costs and performance of alternative forms of slope stabilisation and erosion protection works.

probe tests, or from their particle size. In fine-grained (usually weathered) colluvial soils, index tests (Atterburg Limits and natural moisture content) can provide a general indication of shear strength; a relationship between the effective internal angle of friction (ϕ') and Plasticity Index (PI) for remoulded clays can be used to estimate 0'. In mountainous areas, clayey colluvial soils are generally of very low plasticity with values of ϕ' in excess of 30°.

7.26 Residual soils, usually containing a higher proportion of clays, often occur on flat ridge tops and structural benches where in situ weathering has predominated over erosion and leaching of clay minerals. Unlike colluvium, these soils have not been remoulded and can exhibit an additional cohesion associated with the residual rock structure, thus increasing their strength. In these soils the PI/ϕ’ relationship may not apply, and ϕ’ values so derived would be conservative in terms of cut slope stability.

7.27 Soil suction can play an important role in controlling the stability of slopes, enhancing the shear strength of the soil by an apparent cohesion. This is more pronounced in the generally finer grained residual soils than coarser colluvial materials. An increase in degree of saturation, by infiltration during rainfall, by seepage, or by a general rise in groundwater level, will lead to a reduction in soil suction and a corresponding decrease in strength and stability. Road construction can significantly alter the pattern of slope drainage and soil saturation, thereby modifying the susceptibility of adjacent slopes to failure. Soil cementation, caused by the secondary deposition of iron oxides and carbonates in groundwater can also significantly increase soil cohesion.

7.28 Any assessment of soil strength based on empirical correlations with grading, PI or probe results, must involve a degree of uncertainty. Frequently, a factor of safety of unity or less for apparently stable slopes can be obtained from stability analysis using strength parameters derived in this way. This may be due to a number of effects, including soil heterogeneity, or the suction, cohesion or cementation effects outlined above not being fully appreciated. These factors can be assessed by back analysis of existing slopes, preferably those that have suffered failure.

Slope stability analysis

Methods of analysis

7.29 Essentially, there are three methods that may be used to analyse a slope:

- assumption of an infinite slope
- slope stability charts
- computer programs

7.30 The infinite slope model allows planar failures to be analysed quite readily without recourse to computer programs, by assuming that in purely frictional soils (no cohesion) the shear strength can be represented by a single value of ϕ'. At the point of failure, when the factor of safety is equal to unity, the following condition applies:

$$\tan\phi'.(1-r_u.\sec^2\alpha.) = \tan\alpha:$$

where

r_u = pore pressure ratio (water pressure/total overburden pressure)

ϕ' = internal angle of friction under effective stress

α = slope angle at failure.

7.31 The value of r_u is zero when groundwater is below the slip surface and approximately 0.5 when at the slope surface. Intermediate groundwater levels, where known, can be assigned intermediate r_u values accordingly. Thus, ϕ' equates approximately to the natural slope angle at failure in the absence of water, and twice the natural slope angle at failure for water at the slope surface (the importance of identifying groundwater levels in the slope can be readily appreciated). However, it is rare that the profile of the slope can be approximated to a uniform gradient and it is common to find surface soils failing on a shallow bedrock surface parallel to the ground slope. This method therefore has limitations.

7.32 Stability charts, relating soil strength parameters, r_u and factor of safety to slope height, were widely used before computers became more widespread, and can be useful for rapid assessment of stability. An assumption has to be made both on slope height and r_u. value, both of which can be difficult in steep and complex topography. For instance, the ground slope above a proposed cutting is very rarely flat (as is assumed in most stability charts) and it may be necessary to assume a greater height of cutting at an intermediate cutting angle. Sensitivity analyses will be necessary to identify the worst case combination of geometry and r_u. Charts are available for varying groundwater profiles rather than r_u values, and these can be useful in some cases (TRL. 1997).

7.33 Design charts require the following parameters to be known or assumed:

- effective cohesion (c')
- effective internal angle of friction (ϕ')
- slope height (H)
- groundwater conditions (based on r_u)
- required factor of safety.

7.34 Where a slope geometry or groundwater profile cannot be suitably represented by chart analysis, recourse will have to be made to computer programs.

7.35 A large number of computer programs are now available to determine slope factor of safety. The use of a computer allows the precise ground slope profile to be incorporated, together with any variation in groundwater profile. Thus, the individual slope characteristics, if known, can be accurately represented. Programs also allow a large number of potential slip surfaces to be considered rapidly; some allow random generation of non-circular surfaces, the most prevalent form of failure on steep slopes formed in granular and heterogeneous soils.

7.36 Computer programs allow the use of either a specified groundwater level or an r_u value, or both. The assumption of a single groundwater level, above which all slope materials are dry is an oversimplification in most situations. Some computer programs enable perched water tables to be analysed, that represent seepage horizons superimposed on a deeper water table. A reasonable representation of groundwater conditions can be approximated by combining observed or assumed water tables with a knowledge of sub-surface variations in geology and soil permeabilities.

7.37 The concept of r_u was first proposed when the majority of design was undertaken using charts. The use of a single ratio between water pressure and total vertical earth pressure at every point along a slope is a crude model of the actual variation in water levels, although it may be the most appropriate means of modelling water pressures where see pages are giving rise to more localised partial or total soil saturation, or where groundwater is not hydrostatic. Furthermore, computer modelling provides the means by which a slope cross-section is divided into discrete zones, each with its own set of soil parameters, including r_u . Nevertheless, whilst there may be no choice but to use r_u where no other information on groundwater is available, use of an observed or assumed water profile, perhaps parallel to rock surface, is usually preferable.

Back analysis

7.38 Analysing a slope by varying soil strength parameters and groundwater levels within plausible limits to obtain a calculated factor of safety of unity, enables an appreciation of likely in situ strengths and water levels which can be used in design. Identification of unfailed bedrock levels assists in limiting the extent of trial failure surfaces and the range of failure mechanisms considered in the analysis. Parameters so derived can then be used for the design of new slopes, or for examining the stability of existing hillsides.

7.39 In steep terrain, it is possible to assume that an intact slope has a factor of safety close to unity and to derive soil parameters, but the calculated values would be conservative as the actual factor of safety must be higher than assumed (but to an unknown degree). Nevertheless, the derived parameters can often be greater than would otherwise be obtained from testing or empirical assessment, and can lead to more economic

7.40 To back analyse an intact slope, it is necessary to assume the potential failure surface. This is greatly helped if the depth to rockhead or other competent material is known, either by mapping or by investigation. It is then necessary to undertake a sensitivity analysis to assess the most likely failure mechanism. The ranges in both groundwater levels and soil strengths also need to be assessed. Information on groundwater levels from standpipes installed in boreholes is useful, but it is rare for worst-case groundwater levels to be recorded, unless the monitoring periods spans a number of wet seasons. Worst-case groundwater levels, therefore, usually have to be assumed.

7.41 In the case of failed slopes, there is no doubt as to the slope having reached and passed the point of limiting equilibrium. However, difficulty remains in determining the failure surface and the groundwater conditions at the time of failure, particularly where stability is immediately improved by a release of groundwater or a significant reduction in slope angle when coming to rest. Ground investigation (boreholes, trial pits and trenches) may allow soil type, soil horizons and depths to rock head to be determined, but unless failure has occurred on an obvious surface, or variations in rock strata or foliation can be used to infer failure depth, it can be difficult to determine the exact depth of failure from ground investigation data alone. Furthermore, an estimate has to be made of the likely range in water level, with a considered view taken as to the probable water profile upon which to base the analysis. These considerations are illustrated in Box 7.3 with respect to recent geotechnical investigation, analysis and design in Nepal.

7.42 Where a slope failure continues to have a factor of safety close to unity, it will normally continue to fail on a regular basis, and consequently failure depth and groundwater conditions can be modelled more closely.

Slope design

7.43 Some form of analysis will be required for routine checks on the stability of earthworks, with more comprehensive analysis required where deep cuts or landslide stabilisation are envisaged. Vegetation can have a significant influence on the factor of safety, but its effects are not easily quantifiable and it is safer to exclude them from design calculations, unless planting or revegetation is specifically designed to achieve an acceptable factor of safety.

7.44 In carrying out this analysis, it is prudent to test the sensitivity in the factor of safety to small variations in the value of soil strength parameters and water levels. This will identify whether a particular slope design is more sensitive to some of the assumptions made than others, and it may affect the final choice of parameters.

7.45 Although a computer program may generate failure scenarios and factors of safety that appear sensible, it is extremely important to ensure that ground conditions at failure suggested by the computer give a realistic picture of those that exist at the site. This may seem an obvious observation, but only too often a computer result is taken to be infallible, where in truth the conditions that it has modelled bear little relation to actual ground conditions.

7.46 Difficulty often lies in the choice of factor of safety to be achieved in the design. As the majority of adjacent slopes are likely to be near to a factor of safety of unity, it may be difficult to justify factor of safety values much greater than this, as might be expected in less severe terrain. Also, it is frequently very difficult to achieve high factors of safety without prohibitively expensive engineering works.

7.47 The choice of factor of safety will depend upon the reliability of the soil strength data and groundwater profile used, together with the consequences of failure should it occur. It is also dependent on whether the slope parameters chosen are average, conservative or worst credible. Each slope must be considered as an individual case. A factor of safety of 1.1 is reasonable for most lightly-trafficked roads, where the consequences of slope failure are relatively low. Where the consequences of failure are high, such as in the situation where road loss could cause significant traffic disruption, then a factor of safety of 1.2-1.3 might be more appropriate. The philosophy of slope design and the choice of factor of safety is discussed further with respect to cut slopes in Chapter 9.

Empirical methods

7.48 An examination of the relationship between the geometry, groundwater and materials characteristics of natural slopes can enable stable and unstable slope scenarios to be identified. This information can then be used to assess the stability of both new and existing slopes.

7.49 Table 7.1 is based on observational data from Nepal, and shows the range of limiting slope angles for the common soil types found there. The dry and wet cases for colluvial silts and sands correspond approximately to the $r_u = 0$ and $r_u = 0.5$ conditions respectively, mentioned earlier. The limiting slope angles for residual soils and sands are significantly higher, due to the presence of cohesion or suctions.

7.50 Near-surface throughflow, often induced by a shallow rockhead surface, can give rise to locally perched groundwater and r_u values of 0.5. Mudslides in clayey silt residual soils have been found to fail across smooth bedrock surfaces on slopes as low as 12°.

Box 7.3 Examples of geotechnical investigation, slope stability analysis and design from Nepal

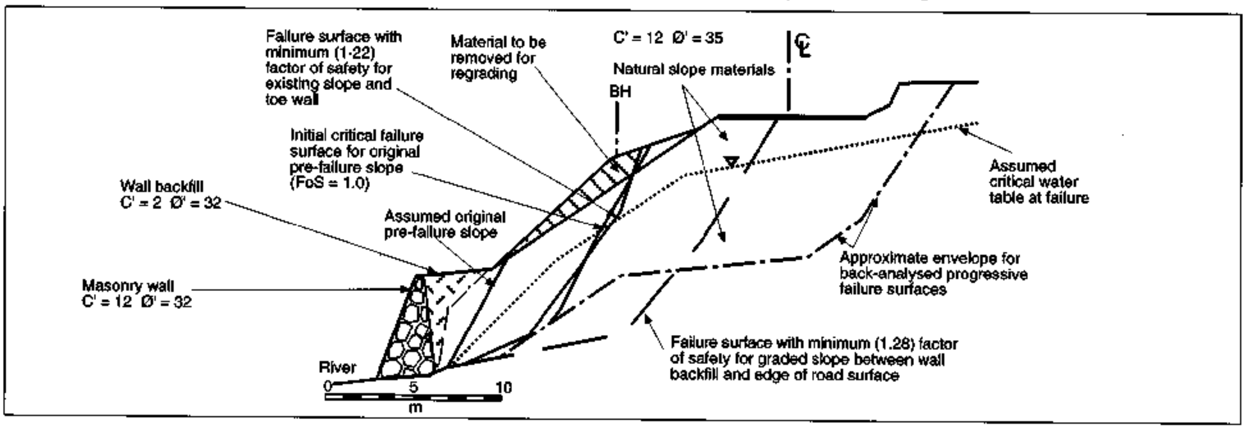
During heavy rains in July 1993, flooding, scour and slope failure caused significant damage to the two main roads that link Kathmandu with the majority of the country and India to the south.

SITE A: Severe road settlement and deformation occurred over a length of 80m at this locality due to slope failure caused by toe erosion and high groundwater tables. Near-vertically dipping phyllites and sandstones, that strike approximately parallel to the course of the stream at the base of the slope, probably impeded water movement downslope, thus locally raising the water table.

Investigation: Mapping of the strata orientations helped identify the failure mechanism as a progressive topple and collapse, probably with a very irregular failure surface. A total of 7 drillholes were put down for purposes of stability analysis and foundation investigation. The drillholes encountered groundwater at a depth of between 8 and 11m, although this was presumed to be significantly lower than the level at the time of failure. Inspection of the drill core revealed a change in strata dip angle at 6-7m below ground level. Furthermore, core recovery increased significantly below 6-8m.

Analysis: One of the analysed cross-sections is shown below. A knowledge of the failure mechanism and the likely depth of failure enabled sensitivity analyses to be carried out, using a factor of safety of 1.0, to define the most likely combination of parameters at the point of failure. Cross-sections were re-analysed with a front slope formed along the preceding critical failure surface in order to simulate progressive failure, until a factor of safety significantly greater than 1.0 was obtained. Although the investigation and analysis were judged to have been successful, the computed maximum failure envelope corresponded with the failure depth determined by investigation in only one of the two cross-sections analysed, reflecting the difficulty in modelling complex topple failures using standard limit equilibrium methods of analysis.

Design: With the existing masonry toe wall, constructed as a scour protection and revetment structure immediately after the failure, together with slope grading, an average factor of safety of 1.3 was considered acceptable for design.



SITE B: Here the road was cut for approximately 60m of its length by a progressive failure in a previously failed mass of completely weathered slate and conglomerate colluvium. From surface mapping the mechanism of failure appeared to be predominantly shallow planar, and was caused by toe erosion and a combination of high groundwater and seepage pressures.

Investigation: As stabilisation measures were required to be largely in place before the onset of heavy rains, the investigation, by necessity, was carried out during the intervening dry season, and therefore even "normal monsoon" groundwater levels could not be recorded for analytical purposes. Three boreholes were put down along the longitudinal centre of the landslide. All three failed to encounter bedrock or to identify materials that could be interpreted confidently as the failure surface(s). Attempts to recover undisturbed samples proved unsuccessful due to the extremely heterogeneous nature of the materials encountered.

Analysis: The analysed cross-section is shown below. The only failure conditions that could be represented with any degree of confidence prior to sensitivity analysis were the topographic slope of the failure and the depth to what was interpreted as unfailed bedrock in the central borehole that was advanced by rotary drilling. Mapping evidence was used to define seepage horizons, while sensitivity analyses and a factor of safety of 1.0 enabled the most likely failure surface, strength parameters and critical groundwater table to be identified.

Design: The design compared the geotechnical benefits and costs of a toe wall (with backfill, slope trimming and drainage works) with those of an anchored contiguous bored pile or caisson wall at road level. The toe wall with drainage (shown on the cross-section) was chosen to be the most cost-effective, offering a minimum computed factor of safety of 1.25.

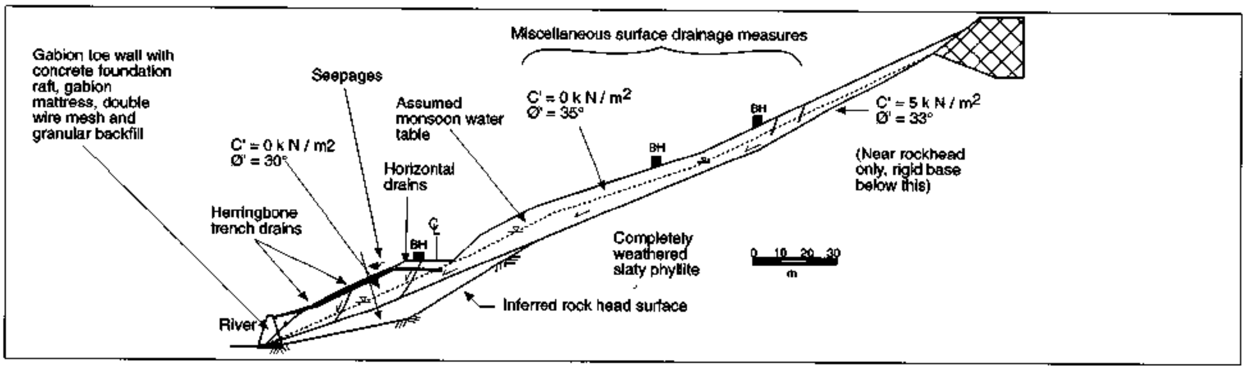


Table 7.1 Range of limiting angles (in degrees) for natural slopes (based on observational data from east Nepal)

Soil Type	*Residual soil*		*Colluvial soil*		*Perched water table*
	Dry	Wet	Dry	Wet	Wet
Clayey silt	33-36	16-17	28-31	14-15	11
Silt	33-36	16-17	28-31	16-17	12
Sandy silt	33-36	16-17	31-34	16-17	14
Silty sand	36-39	19-20	31-34	16-17	17
Silt & boulders	36-39	28-29	31-34	23-24	19
>50% boulders	36-39	31-32	33-36	23-24	19

NB Dry Soils: groundwater below potential slip surface
Wet Soils: groundwater above potential slip surface
Perched: groundwater seepage at slope surface.

Rock slope stability analysis and design

7.51 The analytical and design approach adopted for rock slopes is dependent upon depth of cut, a qualitative assessment of its stability, and the consequences should failure occur. For cut slopes less than 10m in height it is usual to rely on engineering geological judgement and observational data. Table 7.2 has been compiled from cut slope inventory data gathered in east Nepal, by way of illustration.

7.52 For deeper rock cuts and unstable rock slopes, or those where the consequences of failure are severe, engineering geological mapping of rock type, rock discontinuity orientation, waviness and roughness, spacing, persistence, size, infill, rock weathering grade, groundwater and see pages will be required to build up a picture of the structural and mass strength of the rock slope. Identification and recording of existing rock failure mechanisms on surrounding slopes can give an indication of the most likely mode(s) of failure.

7.53 It is usual to analyse recorded discontinuity orientations by means of stability charts and sternest, and usually with the aid of a computer database and graphics. Stereonet analysis will indicate the most likely joint orientations upon which failure will take place. Although planar slides and rock topples are frequent mechanisms of rock slope failure in mountain areas, it is usual for a stereoplot of joint orientations to indicate that wedge failure, along intersecting joint surfaces, is one of the most likely mechanisms by which a slope will fail. The results of the stereonet analysis can then be used in conjunction with field data to judge the likelihood of failure in excavations cut to different slopes and orientations, and the possible extent of failures should they occur. Separate analysis can be carried out for different cutting angles.

7.54 Depending upon the consequences of failure, and the confidence placed on the analysis, further geotechnical investigation and slope stability analysis may be required. However, due to the costs involved to reducing geotechnical uncertainties, sophisticated methods of analysis and design are comparatively rare on low cost mountain roads.

Table 7.2 Range of maximum slope angles for cut slopes in rock (based on observational data from east Nepal)

	Cut slope height (metres)			
Rock type	≤5	6-10	11-15	16-20
Mudstone/shale	50-80	50-75	45-65	40-60
Phyllite	60-85	60-80	50-75	50-70
Schist	60-85	60-80	60-80	60-75
Gneiss	50-80	40-75	Insufficient data	
Quartzite	60-75	60-75	60-75	60-75

Note Each rock type has been given a range of maximum cut slope angles for each of the four categories of cut slope height. The range refers to materials of low and high composite strength respectively (ie a combination of groundwater condition, joint/foliation orientations and weathering grade). This table is for illustration purposes only.

8 HYDROLOGY AND HYDRAULIC DESIGN

INTRODUCTION

8.1 Hydrological analyses of rainfall and runoff data are required to establish relationships between flood flow and flood frequency for hydraulic design, for which the following steps are taken:

- classification of risk elements within the road drainage system (road side drainage, small culverts, large culverts, short span bridges and major bridges)
- allocation of a conceptual design life for each risk element based on the accepted probability of the element not being surcharged or overtopped during this period
- calculation of the discharge of the flood that corresponds to this return period.

8.2 Unfortunately, in most remote areas the data needed to carry out these calculations are often extremely limited in terms of geographical spread and length of record. Rainfall and flood frequency analyses must be combined to yield an aggregate assessment and corroborated with field evidence, where possible, before the results can be confidently used for hydraulic design.

DESIGN LIFE

8.3 An evaluation of what is an acceptable design life for each category of road drainage structure will allow investment in construction design, maintenance and eventual replacement to be apportioned in the most cost-effective manner. In the case of side drains, culverts and short sections of road in flood-prone areas, this usually comprises 1-10 year events that are accommodated in design, 10-20 year events that are accommodated in design and maintenance, and 20-50 year events that are usually accommodated by reconstruction. 50-100 year events and greater are only accommodated in design by major structures and by the rest of the road where possible in adopting the safest available alignment. Defining the conceptual design life of any given hydraulic structure is comparatively straightforward; the main problem concerns the identification of the storm or flood magnitude with the equivalent recurrence interval.

8.4 Even if a reliable set of flood magnitude-recurrence interval relationships can be developed, there are other factors to consider:

- the very nature of probability theory with respect to flood-frequency (Table 8.1) means that the structure may not receive its design discharge throughout its life, or it may receive a large excessive discharge in the first year following construction
- the very localised nature of orographically-controlled rainfall can mean that a high magnitude storm may occur over one catchment, while an adjacent catchment receives little or no rain. The recurrence interval of the high magnitude storm may be long, but the following year a storm of similar size could occur over a neighbouring catchment. The recurrence interval of these storms, as measured by individual raingauges, could be correct (ie each raingauge might record a one in twenty year storm on average every twenty years), but a road traversing these catchments would experience both storms and an apparent decrease in recurrence interval (increase in storm frequency) over its length. This introduces the complex subject of combined probability. The combined probability of flood damage or road closure at any one of many cross-drainage structures is far higher than the probability at any individual site
- rainfall intensity will vary significantly throughout the duration of a storm, and between one catchment and the next, while floods of similar volumes in the same river can be accompanied by markedly different velocity distributions and hence scour potential due to variations in the flood hydrograph or changes in channel cross-section. Rainfall intensity and maximum flow velocities are among the most important parameters in hydraulic design, and yet are the most difficult to evaluate in terms of design life
- drainage structures in mountain regions will have to resist scour and transmit debris every year and failure to do so will damage or destroy them more effectively than a hydraulic surcharge. It is apparent from experience in mountainous terrain that culverts and bridges are rarely subjected to surcharge or overtopping before the foundations are undermined by scour or the waterway is blocked by debris, both of which can occur during the course of a single flood
- due to variable rainfall-runoff relationships, a storm of a given recurrence interval will rarely give rise to the same recurrence interval flood.

8.5 The concept of a design life for a hydraulic structure is therefore difficult to apply or to achieve in practice when there are so many other factors to consider. Nevertheless hydrological analysis should proceed through some and occasionally all of the following logical stages:

- choice of design life
- magnitude - frequency analysis of rainfall data
- rainfall - runoff conversion
- review of regional flood formulae
- flood frequency analysis
- slope area method and field verification.

Table 8.1 Percentage risk of encountering an event within a particular design life for different return periods

Design Life (L) (years)	Return period T (years)								
	5	10	20	30	50	100	200	500	1000
1	20	10	5	3	2	1	-	-	-
2	36	19	10	7	4	2	1	-	-
3	49	27	14	10	6	3	1	-	-
5	67	41	23	16	10	5	2	1	-
7	79	52	30	21	13	7	3	1	1
10	89	65	40	29	18	10	5	2	1
15	96	79	54	40	26	14	7	3	1
20	99	88	64	49	33	18	10	4	2
30	-	96	78	64	45	26	14	6	3
50	-	99	92	82	64	39	22	9	5
75	-	-	98	92	78	53	31	14	7
100	-	-	99	97	87	63	39	18	10
150	-	-	-	99	95	78	53	26	14
200	-	-	-	-	98	87	63	33	18
300	-	-	-	-	-	95	78	45	26
500	-	-	-	-	-	99	92	63	39
1000	-	-	-	-	-	-	99	86	63

Note: Percentage risk is calculated from $R = 100\left(1 - \left(1 - \frac{1}{T}\right)^{L}\right)$

8.6 Despite these logical steps in the hydrological analysis, flood frequency records, if they exist, are likely to form the most reliable basis for design.

RAINFALL ANALYSIS

8.7 Although rainfall is usually recorded far more frequently than river flow, the distribution of raingauges in mountain areas is often grossly inadequate to record anything other than the regional pattern in rainfall variation. Even so, a raingauge spacing of as little as 1km may still be inadequate to record storm rainfall variability in some areas, brought about by orographic and rain shadow effects. Usually, the only practicable approach will be to identify broad areal groupings of rainfall data in order to classify hydrological regions. This regionalisation of rainfall for design purposes is not as great a problem as might at first appear, because the economic implications for underdesign (or overdesign) increase as the size (and usually catchment area) of a given hydraulic structure increases. The fact that a dense network of raingauges might not exist, therefore, may not necessarily adversely affect the validity of the hydraulic design.

8.8 Localised intense storms, giving rise to the surcharge of one or a small number of culverts, are bound to occur from time to time. These storms may have significant effects in individual catchments, but their impact on large hydraulic structures further downstream in the drainage system will be minimal. The river flow regime at these larger structures will be determined instead by the rainfall pattern in the larger catchment or hydrological region as a whole. Therefore, as long as a regional network of topographically-representative rain gauges exists, it is length of record, data validity and recording interval, that become the chief concerns.

8.9 Most raingauges in remote mountain areas are operated manually and record rainfall on the basis of 24 hour totals only. 24 hour data provide little indication of rainfall intensity or the duration-frequency characteristics of intense rain that are most important for hydraulic design. For instance, the same 24 hour rainfall depth can indicate:

- prolonged rainfall of moderate intensity
- short duration rainfall of high intensity
- a portion of a total storm rainfall that was still in progress when the raingauge was read.

8.10 This problem can be partially overcome by using rainfall ratios or by comparing 24 hour rainfall data with the intensity and duration relationships obtained from continuously recording raingauges nearby or in the same hydrological region (see below). The usual procedures for analysis of both 24-hour and continuous rainfall data are summarised in Figure 8.1. The method by which recurrence interval is calculated from 24-hour rainfall records is outlined in Box 8.1. Selection of the 24-hour annual maximum rainfall may exclude a number of other large storm rainfalls from a particularly wet year that may not be repeated during following drier years. This can be overcome by analysing partial series data that include rainfalls larger than a certain threshold.

8.11 Continuous rainfall data are usually analysed and presented in the form of intensity-duration-frequency curves, such as those shown in Figure 8.2. The rainfall intensity with a 24 hour duration can be calculated for each recurrence interval and compared with the corresponding intensities derived from the 24 hour annual maximum regression. An adjustment factor can then be used to derive intensity for durations of less than 24 hours from other daily rainfall in the same hydrological region.

8.12 In the absence of any continuous rainfall data, it may be necessary to assume an effective rainfall duration for hydraulic design purposes and refer to published rainfall ratio tables to identify suitable values for the constants contained in the following equation for rainfall ratio over a period (RRt):

$$RRt = \frac{\text{Rainfall intensity for duration t}}{\text{(Rainfall intensity over 24 hours)} * (24)} = \frac{t}{24}\left(\frac{b+24}{b+t}\right)^{n}$$

where b and n usually range between 0.2 and 0.5 and 0.5 and 1.0 respectively in the humid sub-tropics.

8.13 Design rainfalls derived in this manner must be treated with extreme caution and it is probably safer to combine results with worst case, short-duration rainfall intensities based on experience from other regions with similar climatic conditions. For example, an intensity of 120mm/hr for a 30 minute duration is often considered to be acceptable for the design of culverts for small steep catchments.

RAINFALL - RUNOFF ANALYSIS

8.14 Figure 8.3 illustrates common procedures for design discharge modelling. The simplest and most frequently used rainfall-runoff technique is the Rational Method:

$$Q = 0.277CIA$$

where

Q is peak discharge in cumec

C is runoff coefficient

I is rainfall intensity in mm/hr

A is catchment area in km^2

0.277 is an arithmetic conversion factor that allows the above parameters to be expressed in the conventional units.

Box 8.1 Calculation of storm recurrence internal from 24-hour rainfall data

At least 10 years of rainfall record is generally required for annual series analysis. The highest 24-hour rainfall recorded in each year of record are ranked in order of magnitude and a recurrence interval is calculated for each rainfall based on the following formula:

$$T = (N+1)/m$$

where

T is recurrence interval in years

N is the number of years in the series, and

m is the rank of each 24-hour record.

The probability of a rainfall with a given recurrence interval occurring at least once in N years is expressed by the equation:

$$P = 1\text{-}q^{N}$$

where q is equal to the probability of the rainfall not occurring in any particular year.

The recurrence interval (T) is plotted on a logarithmic scale against maximum 24 hour rainfall on a normal scale using Gumbel extreme value type 1 or Pearson type iii paper. Linear regression is then used to define the relationship between recurrence interval and peak 24 hour rainfall. It is worthwhile tabulating the 2-1 hour rainfalls calculated for each raingauge with 2 year, 2.33 year (approximate average annual rainfall), 5 year, 10 year, 20 year, 50 year and 100 year recurrence intervals. The mean and standard deviation between raingauges will assist in defining hydrological regions within the study area.

8.15 Because of the simplifying assumptions on which it is based, the Rational Method is most applicable to small catchments (usually of the order of $1km^2$). This size limitation is especially relevant to mountain areas where storm rainfall can be unevenly distributed within catchments much larger than this, and thus the runoff response is not catchment-wide, as the model implies. The value of rainfall intensity used in the calculation should not be the instantaneous peak, but the maximum average sustained during a period equivalent to the time to concentration of the catchment. The catchment time to concentration is defined as the time required for rainfall falling in the remotest part of the catchment to reach the catchment outlet, or culverting point, and can be calculated from one of a number of equations. It is typically well below 1 hour for very small, steep catchments. It is the variability in C, both in time and over short catchment distances, together with difficulties in determining design rainfall intensities, that can result in as much as a 50% error in design discharge computation.

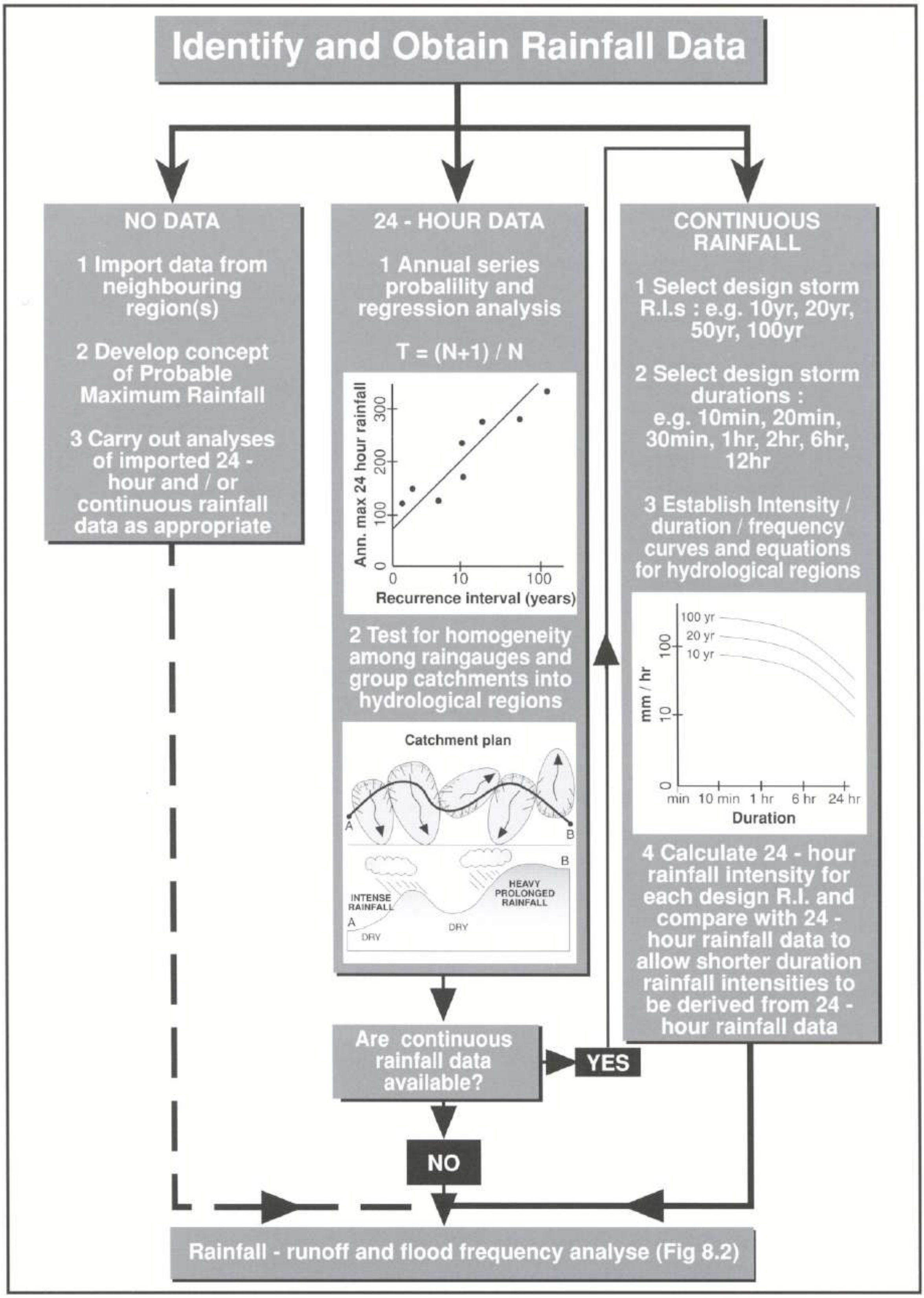
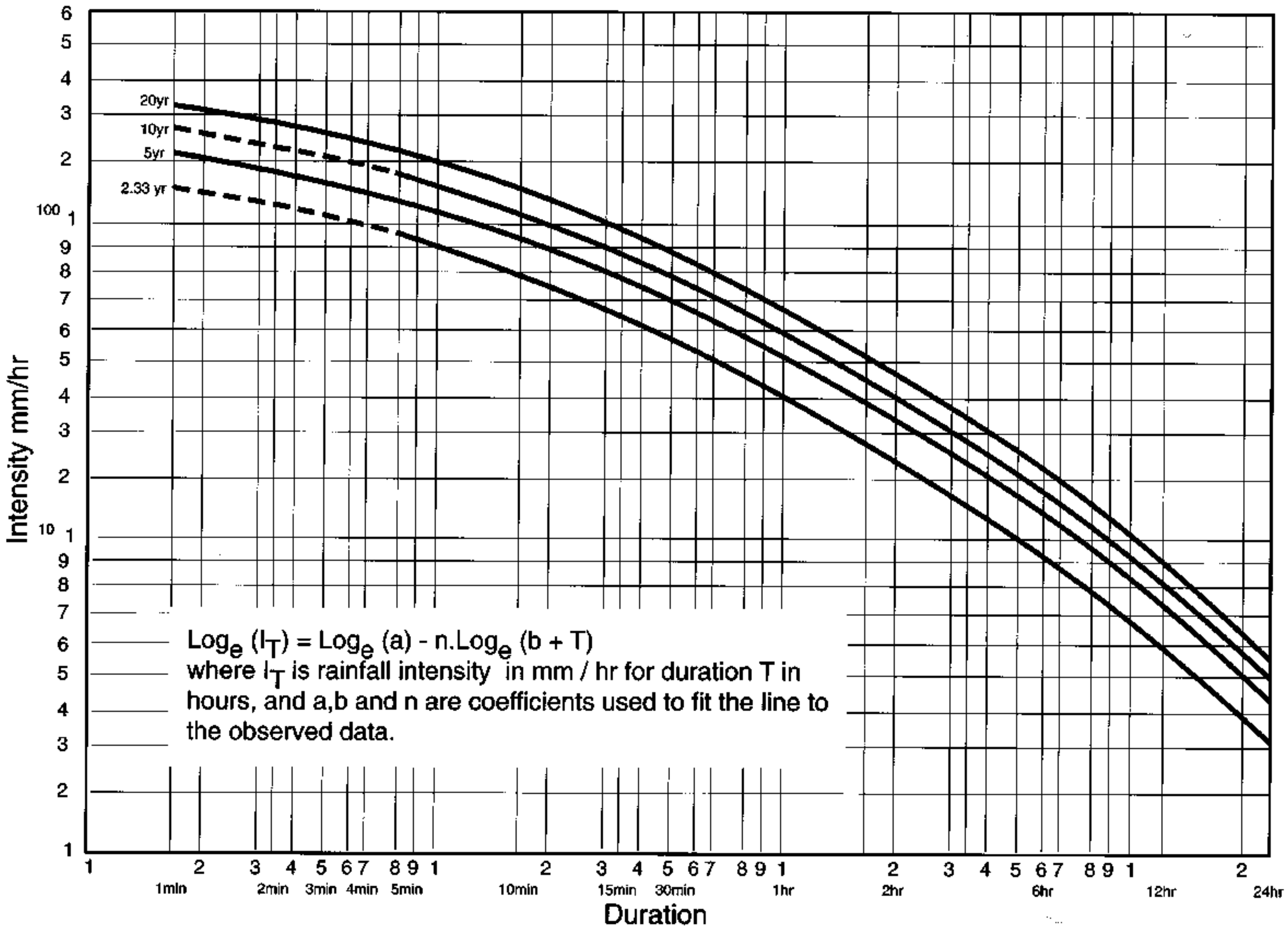


Figure 8.1 Outline procedure for rainfall analysis



For illustration purposes only

Figure 8.2 Intensity duration frequency curves from east Nepal

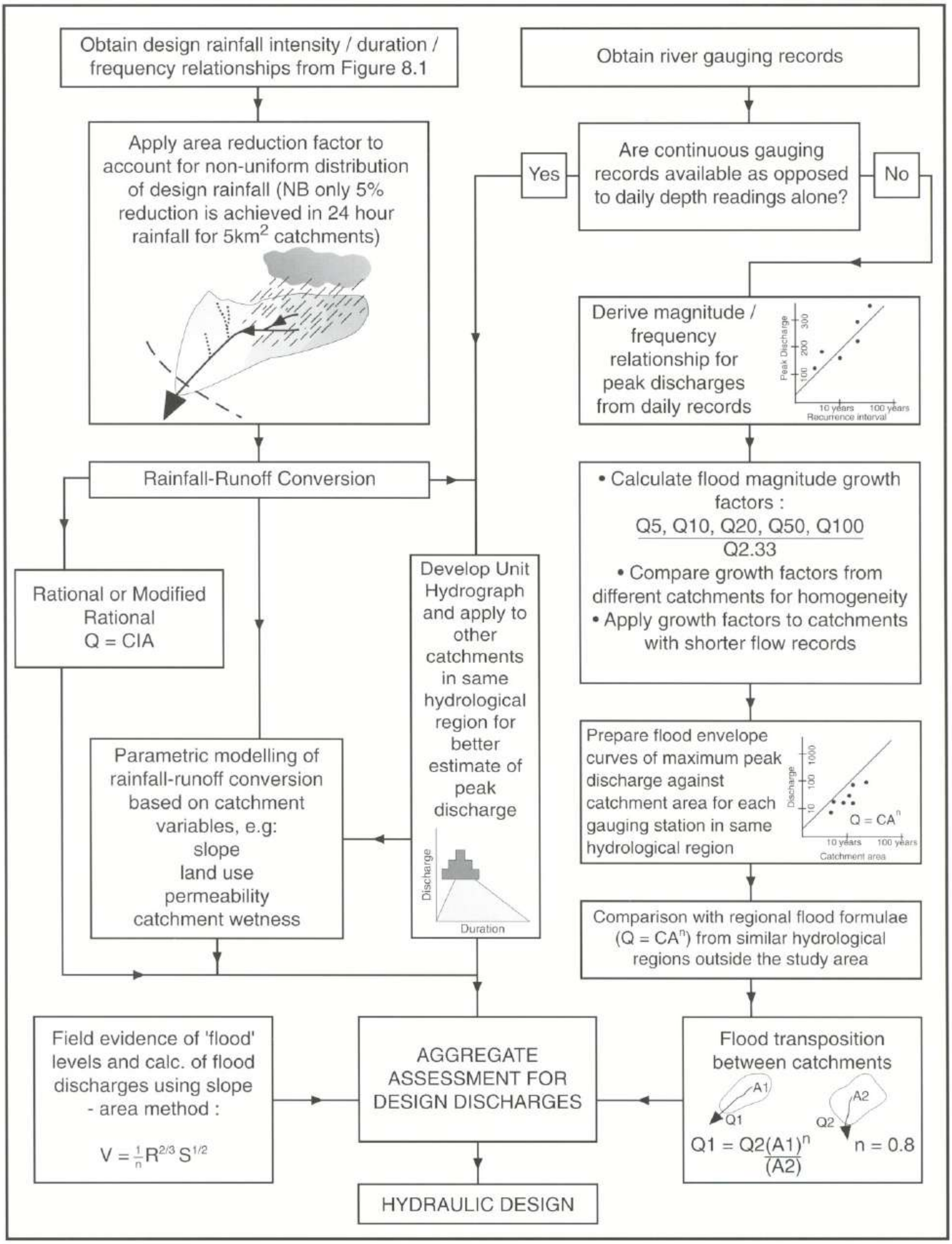


Figure 8.3 Commonly applied procedures for design discharge modelling

8.16 The runoff coefficient, C, is all-encompassing and, in theory, should embrace the following factors:

- catchment slope
- catchment shape
- soil type, depth and permeability
- bedrock permeability
- groundwater levels, antecedent moisture conditions and base
- flow
- vegetation cover and land use effects
- drainage density and hydraulic conductivity
- catchment storage
- rainfall intensity.

8.17 It is obviously inconceivable for all, or even any, of these factors to be adequately represented by a single runoff coefficient. Thus, values of 0.7 or 0.8 are commonly used by default in mountainous areas. Nevertheless, a comparison of rainfall and runoff data can allow suitable values or ranges of C to be identified for different catchment types.

8.18 Table 8.2 summaries C values derived by `back-analysis' of storm runoff from small catchments in east Nepal. Although the values are only intended as a guide, they indicate that the minimum value of C for small mountain catchments ought to be 0.7 and that, from the point of view of design, near 100% runoff should be expected in the most steep and rocky catchments during extreme rainfall.

8.19 Other models have been developed using the same basic concept as the Rational Method, but with the inclusion of several other indices to take greater account of rainfall variability and catchment characteristics. The Gupta Modified Rational Method and the Generalised Tropical Flood Model are summarised in Box 8.2. Generally, however, the Rational Method is the preferred approach for small catchments under the usual conditions of limited rainfall and catchment data.

8.20 Where the requisite data exist, storm flow modelling by hydrograph analysis is the preferred alternative to the Rational-type approach. The Unit Hydrograph method relies on the derivation by continuous flow gauging of a flood hydrograph generated by a short duration, well defined, single-peaked storm uniformly distributed over the catchment concerned. This hydrograph then reflects the combined physical characteristics for the catchment, and the ordinates of future hydrographs with a common base time will be directly proportional to the total amount of direct runoff (excluding base flow). The principal advantage with the technique lies in the ability to model peak discharge more closely in catchments with similar runoff regimes to that in which the Unit Hydrograph was derived. Usually, however, due to lack of data in most remote mountain areas, the technique has limited potential application for hydraulic design.

Table 8.2 Runoff coefficients based on data from east Nepal

Predominant land use	Shallow (<25°)	Moderate (25-35°)	Steep (>35°)
Agricultural terrace (large storage capacity)	0.5-0.6	0.6-0.7	Not represented
Mixed terrace scrub and forest	0.6	0.7-0.8	0.9-0.95
Forest (usually steep rocky slopes, thin soils)	Not represented	0.8-0.9	0.9-1.0'

Note 1. Steep, saturated, rocky catchments

BOX 8.2 Gupta Modified Rational Method and Generalised Tropical Flood Model for calculation of peak discharge

Gupta Modified Rational Method (Gupta, 1973)

$Q = 640 \times PRAe^x$

where
$x = 0.92 + 1/42.\log Ae$
Q is peak discharge in cusec (ft^3/sec)
P is average area rainfall in inches/hour
R is runoff coefficient
Ae is effective catchment area in square miles.

The average area rainfall (P) is determined by multiplying the design intensity by a catchment shape factor. The runoff coefficient is derived from P-I/P where I is the infiltration rate in inches/hour. Infiltration rates recommended by Gupta vary between 0.22 ins/hr for barren, hilly catchments to 0.7 inches/hour in forested catchments with sandy soils. An area reduction factor is then applied to account for the fact that area-specific discharges (Q/area) tend to decrease as catchment area increases. The effective catchment area is calculated by multiplying the catchment area by a factor (less than unity) based on time of concentration.

Generalised Tropical Flood Model (Watkins and Fiddes, 1984)

$Q = FCPA/360Tb$

where
Q is peak discharge in cumec
F is peak flow factor
C is runoff coefficient
P is storm rainfall in millimetres
A is catchment area in square kilometres
Tb is base time in hours.

The calculation of C in this model is fairly comprehensive and requires evaluation of indices for soil permeability, catchment slope, catchment wetness and land use. The peak flow factor is calculated by multiplying average storm discharge by a factor of between 1.7 and 3.9, while hydrograph base time is determined from the following equation:

$Tb = CA^{0.5}/S^2 + Ts$

where
C is 30 for humid catchments
S is catchment slope class
Ts is a base time adjustment in hours to allow for surface cover, and varies from 0 to 20.

Both these models were tested against extreme rainfall data from small (less than $1km^2$) catchments in Nepal. The Gupta model was found to be an improvement on the Rational Method but it was concluded that the comparatively small improvements in accuracy did not warrant the additional calculation and analysis. When applied to a $50km^2$ catchment, the Gupta model performed significantly better. The Rational Method tends to significantly overestimate peak discharges from larger catchments due to the simplifying assumptions upon which the method is based.

Despite being more comprehensive in its analysis of catchment characteristics, the Generalised Tropical Flood Model performed less favourably than either the Rational or Gupta models. This is probably due largely to the fact that the majority of catchments investigated are extremely variable in terms of slope, soils, ground cover and land use. Also, significant errors can be expected if the wrong value is assigned to some of the indices, especially as some of the factor definitions are ambiguous as far as Himalayan catchments are concerned.

Sophisticated catchment modelling for the design of road drainage structures is only appropriate when the various factors can be evaluated with an acceptable degree of confidence and there is proven method for applying these factors to the conversion of rainfall into runoff. Usually, the most successful methods of achieving this combine rainfall loss equations with the unit hydrograph approach. Where sufficient rainfall and runoff data exist flood routing can be carried out to take account of sub-catchment variability.

REGIONAL FLOOD DATA AND FORMULAE

Published area-specific discharge data

8.21 Maximum recorded area-specific discharge data. expressed in cumec/km^2 of catchment are frequently published in the hydrological and geomorphological literature, and can provide useful additional information with which to formulate an aggregate overview of discharge potential. They should not, however, be used in isolation, especially as they usually offer no reliable indication of recurrence interval.

Empirical discharge-area equations

8.22 These equations are based on maximum recorded or maximum probable discharge. They take the general form of:

$$Q = CA^n$$

where

Q is peak discharge

C is a runoff coefficient

A is catchment area

n generally ranges between 0.7 and 0.8, and is dependent on recurrence interval.

8.23 As these formulae equate discharge directly with catchment area, and do not take rainfall depth or intensity into account they should strictly only be applied in those hydrological regions for which they were derived. This is especially important in sub-tropical mountains, where specific water yields are often higher than anywhere else in the world. As stand-alone techniques of flood assessment for hydraulic design, they should be used at the feasibility stage only with adequate safety margins. For detailed design they should only be used in a corroborative capacity. A selection of empirical equations developed in the Himalayan region is listed in Table 8.3.

8.24 The Dickens equation is probably the most commonly applied of empirical techniques, although it is often correlated with the Probable Maximum Discharge or other extreme floods that are associated with too high a recurrence interval for the design of most road drainage structures.

Flood envelopes

8.25 Flood envelopes are obtained by plotting maximum recorded discharge against catchment area on logarithmic graph paper and drawing a line that bounds the upper plotted points (Figure 8.4). Maximum recorded discharges within the same hydrological region are usually obtained from river gauging records, although it may be possible to supplement these data with field assessments of maximum discharge based on the slope-area method. These envelope equations are no different in their method to the empirical equations described above. Although some indication of recurrence interval may be possible from flood frequency data and historical accounts, it will generally not be possible to assign a recurrence interval to a flood envelope, as it represents a number of extreme flood events from different catchment areas. Flood envelopes based on historical floods

Table 8.3 Some empirical discharge-area equations from the Himalayan region

Equation	Explanation	Origin
DICKENS (1865) $Qs = CD^{0.75}$	Qs = Peak discharge (cusecs) C = Runoff coefficient D = Catchment area (sq miles)	INDIA
CRAIG (1884. Full reference not found) $Qs = 440CW.Log_e(8L^{2/W})$	L = Catchment length (miles) W = Mean catchment width (miles)	INDIA
INGLIS (1949) $Qs = 7000D/(D+4)^{0.5}$	As above	INDIA
HEARN (1987) $LogQs = 0.479LogAa + 1.888$	Aa = Catchment area (acres)	NEPAL
WILLIAMS (1991) $Qm = 12.13A^{0.80}$	Qm = Peak discharge (cumecs) of 100 yr flood A = Catchment area (km^2)	NEPAL

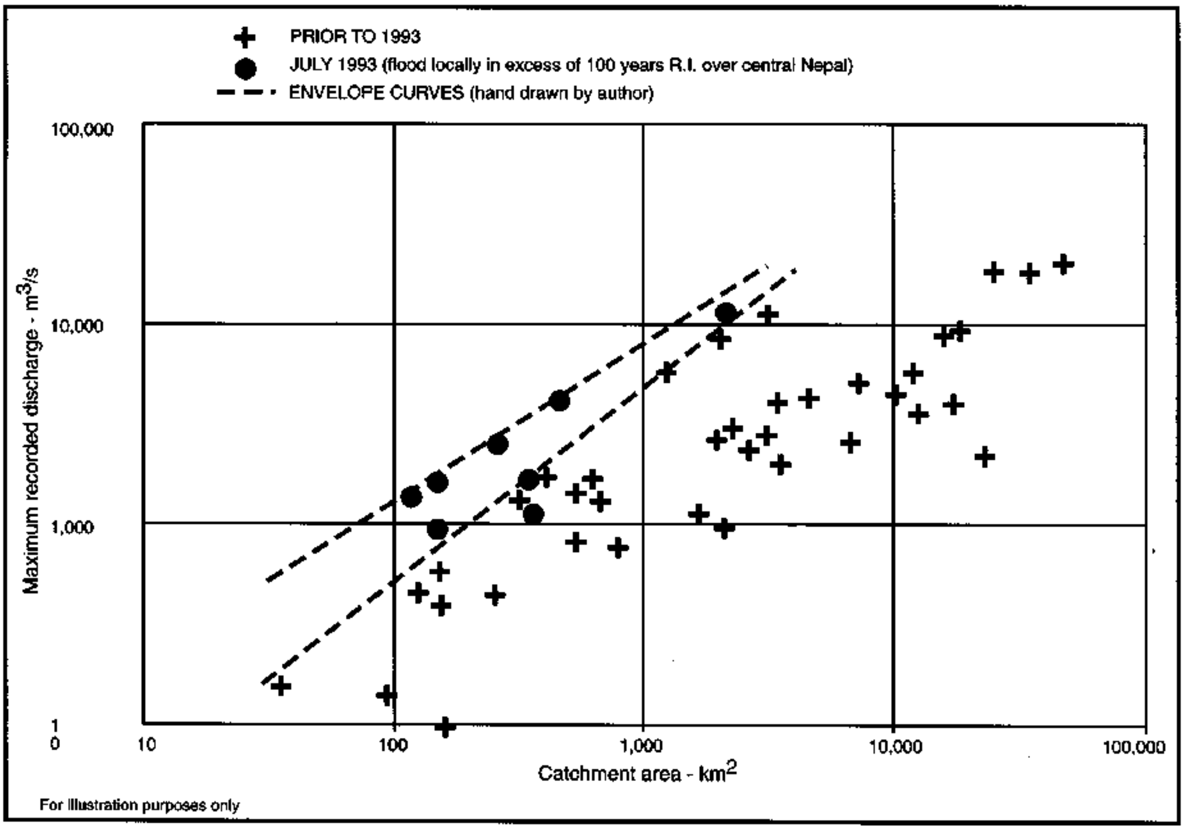


Figure 8.4 Envelope curves for maximum recorded floods in Nepal (Source: SMEC, 1993)

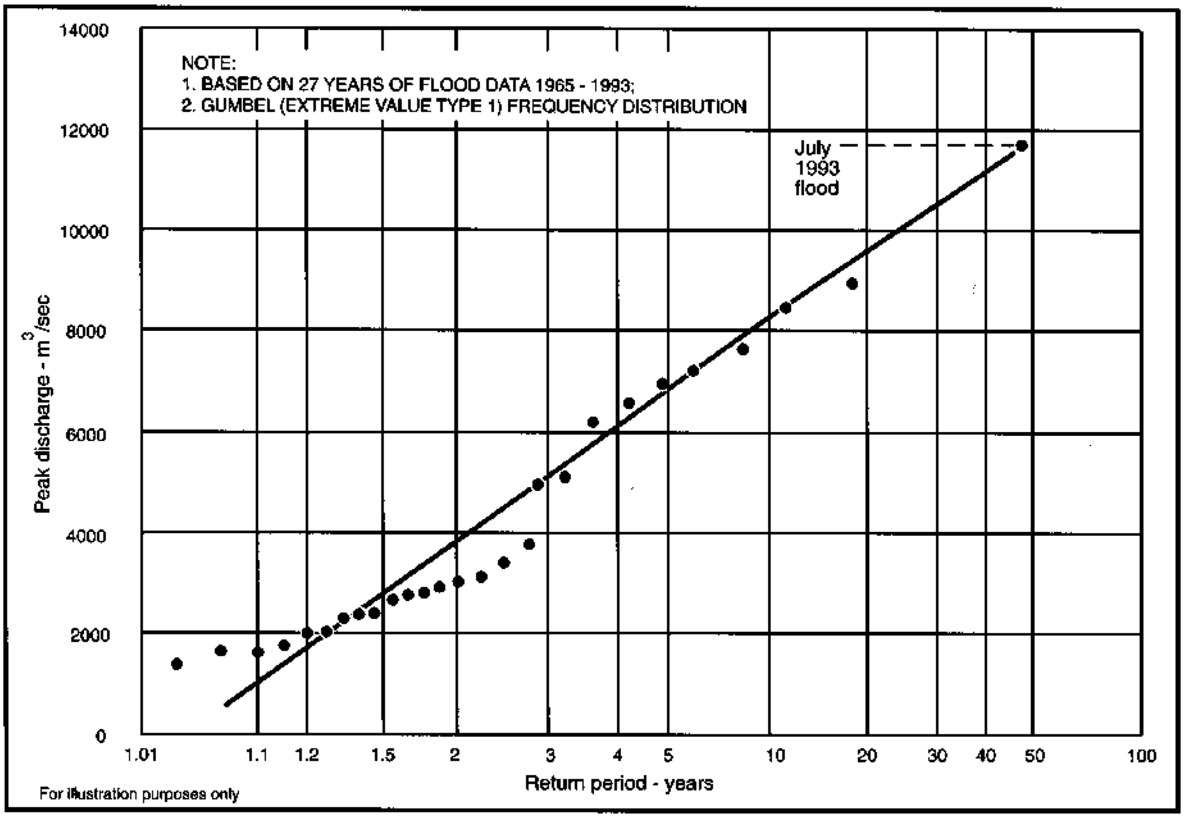


Figure 8.5 Flood frequency distribution for the Bagmati river, Nepal (Source: SMEC 1993)

tend to be conservative in their estimate, although large floods may be becoming more frequent due to land use and climatic change, or their frequency may have been underestimated from short term records.

FLOOD FREQUENCY ANALYSIS

8.26 Where available, selected river gauging data provide the most accurate assessment of flood frequency for design purposes (Figure 8.5). Nevertheless, it is worth bearing in mind that the accuracy of flow depth measurements, and hence discharge calculations, during floods is often only described as `poor' or `fair' by gauging authorities due to the difficulty in reading graduated staffs during heavy rain, and frequently at night. Furthermore, flood flows will contain significant volumes of debris which will increase the apparent water discharge above that due to runoff alone.

8.27 Annual flood series analyses of flow records not less than 10 years duration can be undertaken to establish regression equations relating peak discharge to recurrence interval (eg Q2.33 (approximately mean annual flood), Q5, Q10, Q20, Q50 and Q100) for each gauging station. A comparison of discharge ratios, such as Q10:Q2.33, will provide a further test for homogeneity between catchments and flow records. Average longer period discharge ratios (eg Q50/Q2.33) can be calculated for catchments with long flow records and extrapolated to shorter period flow records from catchments within the same hydrological region. Where these catchments have no flow records at all, the technique of flood transposition can be applied if catchments are sufficiently similar with regard to rainfall, topography, soils and land use. The formula is written as:

$Q1 = Q2.(A1/A2)^n$ for a given recurrence interval flood, where n varies between 0.8 and 0.5 for catchment areas of between $100km^2$ and $1,000km^2$ respectively.

8.28 The ratio A 1/A2 should not exceed 2 or be less than 0.5, as the case may be. When discharge data are available for more than one catchment area then the transformation calculation should be repeated to derive upper, lower and mean discharge estimates.

SLOPE-AREA METHOD

8.29 The identification of flood levels from site inspection allows peak discharge to be calculated from the cross-sectional area of stream channels combined with the maximum velocity inferred from a suitable open channel flow equation. This method of peak discharge assessment has fairly standard application at bridge sites, where it is important to augment design discharges based on hydrological modelling with actual field observation.

8.30 In the absence of flow gauging data the following evidence can be taken as indicative of previous flood levels:

- accounts of flood height given by local people with respect to readily identifiable features, such as river terraces, trees, large boulders and houses, bearing in mind that night-time flood peaks are unlikely to be observed with any accuracy unless they affect occupied dwellings
- stranded sediments and vegetation on river banks, valley-side slopes and in neighbouring trees or bushes, although drift wood is usually quickly removed for fuel in populated areas
- erosional features, including scoured vegetation lines and indentations on river banks and valley slopes.

8.31 Once the peak flood cross-section is defined, peak velocity is usually calculated by reference to flow roughness equations, and traditionally the Manning formula has been the most popular:

$V = R^{2/3}.S^{1/2}.n^{-1}$

where

V is peak velocity in metres per second

R is hydraulic radius of flood channel in square metres

S is water surface slope in percent

n is an all-encompassing roughness coefficient.

8.32 This and similar roughness equations provide a rapid and useful means of flow computation in comparatively wide and uniform trunk rivers where channel cross-section and flow regime are controlled more by hydraulic considerations than by geology. However, a number of considerations should be borne in mind (Box 8.3) regarding their use in steep mountain streams with coarse bedload.

8.33 Published values of Manning's n for mountain streams and flood plains are reproduced in Table 8.4. The most effective way to determine a representative value for n is to back-analyse from measured velocity by flow monitoring. Even then, the value of n will vary according to the volume and size of sediment transported, as well as changes in channel and flow configuration during individual floods. Back-analysed n values for small mountain streams in Nepal varied between 0.05 and 0.2.

8.34 The Stickler method, which offers an alternative approach to the calculation of n from the median size of the bed material, is of little value in steep and irregular mountain streams because it ignores the effects of other forms of roughness. In steep, boulder-filled streams, the Strickler equation was found to estimate velocities that were, on average, 40% higher than those derived from a constant n value of 0.045. When applied to a 100m-wide, active flood plain with coarse bed material the equation yielded an n value of 0.02-0.03. A value closer to 0.1 is considered to be more appropriate. Obviously, the performance of any one

- unless flood levels can be correlated with a recent storm event, for which a recurrence interval can be assigned, there may be no reliable method of determining the frequency of the calculated flood discharge

- channel scour during subsequent flows can significantly increase the size of a cross-section while, by contrast, post-peak deposition of fine-grained material can significantly Deduce the accuracy with which the maximum flood channel is identified and surveyed

- some channels retain cross-sections that are related to earlier drainage patterns and higher base levels, and are consequently over-sized within the present flow regime

- a channel can be formed or greatly modified by a single debris flow event, due to the scouring effect of sediment-laden flow. This process of channel formation will generally bear no relation to uniform flow theory

- steep, rocky channels may be so irregular that measured relationships between velocity, roughness, slope and cross-section are meaningless. The floor of many mountain streams is often better represented by a series of cascades with flow interchanging rapidly between supercritical and subcritical. Relationships such as the Manning equation do not apply because of the rapidly varying flow, but they can be used as a surrogate by adopting a high roughness coefficient.

Table 8.4 Published values for Manning's roughness coefficient (n)

Channel Type	Roughness coefficient		
	Good	Average	Poor
Mountain Streams			
- Bottom mainly gravel	0.030	0.040	0.050
- Bottom cobbles and boulders	0.040	0.055	0.070
Flood Plains			
- Grass or low crops	0.030	0.040	0.050
- Brush	0.050	0.075	0.100
- Trees	0.075	0.100	0.150

flow roughness formula will vary from channel to channel and, wherever possible, it would be wise to test various formulae against flow data before any is used in preference to the others.

8.35 Computer programs can be used to model flow behaviour by computing water surface slope, flow velocity and Froude number from surveyed channel cross-sections and assumed channel roughness. As with flow roughness equations, the model will have its greatest potential application in streams that do not display rapidly varying flow conditions.

8.36 There are many instances where correlations have been derived between bank-full discharge, or channel capacity, and catchment area, or distance downstream. Although these equations may provide useful supplementary data to that derived from rainfall-runoff analysis and the Slope-Area Method at bridge sites, they offer little advantage for culvert design.

GLACIER LAKE OUTBURST FLOODS (GLOFs)

8.37 If an engineering structure such as a hydroelectric installation or major bridge is to be located in the path of a potential GLOF arising from one of a small number of glacier lakes, then a visit to the glacial region may be required, to assess the degree of hazard. The potential for lake burst can only be assessed by measuring moraine thicknesses and lake levels, melting rates of ice lenses, moraine erosion rates and the potential for slope failure and rock or snow avalanche into or near the lake. Where a GLOF could be generated from a number of widely dispersed sources, it would not be practicable to carry out investigations of this nature for an individual road project. Under these circumstances, volumes can only be estimated by reference to previous GLOFs with sufficient safety margins.

8.38 Once a GLOF hazard has been identified and its volume estimated, it is advisable to assume that the release of water into the drainage system will be instantaneous. The hydrograph generated by a GLOF can be modelled using the same routing procedures as applied to dam break floods. The following data are required for the analysis:

- GLOF volume and initial hydrograph shape
- base flow and tributary inflow at the time of GLOF
- valley floor long-section and representative cross-sections, nominally at 1km intervals downstream
- a value of Manning's roughness coefficient (0.1 may be appropriate to allow for flow turbulence and sediment load).

8.39 The GLOF hydrograph can be routed under unsteady flow conditions. Sensitivity analysis can be used to assess the performance of the model and the extent to which GLOF levels are affected by, for example, an increase in flood volume and a slightly lower or higher value of Manning's n. Bridge soffits should be designed with a freeboard well above the expected GLOF level. Road alignments should also be given sufficient freeboard, but even if freeboard is adequate for the passage of flood waves, a road may still be severely damaged by consequent slope failures and erosion that extend upslope.

HYDRAULIC DESIGN

Culverts

8.40 Culverts are typically designed to run 75-90% full under inlet control to cater for storms of between 10 and 20 years in recurrence interval. Figure 8.6 shows the method of culvert sizing used for a proposed access road in Nepal. Culvert design is discussed further in Chapter 10.

Bridges

8.41 Bridges are designed with a soffit level normally between 1 m and 2m above the design flood to allow floating debris to pass unhindered. This soffit height can be varied according to the degree of confidence in the hydrological analysis. Preliminary design can be limited to a design concept and required aperture, but detailed design will require ground survey and site assessment of flow patterns, scour depths and scour distribution, bedload transport, flood levels and general cross-section stability. There are recent examples from Nepal where bridge decks have been lifted off their bearings by the combined effects of surcharge and entrapped floating debris, and deposited hundreds of metres further downstream.

8.42 The choice of a suitable bridge site is usually made on the basis of the following criteria:

- narrow channel to minimise bridge length
- stable rock for abutment foundation
- straight and constant reach of river channel to reduce the possibility of flow direction changes and natural scour
- minimal backing-up of water from downstream due to tributaries and valley constrictions
- minimal intrusion of bridge structure and approach embankments into channel flow.

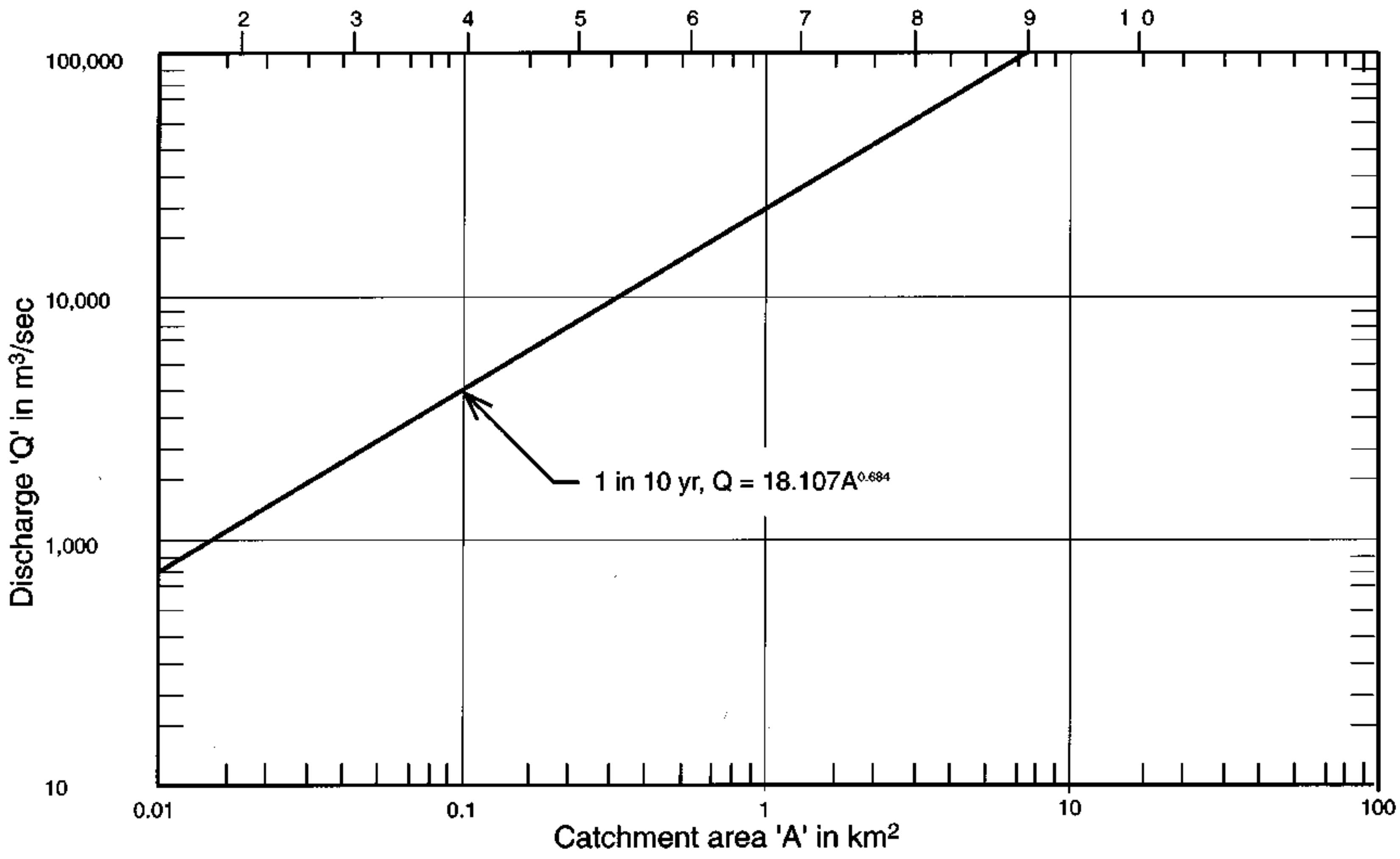
Scour

8.43 Scour is probably the most common cause of bridge failure. For major bridges it is necessary to calculate waterway width and potential scour corresponding to at least the 100 year flood with a margin for factor of safety. Where scour is predicted to be a major hazard, and where valley configuration, construction practicality and economics permit, it may be safer in the long run to design a bridge with a longer span and place the abutments at a higher elevation out of reach of the river, than to opt for a shorter span with protected abutments or piers positioned in the flow. If there is no option but to construct an abutment or pier within the design flood waterway width, such as on wide flood plains and outwash fans, scour protection becomes paramount.

8.44 Scour will occur when flow velocity and turbulence are sufficient to pick up bed and bank material, or when sediment swept along in the flow is large enough and travelling fast enough to abrade the channel boundary. The factors that control the pattern of scour are so difficult to quantify that there is no reliable physically-based method for assessing their combined effect. Methods that do exist generally relate to lowland rivers, or model studies, and in mountainous areas there may be no option but to resort to empirical models based on regime theory, together with experimentation carried out in less severe environments. Useful data can be derived from field assessment but it is important to bear in mind that scour holes may form and fill again on numerous occasions during the passage of a single flood, and observations made afterwards may be spurious.

8.45 Scour is conventionally subdivided into two categories, namely general scour of a channel cross section as a whole, and local scour due to specific flow perturbations such as meander bends, rock outcrops and bridge piers and abutments. Normal scour depth (Y) can be calculated from regime theory using the Kellerhals equation for gravel bed rivers:

$Y = 0.47Q^{0.8} . D_{90}^{-0.12}$ =, where Q = discharge (cumecs) and D^{90} is the 90 percentile particle size diameter (metres).



PIPES:

Culvert group	Size (mm)	Culvert group	Size (mm)
1	600	3	1200
2	900	4	1500

SLABS OR BOXES:

Culvert group	Size (mm)	Culvert group	Size (mm)	Culvert group	Size (mm)
5	1500 x 1500	7	2500 x 2500	9	4000 x 4000
6	2000 x 2000	8	3000 x 3000	10	5000 x 5000

For illustration purposes only

Figure 8.6 Look up chart used for sizing culverts in east Nepal

8.46 Scour can be determined by subtracting the peak flow depth under non-scouring conditions from the value of Y obtained in the above equation.

8.47 Scour depths estimated by the regime method will be average for the cross-section as a whole, and it is necessary to multiply these values by a factor of between 1.25 and 2.00 to account for local variations across the sections. The graphical redistribution of scour is an accepted way of achieving this and will also enable reductions in bed width to be taken into consideration as scour proceeds.

8.48 An alternative approach to the regime method is to calculate scour potential using tractive force theory and the mean velocity expected through the bridge waterway during design discharge. Velocity is usually determined from the Manning formula and combined with a representative value of bed material size to assess whether or not entrainment will occur using accepted scour velocities. The representative bed material size is in the range of the median diameter (DSO) and the 80 percentile (D8o), depending on how graded the material is. Trial and error is then used to determine the average general scour level that will make the mean flow velocity equal the threshold velocity for erosion of the material exposed at that level.

8.49 In mountain streams and rivers with small to medium-sized bed materials this approach will yield conservative results because scour holes may become quickly filled with sediment from upstream and bed materials tend to become coarser and more dense with depth. For rivers with a high proportion of boulders, scour depth can be estimated on the assumption that the larger fraction of bed material will protect the smaller material beneath. Under these conditions, the following relationship will apply:

$$D_{50}/d = CF^3$$

where

D_{50} is the median size of bed material (metres)
d is the hydraulic flow depth in metres (area/surface width)
C is 0.22 for a factor of safety of 1.0
F is the Froude Number, $V/(gd)^{0.5}$
V is mean velocity (metres/second)
g is acceleration due to gravity.

8.50 Maximum scour- is usually assessed on a site by site basis, but for preliminary design is taken to be twice the normal scour depth. Normal scour is calculated from regime theory (Box 8.4).

Sediment transport

8.51 Despite the fact that culverts on mountain roads are frequently blocked by sediment or debris, there is still no reliable means of predicting sediment yields from mountain catchments for design purposes. Conventional bedload transport formulae are usually either empirically-based, and hence applicable only to the catchments from which they were derived, or are based on tractive forces and assumptions regarding velocity, depth and bed material size that are untenable outside flume laboratories, and particularly in mountain streams. Also, lack of data in most cases, will mean that parameters contained in these formulae cannot be properly evaluated. Predicted bedloads can vary by as much as ten-fold depending on the formula used.

8.52 Another problem with predicting sediment yields in mountain catchments is the fact that the highest concentrations are often the result of instantaneous injections of landslide debris into the channel flow, that do not form part of hydraulic considerations. Also, not all eroded material is transported through the catchment immediately; large quantities are stored for significant periods in terraces, debris fans and braided channel reaches.

8.53 The only practicable means of assessing sediment potential is to carry out a mapping exercise to determine the relative erodibility of channel material, the potential for slope failures, and the likely efficiency with which storm runoff is able to transport debris through the catchment. In addition, road earthworks and tipped spoil can lead to significant increases in sediment load, as can roadside erosion. Although these considerations can, to a certain extent, be quantified, there is no reliable method by which the data can be translated into sediment transport rates without monitoring records.

8.54 Realistically, the best approach is to identify those catchments that are likely to give rise to long-term problems of culvert or bridge blockage, and design the crossing structures accordingly. A box culvert with inlet guide walls and a slope equivalent to that of the natural stream bed is more self-cleaning than any other design, while a short-span bridge may be preferable to a large culvert where an alignment crosses an eroding gully with frequent debris flow activity. Chapter 13 describes the options that can be considered when designing a road across an active fan.

Box 8.4 Regime theory and scour calculation

From regime theory, the unobstructed waterway width (W) can be calculated from the equation:

$$W = 3.26Q^{1/2} = (Q = \text{discharge in cumecs})$$

If the waterway length of the bridge (L) is less than the regime width then the normal scour depth (Y') is increased according to the following equation:

$$Y' = Y(W/L)^{0.61} \ (Y = \text{regime scour depth})$$

Where the bridge and its approach embankments cause a constriction to the flood waterway, then the maximum scour depth obtained by application of the factors at paragraph 8.45 should be compared with Y.. given by the following equation.

$$Y_{max} = Y(W/L)^{1\ 56}$$

and the greater of the two values adopted for design.

9 EARTHWORKS

CHOICE OF CROSS-SECTION

9.1 The choice of cross-section is usually second only to the choice of alignment and design standard in determining the stability, cost and environmental impact of the final construction. In gently sloping terrain, with ground slopes of less than 20° or so, there are generally few constraints on the choice of cross-section and a balanced cut and fill, which is generally the most preferable in terms of engineering practicability, cost and environmental considerations, is usually achievable. By contrast, in steep mountainous terrain a balanced cut and fill is virtually impossible to achieve over any significant length of alignment given geometric constraints and the abrupt and frequent changes in topography. Where there is no choice but to cross steep and irregular cliff and ravine topography, the need to select the most appropriate detailed alignment location and choice of cross section, on a slope by slope basis, becomes paramount.

9.2 Figure 9.1 illustrates the more important factors to be considered in cross-section design and identifies slope conditions under which various cross-sections are usually most applicable. These conditions are discussed briefly below.

Mostly full cut cross-section

9.3 Traditionally, low cost roads have been constructed across steep side-long ground in full cut with the excavated material side-tipped along the length of the alignment. This approach maximises the use of cheap unskilled labour in excavation and avoids the need to construct comparatively expensive retaining structures. Furthermore, in the absence of compaction plant, a road formation in cut will be more trafficable than one in till.

9.4 However, unless dictated on stability grounds (Figure 9.1), or where there is no practicable alternative, the widespread adoption of a full cut cross-section cannot be recommended for the following reasons:

- it inevitably reduces the factor of safety against failure of the natural slope above, and leads to a greater potential for failure and erosion in the cut slope itself
- it maximises the amount of spoil generated, resulting in problems of disposal and erosion below the road.

9.5 Alternatives to the full cut cross-section on steep rock slopes include tunnelling, half tunnelling, the use of short-span bridges, masonry buttress-supported road slabs (Chapter 11), and masonry road fill retaining walls. Unless tunnelling can be shown to be cost-effective in reducing alignment length, or in avoiding long-term maintenance costs, the high construction costs are usually prohibitive. Half tunnelling with lateral support is only feasible in strong and massive bedrocks with planes of weakness dipping into the hillside (ie favourable to stability). Short-span bridges and masonry buttresses built across steep rock reentrants are cheaper and often more feasible than tunnelling through the adjacent spurs, although falling rock can cause significant damage. It is usual, therefore, to combine elements of full cut with road fill retaining wall in most steep ground situations.

Balanced cut and fill cross-section

9.6 Where there are no underlying stability problems, a balance between cut and fill in any one cross-section can usually be achieved on slopes up to 30° without the requirement for a road retaining wall. However, this usually requires a sinuous alignment to follow the slope contours, leading to an increase in alignment length and not necessarily the correct choice of cross-section in each case. Alternatively, designing for balanced cut and fill over a short alignment length will enable preferred cross-sections to be combined with an undulating vertical alignment. Specifying balanced cut and fill assumes that plant is available to haul excavated material to adjacent fill sites. With labour-based construction methods, the maximum practicable haul distance may be little more than 50m, whereas plant intensive construction will allow almost unlimited haulage distances, albeit at a cost. Specifying balanced cut and fill assumes that plant is available to haul excavated material to adjacent fill sites.

9.7 In designing the cross-section, it may be necessary to allow for a cut-to-fill bulking factor of 20% and a wastage factor of 100, giving rise to a net surplus of 10% cut over fill volume.

Mostly full fill cross-section

9.8 Where foundation stability permits, a full fill or retained fill cross-section is most appropriate under the following slope conditions:

- where persistent joint, bedding or foliation planes are dipping out of the slope and could give rise to failure in excavations
- where excavations for cut slopes in shallow loose scree or similar deposits at their limiting angle for stability could result in progressive slope failure
- where an alignment is located on eroding or highly erodible soils, the stability of which would be further reduced by cut slope excavation
- where a rock fill causeway is the only means of crossing the toe of an unstable slope that is periodically undercut by river scour or kept wet by irrigation (construction of the causeway may lead to an increase in stability through toe loading and scour protection).
- for purposes of pavement drainage, where the alignment is located on a terrace.

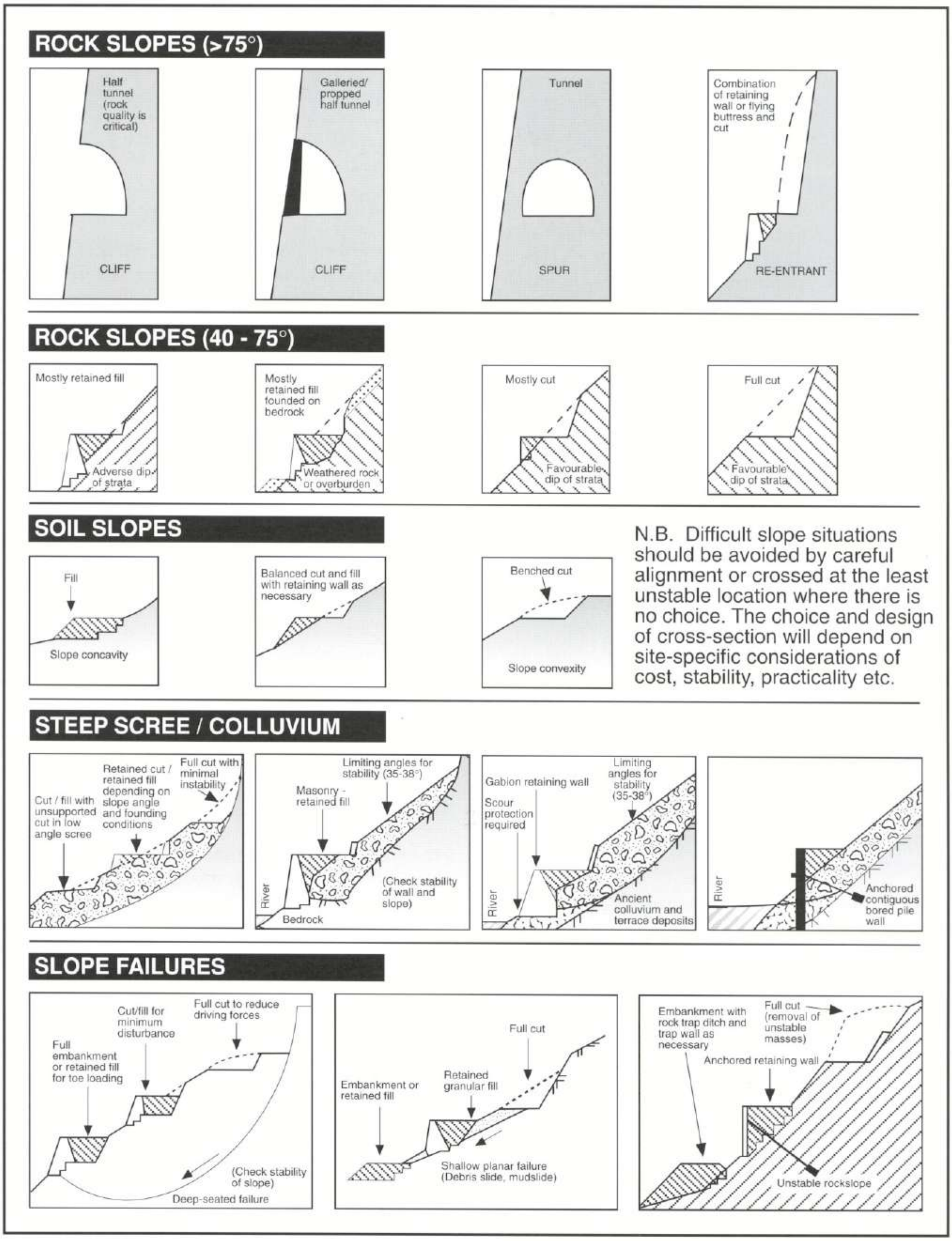


Figure 9.1 Stability considerations in the choice of cross-section

EARTHWORKS DESIGN AND CONSTRUCTION

Cut slopes

9.9 Prior to excavation, and in the absence of detailed subsoil investigations, the design of cut slopes is usually only provisional. It is based on estimated soil and rock conditions from walkover surveys and trial pitting (Chapter 7), modified locally by topographical constraints and right of way limitations. In fact, the engineering behaviour of slope materials will be confirmed only when slopes have been exposed for at least one wet season. Even then, rainfall patterns, differential weathering of slope materials and, above all, the effect of changing land use and irrigation practices, can cause failures several years after construction.

9.10 The combination of geotechnical uncertainty, and the variability in ground conditions, frequently between one cross-section and the next, usually means that assumed or calculated strength parameters may be too low for one slope, leading to over-conservative excavation, or too high for the next, causing slope failure. If it is desired to keep the road open at all times with minimal landslide debris clearance, then the designed slope will need to be on the conservative side with a notional factor of safety of at least 1.2, and with adequate provision for erosion protection where vulnerable soils are exposed over large areas in the excavation. If temporary road closures and debris clearance can be tolerated and allowed for in maintenance, then a factor of safety of 1.1 and a steeper slope may be more economic, even in the long term. Table 9.1 shows a range of cutting angles with a notional factor of safety of 1.1, derived from stability charts for common soil types found in Nepal.

9.11 The factor of safety of a cut slope will changeover time as the slope adjusts to internal and external processes of weathering, drainage, landuse and vegetation. It is advisable, therefore, to combine theoretical considerations with observational data (Chapter 7) in order to make a rational choice of cut slope angles.

9.12 The aim of any low cost approach to earthworks design is to maximise cutting angles without having to resort to extensive use of cut slope support structures. Nevertheless, slope revetment and retaining structures will be unavoidable wherever:

- natural slope angles are close to their limiting angle for stability and there is no choice but to incorporate some cut in the cross-section
- alignments are required to cut through unstable slopes
- the constraints of alignment geometry dictate structural support to earthworks, for example where hair-pin bends and stacked loops are to be constructed on steep ground (35° or greater) in weak materials
- failures in the cut slope occur during construction and maintenance.

9.13 Where cut slopes are less than a metre or so in height, it may be preferable on environmental grounds to slacken them back to an angle that allows the earthworks to appear as part of the landscape. This may also provide additional materials for construction. The value of neighbouring land uses and right of way restrictions may be limiting factors in adopting this approach.

9.14 Cut slope profiles can be single-sloped, multi-sloped or benched. Single-sloped profiles are usually cut in uniform soils and rock slopes. Multi-sloped profiles are cut where an excavation encounters soil overlying rock, or a succession of river deposits of differing resistance to erosion or failure (Box 9.1). Where rock is encountered, persistent joint, bedding or foliation surfaces usually determine the final cut slope profile.

Box 9.1 Terrace deposits

Vertical variations in the particle size and density of alluvial deposits exposed in excavations may warrant a multi-sloped design. While medium dense gravels and cobbles can stand at steep angles (60-80°) with little need for erosion protection, overlying unconsolidated sands should be cut back to an angle of about 40° and protected against erosion. However, sands that are partly cemented by iron oxides, through fluctuations in iron-rich groundwater, can usually be cut to angles of 50-60°. Site conditions will dictate whether a single-sloped or multi-sloped design is adopted.

9.15 The principal advantage of benched cut slopes, from a stability point of view, lies in their ability to slow down the rate of surface runoff, and the fact that shallow failures are usually limited to one bench at a time. The steps on a benched cut slope should slope into the hillside and be provided with collector and cascade drainage systems. Properly designed benched cut slopes are expensive to construct, and erosion and slope failure can be accelerated if benches are not regularly maintained. Their use is comparatively rare in the construction of low cost mountain roads where a maintenance commitment can often not be guaranteed. Furthermore, vegetation is more difficult to establish on the steeper riser slopes than on a uniform slope profile. A benched cut slope is visually less appealing than a single-sloped excavation.

Table 9.1 Provisional design gradient/height relationship for soil cut slopes

		Cut slope gradient (V/H) Cut height (metres)		
Soil type	Water table	0-3	4-6	7-10
Clayey silts (transported)	Low	1.5	1.0	0.8
	Moderate	1.2	1.0	0.5
	High	1.0	0.8	NA
Silts	Low	1.0	50.8	>_0.8
	Moderate	1.0	550.8	5_0.8
	High	1.0	0.8	NA
Coarse-grained colluvium	Low	1.0	1.0	0.8
	Moderate	1.0	1.0	550.8
	High	1.0	0.8	NA
Silty clays (residual)	Low	1.5	1.5	1.0
	Moderate	1.2	1.2	1.0
	High	1.0	1.0	NA

Note The above slope angles have been derived from stability charts with assumed c' and φ' values and an average factor of safety of 1.1. Slope gradients have not been given for 7-10 m high cuts with a high groundwater as this condition is unlikely to occur in granular soils, whose permeability is relatively high. This table is for illustration purposes only.

Box 9.2 Non-adherence to specification during construction can give rise to earthworks failures

Failures in cut slopes often occur as a result of unfinished earthworks rather than faulty design. Contractors sometimes oversteepen the base of an excavation in order to induce shallow failure so that earthworks can proceed at a faster rate. This can give rise to over-excavation and the exposure of a larger excavated slope to erosion. The same condition may occur where road widening is achieved by removing the toe of the adjacent cut slope. Slopes that are left with a convex profile (oversteepened at the base) are more prone to failure than those that are finished to a perfectly straight profile, as specified. On the other hand, concave cut slope profiles result in poor drainage at the toe, that may give rise to progressive failure by slumping and undermining. If slope drainage is an integral component of a cut slope design, then slope failure may occur if the works are not completed by the onset of the wet season.

Unretained fill slopes

9.16 The main considerations in fill slope and embankment design are the maximum permissible angle of side slope and the overall stability of the fill on the hillside. An examination of fill slope and embankment failures along a number of mountain roads shows that many are brought about by:

- inadequate under-drainage under conditions of pronounced seepage
- incomplete removal of vegetation and organic material prior to embankment construction
- construction of embankments on loose spoil material derived from earlier excavations
- erosion on slopes immediately below the embankment
- the presence of pre-existing shear surfaces beneath the embankment in fine-grained soils (infrequent in mountainous terrain)
- the presence of unfavourably orientated planes of rock weakness beneath the embankment.

9.17 Generally, the particle size distribution and the angularity of the particles are the only indicators of the shear strength parameters applicable to the design of embankment side slopes. Appropriate 0' values range between 25° and 40° for clayey and granular materials respectively. With low factors of safety (1.1-1.2) this gives side slopes of between 20° and 33°. In mountain areas, where granular soils predominate, embankment side slopes are normally constructed to 1:1.5 (33°) assuming that the specifications for material size, drainage and compaction can be met. In more gently sloping ground the side slope can be relaxed to accommodate weaker fill material, to provide a convenient means of spoil disposal and to allow the road to become visually less obtrusive in the landscape.

9.18 Adequate compaction of embankment fill can be problematic if it is undertaken by labour-intensive means. The use of hand rammers may achieve only 80% Proctor compaction in schist and phyllite granular fill. Settlement is therefore common, especially along the outer edge of embankments and around culverts. It is recommended that once fill slopes are constructed to formation level they are left for a wet season to settle before the pavement is added. This, of course, is not necessary if compaction plant is available.

9.19 The overall stability of a fill slope on a hillside is more difficult to assess. Before constructing a fill slope on side-long ground, it is necessary to terrace or step the formation in order to prevent a possible slip surface from developing at the interface between the fill and the natural ground. The potential for failure along a deeper surface in the ground beneath should be considered, although this rarely happens since the strength of soils tends to increase with depth. Unless fill is placed on colluvium, the weakest layer is likely to be just below the formation level. If thus material is similar to that used as fill, its stability on the slope will also be adequate as long as the stability of the embankment slope is satisfactory. Problems occur when strata or foliations in the rock masses beneath the fill are dipping parallel to the ground slope, or where the groundwater table is at or very close to the surface. Adverse rock planes can cause the fill to slide, triggered by increased load, or increased pore pressure along the failure plane. Groundwater can soften the founding material, or cause the fill material to be undermined through seepage erosion. In these situations fill slopes require under-drainage to keep groundwater moving freely down slope.

9.20 The design and construction of road fill retaining walls are discussed in Chapter 11.

Rock blasting

9.21 Uncontrolled rock blasting is atypical feature of road construction where labour-intensive methods of excavation rely on a finely-fragmented rock mass for easy removal. Bulk blasting, as it is termed, does not use a regular array of shotholes to produce a clean break, nor generally does it take account of the fracture pattern of the rock to assist loosening. Bulk blasting usually results in significant overbreak and the creation of a highly fractured rock mass which usually ravels and fails to shallower slope angles for years afterwards, creating a permanent hazard and maintenance problem.

9.22 With pre-split (pre-shear) and cushion blasting, overbreak is minimised and the cut slope is left intact and less prone to rockfalls. Pre-split blasting is therefore preferable to bulk blasting but is far more expensive. It is more time consuming because many more holes are needed, and it calls for well-controlled drilling methods that produce a series of closely-spaced parallel shot holes. It also requires high quality explosives and detonators, and electronic detonation. When applied correctly, pre-split blasting produces a clean, straight rock face at the design angle. It also reduces the danger of flyrock to a minimum. However, depending upon the rock joint pattern, it may produce debris of a size that can only be removed by machine unless it is broken up further, and at additional cost, by hand. If properly designed, pre-split and bulk blasting can be used to good effect together to provide a clean rock face and small-sized debris. Pre-split blasting is especially recommended wherever rock mass failure is likely.

Spoil disposal

9.23 However much care is taken to minimise quantities of spoil, it cannot be eliminated altogether. Although all topsoil and overburden should be stockpiled for later reuse, surplus material may arise from a large number of situations including:

- surplus of cut over fill requirements
- cut materials unsuitable for inclusion in 'he works
- the establishment of an initial trace along the road alignment
- the result of operations taking place out of sequence with the main earthworks, such as excavation for structures
- overburden to quarries and borrow areas
- topsoil stripped and not re-used
- surpluses of imported construction materials, such as gravel in excess of requirements for pavement layer construction
- maintenance operations such as the clearance of slips or the removal of silt from side drains.

9.24 The control of spoil disposal is of utmost concern, because it can give rise to a variety of problems, including:

- erosion of the spoil tip itself
- the smothering or removal of natural vegetation (once stripped of plant and soil cover, slopes usually take between 3 and 5 years to revegetate, and as much as 10 years on steeper and more sterile slopes)
- instability within the spoil material itself, especially when infiltrated by water
- slope overloading and resultant failure
- disruption of existing runoff patterns and siltation of water courses and drainage channels
- disruption to agricultural practices.

9.25 There are two steps to be taken in minimising spoil problems within a construction project. The first is to identify those operations that will generate spoil, the places where it will be generated and the quantities involved, no matter how small. The second is to plan in advance for its disposal by the designation of safe tipping sites. The engineer is responsible for the designation of suitable sites, and his criteria for their selection should aim to avoid the problems listed above. When construction is being under-taken through a conventional construction contract, the engineer should ensure that both the contractor and the construction workforce are aware of the restrictions on the disposal of spoil, the location of approved spoil disposal sites and specific requirements for the management of these sites. Contract specifications regarding spoil disposal should be strictly enforced.

9.26 Spoil can be dealt with either by discarding it, or by turning it into landfill. The following guidelines should be observed:

- select areas for spoil tipping on steep slopes formed in relatively resistant bedrock, where tipping will result to no more than the removal of vegetation and shallow soil, with negligible slope incision thereafter. Bitumen drum disposal shutes can be used to convey the spoil down a short slope to a safe site below

- when creating a landfill site, make maximum use of terraces, level ground and spurs

- build many small spoil benches rather than a few large ones, to avoid slope overloading

- provide a drainage blanket beneath a spoil bench where there is any indication of a spring seepage at or near the spoil site

- compact spoil benches during construction. While benches cannot be compacted in the formal sense, they can be constructed in definite lifts normally not more than 0.5m thick, with the top surface of each lift approximately horizontal. This will allow machines involved in spreading the spoil to track the surface and provide some degree of compaction

- where spoil benches are constructed on agricultural land, form the tip into a benched profile so that it can eventually be returned to agricultural production. In the meantime the risers between levels must be protected against erosion by constructing dry stone walls and applying vegetation

- where the top surface of the bench is large, runoff should be reduced by the provision of regular shallow interceptor drains. The slope of these drams should be constant as far as is practicable and should not be so steep as to induce erosion

- on completion, spoil benches should be left in their required shape and planted with grasses, shrubs and trees as appropriate, to encourage maximum stability and resistance to erosion.

Do not permit the following:

- tipping of spoil into stream channels other than major rivers, as the increased sediment load will lead to scour and siltation downstream

- tipping of spoil onto slopes where road alignments, housing areas or valuable farmland downs lope might be affected

- use of areas of past or active instability and erosion as tip sites

- the discharge of runoff over the loose front edge of a tip bench during or after construction

- tipping of spoil in front of road retaining walls where impeded drainage could soften the wall foundation.

10 DRAINAGE

GENERAL PRINCIPLES

10.1 Conservation of the natural drainage system around the road alignment should be one of the most important concerns during design and construction. By effectively creating a barrier to natural surface drainage that is only punctuated at intervals by culverted drainage crossings, road construction can lead to significant local increases in catchment area, perhaps by up to 100% in some ridge top localities. Furthermore, in the case of paved road construction especially, road drainage reduces the tune taken to reach maximum flow by shedding water from impermeable surfaces. Therefore, in addition to constructing side drains, culverts and bridges to convey their design runoff without surcharge, blockage by sediments, or scour (Chapter 8), great attention must be paid to strengthening those parts of the natural slope drainage system that experience increased runoff, and hence erosion potential, as a result of road construction. The main ways of doing this are to:

- control road surface drainage
- design culverts or drifts that convey water and the expected debris load efficiently
- maximise the frequency of drainage crossings to prevent excessive flow concentration
- protect drainage structures and stream channels for as far downstream as is necessary to ensure their safety
- plant vegetation on all new slopes and poorly-vegetated areas, around the edges of drainage structures and appropriately along stream courses, without impairing their hydraulic efficiency or capacity.

ROAD SURFACE DRAINAGE AND SIDE DRAINS

Crossfall

10.2 Control of road surface drainage and side drain runoff should be a major component of the cross-fall geometry. On all minor roads, an inward-sloping road carriageway is the normal means of shedding water from the road surface. The inward slope incorporates an inherent factor of safety in retaining water that has accidentally escaped from the drainage system. A cross-fall gradient of 3-4% is commonly adopted in order to prevent ponding on slack road gradients, and longitudinal scour of an unpaved road surface on long, steep sections of alignment. For reasons of traffic safety, outward cross-falls are required on some bends of trunk roads. In such situations it is usually necessary to prevent water from discharging overfill slopes in an uncontrolled manner by introducing some form of bund or upstand along the outside edge of the road.

10.3 Occasionally, an outward-sloping road surface has been advocated for minor roads, on the grounds that, by allowing water to disperse gently onto the hillslope along the whole length of the road, the potential for erosion will be reduced. In practice, the opposite is true. The method undoubtedly offers very large financial savings in the reduction of drainage structures, but it is a highly hazardous form of design that cannot be recommended except in areas of very low erosion potential. The design carries the following weaknesses:

- in practice it is impossible to design a road geometry for a distributed flow of water (topography is the controlling factor)
- road settlement, which cannot be predicted, will in time change the design cross-fall
- road repairs will locally alter the cross-fall
- partial blockage of the road by debris results in a change of the flow pattern of drainage water, and instant local surcharge
- erosion can reach a disastrous level before maintenance crews can be mobilised
- slope protection from uncontrolled runoff would require a lengthy period of post construction monitoring and remedial works that is not practicable for normal contractor-based operations.
- road safety. Vehicles can slide sideways uncontrollably across a wet road surface and over the edge

Side drains

10.4 Side drains serve two main functions: to collect and remove surface water from the immediate vicinity of the road and to prevent any sub-surface water from adversely affecting the road pavement structure. The latter function is achieved in the simplest case by leaving weep holes in the side drain (on both walls) set at 50-100mm above the invert. Groundwater in the subgrade can be released either by using a drainage blanket at the base of the pavement, or by incorporating gravel cross-drains (‚grips) in the road base that exit via a weephole in the side drain backed with a piece of filter fabric. Deeper drains, comprising a filter-wrapped perforated pipe within a graded gravel backfill, can be constructed under very wet slope conditions to a depth of 1-1.5m below the level of the side drain invert, and led to the nearest culvert inlet.

10.5 Side drains (as well as the road itself) should have a minimum longitudinal gradient of 0.5‰, except on crest curves. Slackening of the side drain gradient in the lower reaches of significant lengths of drain should be avoided in order to prevent siltation. On potentially unstable slopes, side drains should be lined with heavy duty polythene, or some other impermeable material, before the masonry pitching is applied. This will prevent water entering the slope if the masonry becomes cracked by movement. The gap between the drain and the hill side must be filled with compacted material sloping towards the drain to minimise infiltration behind it.

10.6 The choice of side drain cross-section (Figure 10.1) will need to take consideration of hydraulic capacity, ease of maintenance, space restrictions and traffic safety. As far as traffic safety is concerned, a wide and shallow drain for a given flow capacity is preferable to a deeper one but in particularly steep ground the extra width required to achieve this may be impracticable, or too expensive. Side drain covers can be used to provide extra road width in places where space is limited: their widespread use, however, is not recommended as they can hinder the progress of routine maintenance. Under normal circumstances, the adoption of a trapezoidal cross-section will facilitate maintenance and will be acceptable from the point of view of traffic safety.

10.7 Design volumes of runoff are usually estimated using the Rational formula (Chapter 8). Flow velocities are calculated from the Manning equation using roughness values shown in Table 10.1. Most published roughness and velocity data are based on clean water flow, whereas sediment-laden water is more common along mountain roads. Side drains are left unlined or are lined with masonry according to the strength of the material in which they are excavated, and the velocity of runoff they are expected to carry. Usually, a lined drain is required when the underlying materials are soils or weathered rock. Concrete can be used for greater strength if vehicle trafficking is likely in narrow roadway sections.

10.8 When the cross-sectional area is less than about $0.1m^2$ and the gradient is gentle, drains can be lined with unbound masonry. Larger and steeper drains are lined with mortared masonry, although they can be up to ten times as expensive. Where masonry check dams are used to reduce flow velocities in side drains, there must be sufficient cross-sectional area above the check dam to take the maximum design flow. Cascades or steps in the drain long-section can also be a useful means of reducing flow velocity, although both check dams and cascades can impede the transport of debris, increasing the risk of blockage.

10.9 If there is no opportunity to safely discharge side drain water via culverts into existing stream channels, then it is usual to construct side drain turnouts, especially in bend locations (paragraph 10.40). However, it may be preferable to increase side drain capacity to convey runoff to the next available safe discharge point, rather than to construct side drain turnouts or relief culverts on erodible slopes.

Culvert location

10.10 The desire to maintain the existing slope drainage pattern as much as possible will require all watercourses to be culverted. This may result in an average spacing of one per 100m, and a range of spacing of between 30m, or less, to greater than 300m, depending upon topography. Where stream channels are poorly developed, for instance where permeable soils occupy gently sloping ridge top locations, the opportunities for discharging road side drainage into established stream channels, that will not undergo subsequent erosion, are much reduced.

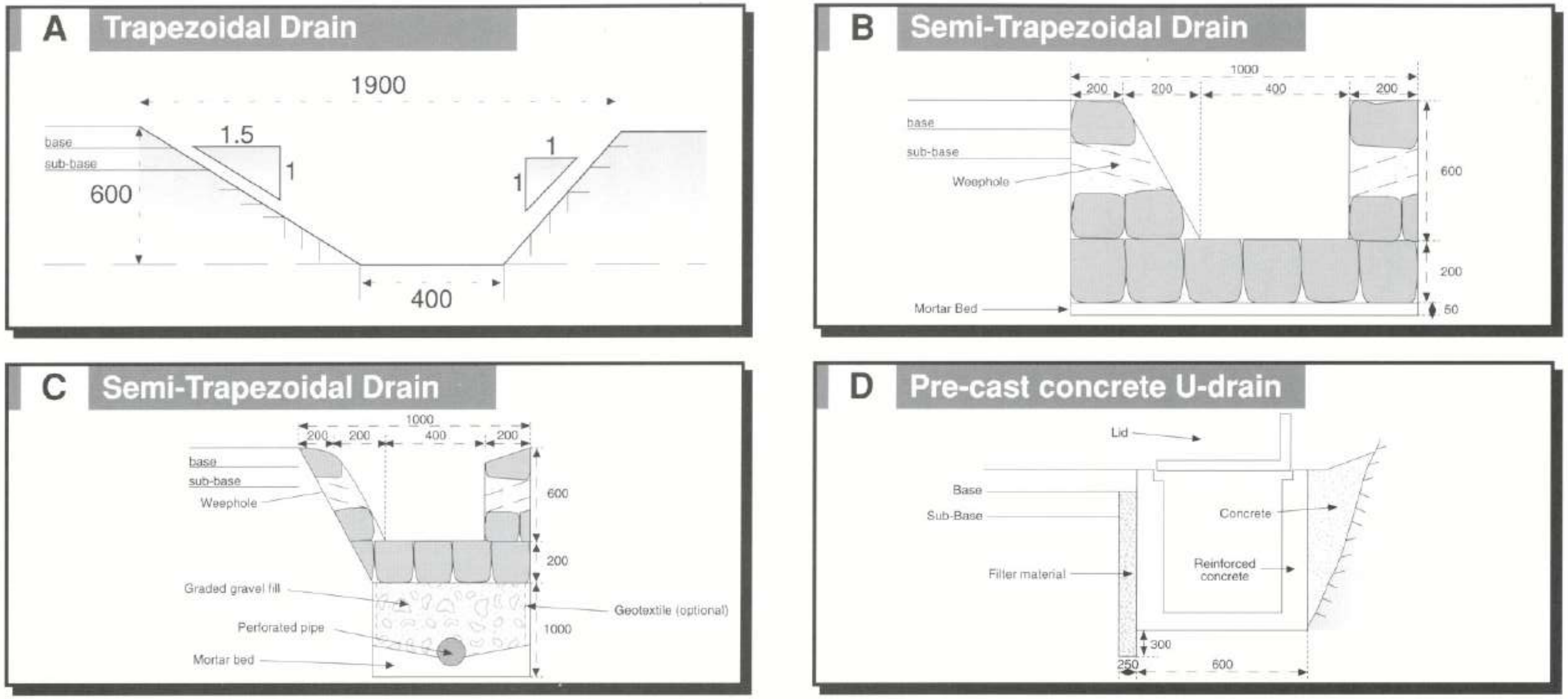
10.11 Where there is no choice but to introduce relief culverts to the road drainage scheme, they should be discharged onto bedrock surfaces, or over convex slope profiles with appropriate protection works, in order to maximise energy dissipation with minimum erosion. The discharge of side drain relief culverts onto virgin soil slopes can result in the rapid erosion of the soil and weathered rock mantle to depths of 5m, or even 10m in extreme cases, which may ultimately undermine the road.

10.12 Road construction across irrigated farmland must not materially disrupt the pattern of water flow supplying the terracing. The principal irrigation channels should be taken beneath the road and this may require a larger number of culverts than normal. When the road is in cut, irrigation water can be conducted across the road via a cascade and culvert in the normal way. In box cut it is necessary to construct an appropriate diversion channel, employing an aqueduct if necessary.

10.13 There are occasions when a culverted stream crossing is impracticable. These are:

- culverts (narrower than 600mm) can become easily silted and prevent easy maintenance. In this case discharges must be small enough to be collected in the side drain and delivered elsewhere (without adversely affecting community water supply)
- when the local topography is too awkward to enable a culvert to be accommodated
- when a constant supply of debris transported through the channel would continually block a culvert. In this case a concrete drift with a shute below is usually the best solution.

10.14 Notwithstanding the above, the control of water and the prevention of erosion should remain the highest priority.



A Trapezoidal Drain
- High capacity
- Suitable only where there is adequate space for construction and where materials can withstand scour
- Comparatively safe to traffic

B Semi-Trapezoidal Drain
- Moderate capacity
- Used where space for construction is limited
- Comparatively unsafe to traffic

C Semi-Trapezoidal Drain
- Moderate capacity
- Used where groundwater is high and drainage is required to maintain water table below road formation
- Comparatively unsafe to traffic
- High cost

D Pre-cast concrete U-drain
- Moderate capacity
- Used where space for construction is especially limited
- Traffic can run on drain cover if necessary
- Difficult maintenance
- Very high cost

Figure 10.1 A selection of side drain types and their typical dimensions

Table 10.1 Flow roughness and velocity in open drains

Material	Roughness `n' value	Max vel m/sec
Sand, silt (unbound)	0.022-0.025	0.3
Loam, fine Gravel	0.022-0.025	0.9
Stiff clay	0.018-0.022	1.2
Good grass Cover	0.025-0.030	
-easily eroded soil	-	0.9
-other soil	-	1.5
Coarse gravel	0.030-0.035	1.5
Rock		
-smooth	0.035-0.040	
-jagged	0.040-0.045	
-soft	-	2.5
-hard	-	5.5
Masonry	0.025-0.030	
Concrete	0.015-0.020	

Culvert design

10.15 In mountainous areas culverts usually operate as hydraulically short drainage structures under conditions of inlet control (culvert flow Type 1, Figure 10.2). Typically, they are sized to flow 75-90% full, with measures to reduce velocities at the outlet. Culvert design procedures are fairly standard and use can often be made of standard graphs and charts. However, it is important to ensure that graphs and charts are appropriate to the culvert flow conditions, and that the correct flow type (Types 1-6, Figure 10.2) is assumed for design.

10.16 It is a false economy to reduce construction costs by minimising the aperture of culverts and allowing them to be surcharged or blocked too frequently. The decision as to whether to design a large culvert or a short-span bridge will depend on design discharge, anticipated sediment loads, configuration of flow, foundation conditions, ease of construction and cost. Short-span bridges may be the only solution when crossing ravines and waterfalls, debris flow channels and debris fans.

10.17 The choice of culvert type depends upon the local topography, sediment load, access to the site, and the availability of materials and local masonry skills. The main options and choice criteria are set out below:

- masonry arches are economic where vehicle loads are comparatively low, and where adequate stone is available (also, they look attractive in a rural environment)

- concrete pipes are comparatively cheap and preferable when sediment loads are not excessive. Vehicular access to the culvert site is necessary. The maximum pipe diameter is 1200mm

- corrugated metal pipes tend to be a more expensive form of construction, although they are portable and easy and quick to install. They are especially suitable in areas where stone for masonry work is in limited supply. Pipe diameter is effectively unlimited. Segments can be portered to site and assembled by hand, an advantage in places where culverts are built in advance of the vehicle access track

- causeways are preferable where sediment loads are high or where large-size debris is expected. The floor can be of masonry or of concrete if extra scour resistance is required

- concrete box culverts are necessary when the required opening exceeds more than about 4m= and when the sediment load is large and abrasive. Boxes are effective in channels sloping at up to 30°, but it is important to prevent settlement and seepage erosion beneath the structure by constructing it on a bedrock foundation. In addition, an apron on the downstream side may be necessary to reduce erosion.

10.18 Various options for culvert configuration are presented in Figure 10.3. Where sediment loads are low to moderate, the combination of a nominally 1 m deep catch pit inlet, a moderately sloping culvert long-section, and sufficient energy dissipation and erosion protection works at the outlet, is recommended (F). Culvert catch pits should be designed to be easily cleared of debris. Drop outlets (D) should be avoided wherever possible, unless the channel bed materials are erosion-resistant. Where there is no choice but to construct a drop outlet, or where scour protection is required beneath an existing culvert, then the apron protection shown in Figure 10.4 may be appropriate. The apron should have a downstand cut-off and raised side walls or wing walls to contain water splash. Where sediment loads are high, a chute inlet (Figure 10.3E), a wide culvert and greater erosion protection works at the outlet are usually required.

10.19 More elaborate methods for dealing with high sediment loads include:

- sediment retention schemes upstream. However, if their retention capacity is much less than the volume of material moving down the stream channel, and if they cannot be emptied regularly, they will simply be

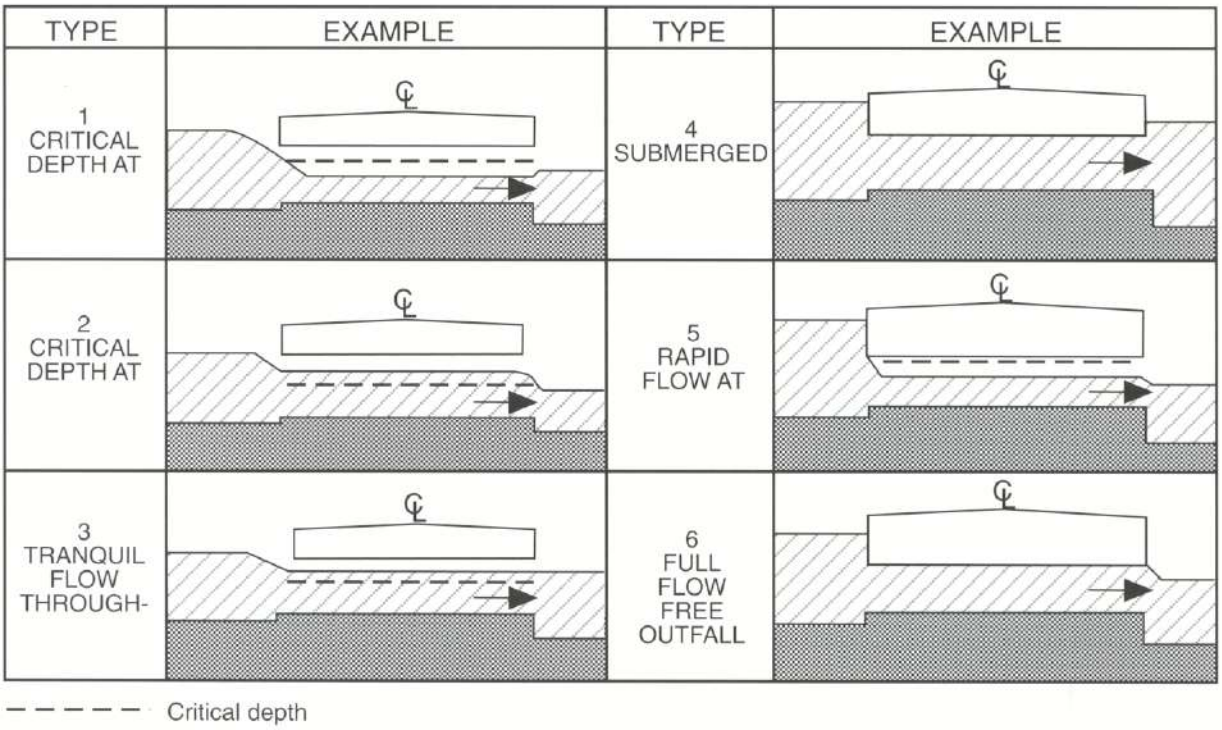


Figure 10.2 Classification of culvert flow

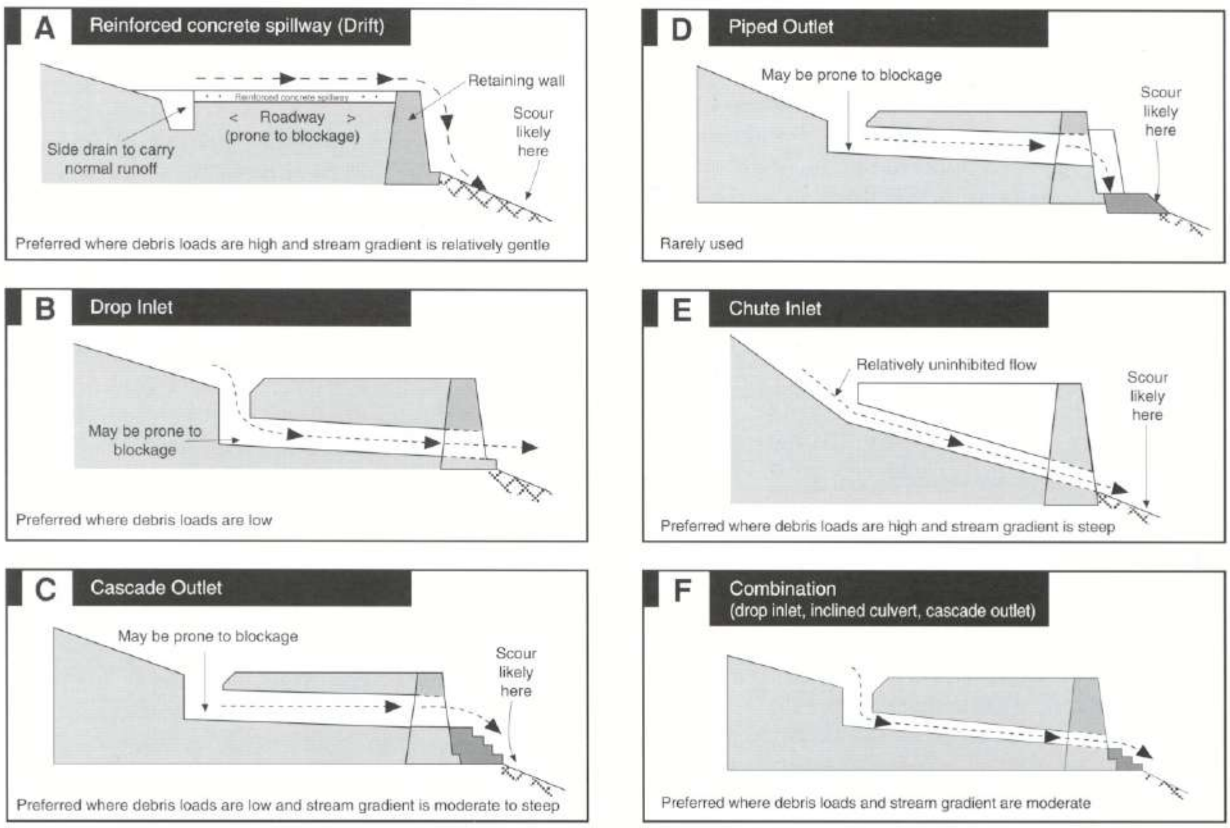


Figure 10.3 Common types of culvert cross-section (not to scale)

overridden by new debris. Although they serve to reduce flow velocities through the creation of lower bed gradients, there is a risk that the raised bed level will cause erosion of the channel sides

- the construction of a reinforced concrete causeway at road level, accepting that the road will periodically become blocked by debris and thus require regular clearance
- the provision of a culvert or bridge with a waterway area substantially larger than that warranted by the design flow alone, in order to accommodate transported debris
- passing the road itself through a tunnel formed as a large culvert, to allow debris to flow over the top. This design is applicable only in extreme cases of debris fan aggradation (Chapter 13), and is inappropriate for low cost roads.

Stream course protection

10.20 It is often impossible to make reliable predictions concerning the full extent of erosion protection likely to be required until the road drainage system is fully functioning and the slopes and drainage channels have responded to the new drainage regime.

10.21 The general design philosophy of stream course protection is to dissipate as much water energy as possible in the vicinity of the road itself, where erosion is likely to be worst, and protect the outfall channel down to a point where it is large enough or sufficiently resistant to withstand the increased flow. Outfall channel protection usually consists of check dams, cascades and channel linings (Figures 10.5 and 10.6). It is not uncommon to build protection works for 20-60m downstream of culverts, and there are instances where they have been constructed for distances of 500m, or more. If investment to this level of protection is considered necessary, it is clearly important to be sure that the measures will be effective. Protection of erodible channels upstream of culverts is usually accomplished by check dams and cascades constructed over much shorter lengths, and usually within 20m of the inlet.

Check dams

10.22 By trapping sediment on their upstream side, check dams create a stepped channel bed profile, thus reducing velocities and channel downcutting, and thereby ultimately halting the progress of erosion. The principle of check dam spacing is that each check dam is placed at the taper of the sediment wedge formed behind the check dam down-stream. However, mountain streams are generally too steep to allow channels to be protected in this way, as this would result in an excessive number of check dams. Furthermore, the final location of each check dam is determined more by the need to find a stable cross-section with strong-points for founding and keying-in the structure, rather than by hydraulic considerations alone.

10.23 Because check dams need to be flexible and free-draining they are almost always constructed in gabion and to heights of up to 4-5m. It is important that their margins are adequately keyed into channel banks, although it may not be possible to achieve this in channels whose banks are composed of soil, river deposits or weak rock. Unfortunately, it is usually under these conditions where check dams are most needed. Where appropriate, bank revetments or side-walls should be constructed upstream and downstream of each check dam to provide additional support and to reduce the potential for scour around the sides of the check dam.

10.24 The strength of a check dam can be increased by constructing it with an arc shape in plan, its convex side facing upstream. However, this may tend to concentrate stream flow against vulnerable stream banks before the sediment wedge has been deposited, or afterwards, if it is later scoured out. Scour and seepage erosion on the down-stream side can be reduced by constructing a masonry or rip-rap toe apron. Where scour potential downstream of the structure is severe, larger scale protection works in the form of gabion or rip rap revetments, masonry sills and cascades can be considered.

10.25 If the primary function of a check dam is to reinforce a natural knickpoint that is being undermined by erosion, and where foundation conditions are good, check dams can be built of bound masonry. These are especially useful in places where space is limited, or where the geometry of the ground is too awkward for a gabion structure. For very small watercourses with low erosion potential, unbound masonry check dams, or check dams built of logs, may suffice.

10.26 Where erosion is already well-advanced, or where foundation conditions are poor, check dam construction may only be possible with large structures, placed on deep foundations or anchored into the bedrock at depth. Plans for such structures must be carefully considered in the light of cost, practicality of construction, probable life of the structure and overall effect.

Cascades

10.27 The principal function of a cascade system is to dissipate the kinetic energy gained by flowing water from a sudden drop in elevation. This is achieved by dissipating energy in increments at each step in the cascade. For discharge rates up to about lm^3/sec per metre width of channel (depending on geometry), cascades are efficient and are an acceptable method of controlling water speed. When discharges are greater than this, the energy-dissipating effect of cascades is reduced, as the water tends to jump

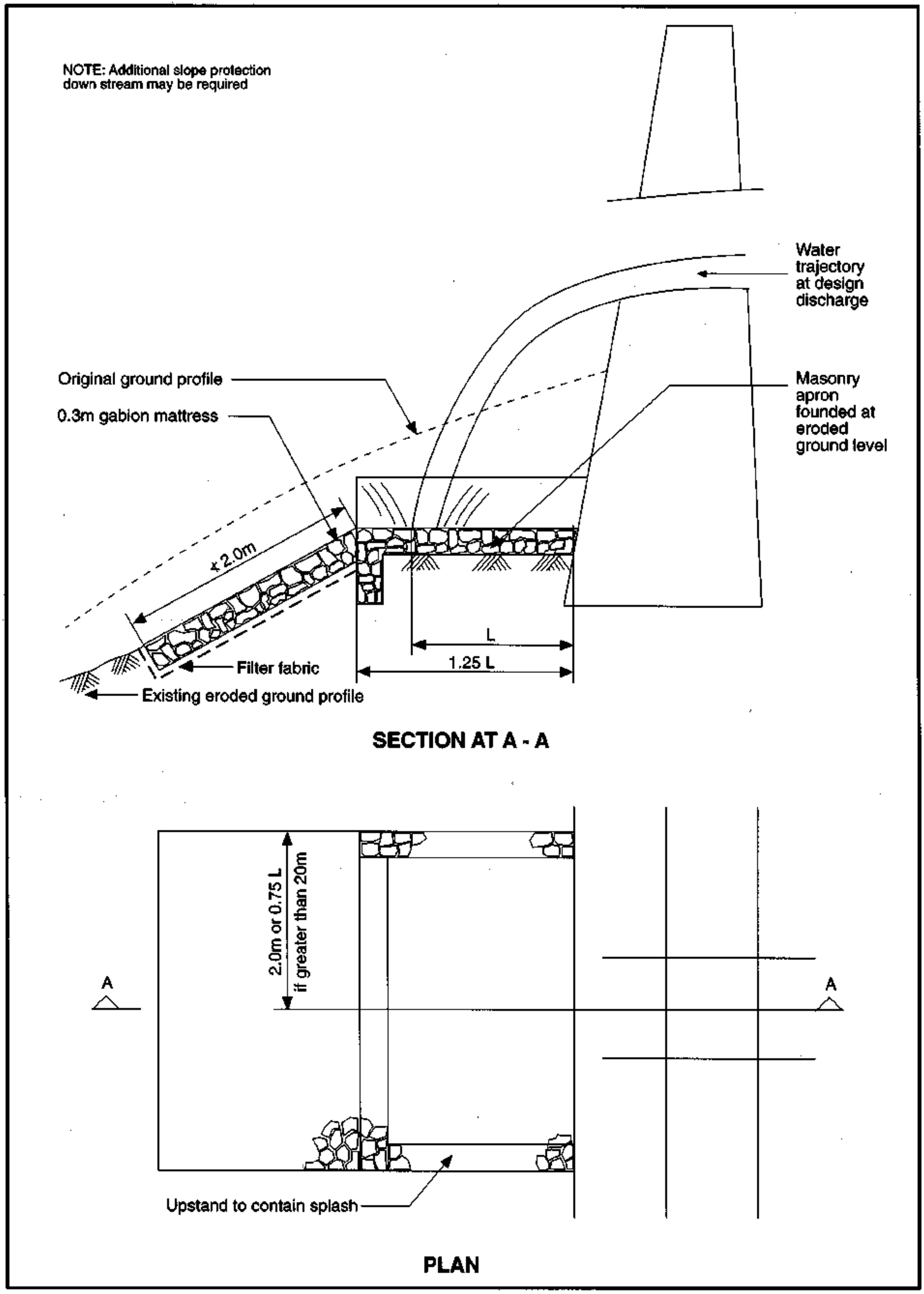


Figure 10.4 Culvert spout outlet apron

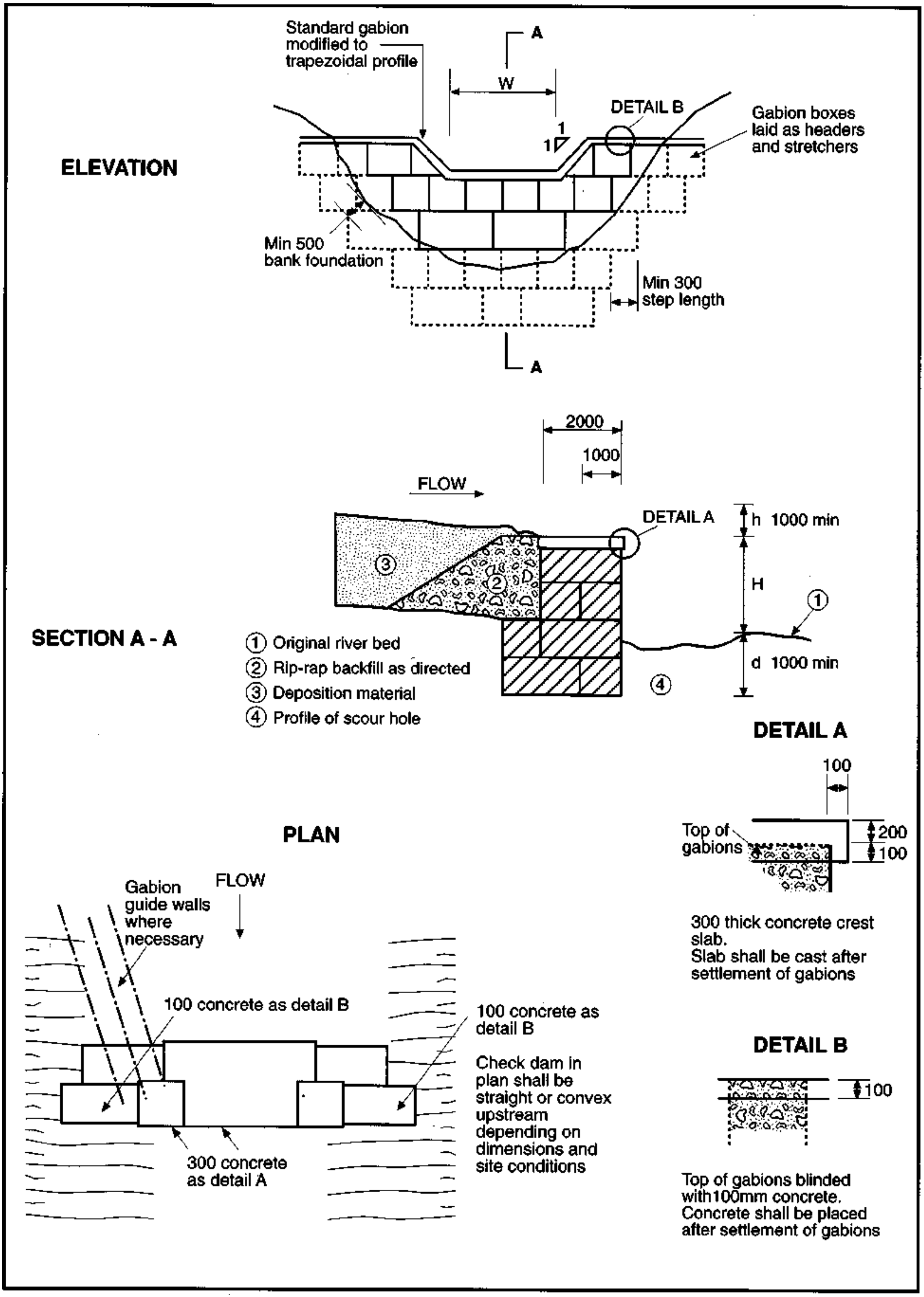
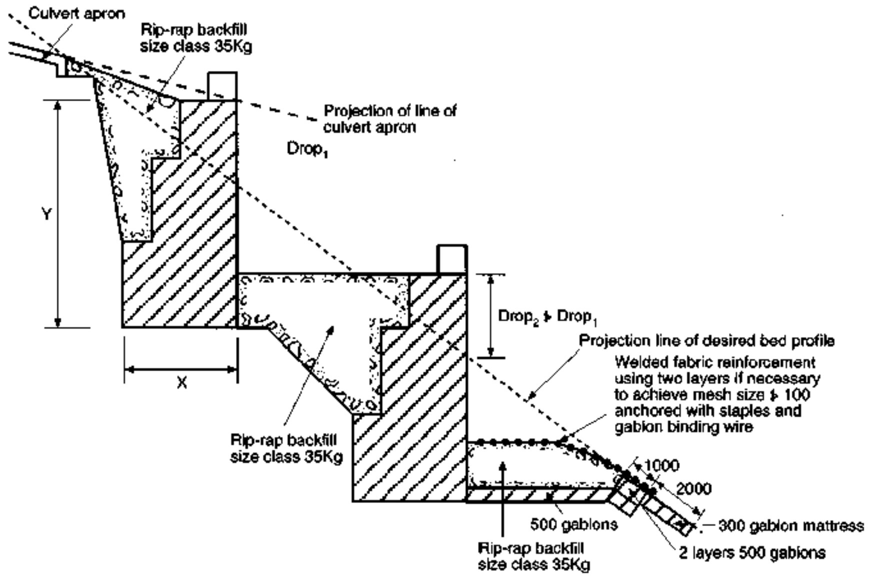
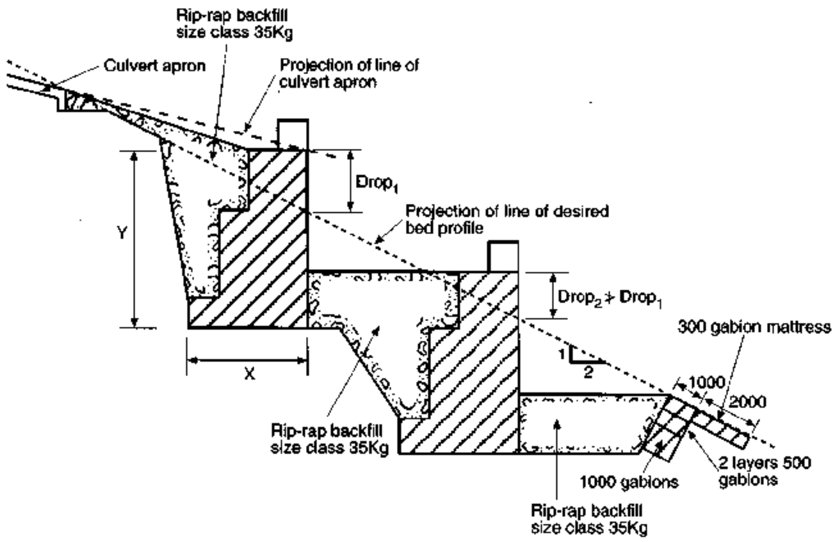


Figure 10.5 Typical gabion check dam details



TYPICAL ARRANGEMENT OF PAIR OF CHECK DAMS DOWNSTREAM OF CULVERT ON STREAM WITH BED OF ERODIBLE MATERIAL. SLOPE 3V: 4H



TYPICAL ARRANGEMENT OF PAIR OF CHECK DAMS DOWNSTREAM OF CULVERT ON STREAM WITH BED OF ERODIBLE MATERIAL. SLOPE 1V: 2H

NOTES

1 Water dropping over check dam or the final check on a series of dams has a high scour and erosion potential which will cause the structure to fail if not controlled. The measures proposed are designed to form a plunge pool for controlled energy dissipation. The use of rip-rap is to provide a guaranteed grading of bed material in the pool sufficient to withstand the turbulence which will occur.

2 Dimensions X and Y determined by slope geometry and stability considerations.

3 The disturbance of beds and banks of streams shall be limited to that required for construction of the permanent works.

4 Rip-rap backfill size class 35Kg shall be used for downstream protectionl when the D_{95} of the bed material is less than the D_{95} of the rip-rap.

5 Welded fabric reinforcement for rip-rap apron shall have a minimum wire size of 6mm.

6 Concrete to crest class 250/20

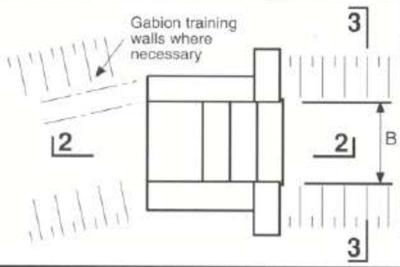
7 Gabion mesh size 80 x 100 mm
Mattress mesh size 60 x 80 mm

8 Foundation bearing surfaces shall be approved by the engineer.

9 Foundation bearing surfaces shall be horizontal. Unsuitable material shall be replaced with suitable fill.

Figure 10.5 Typical gabion check dam details (continued)

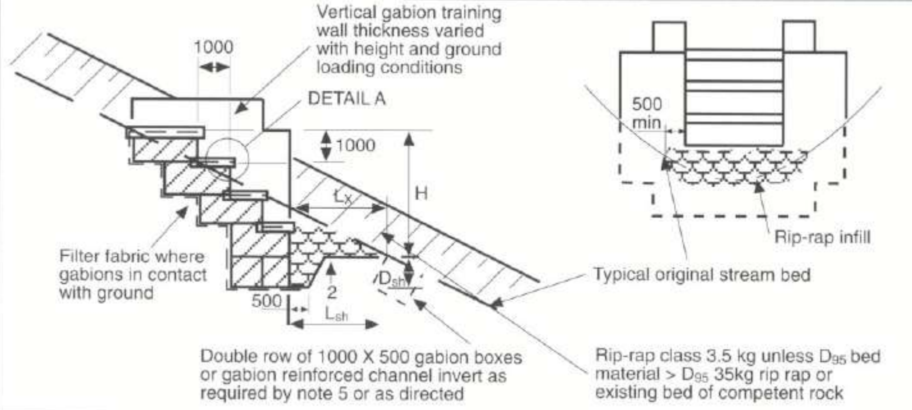
GABION CASCADE TYPE A



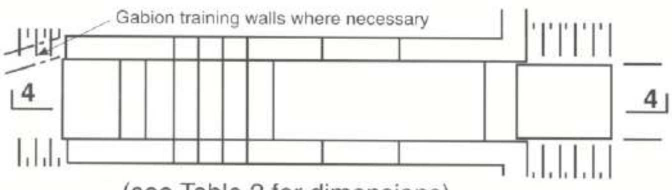
Culvert size	'B'	Drop 'H' 3		4		5		6		7	
		D_{sh}	L_{sh}	D_{sh}	L_{sh}	D_{sh}	L_{sh}	D_{sh}	L_{sh}	D_{sh}	L_{sh}
0.6 dia	2.0	1.0	2.3	1.0	2.4	1.0	2.5	1.0	2.6	1.0	2.7
0.9 dia	3.0	1.0	2.9	1.0	3.1	1.0	3.3	1.5	3.4	1.5	3.6
1.2 dia	4.0	1.5	3.6	1.5	4.0	1.5	4.2	2.0	4.4	2.0	4.6

TABLE 1 (dimensions in metres)

SECTION 2-2 / SECTION 3-3



GABION CASCADE - TYPE B



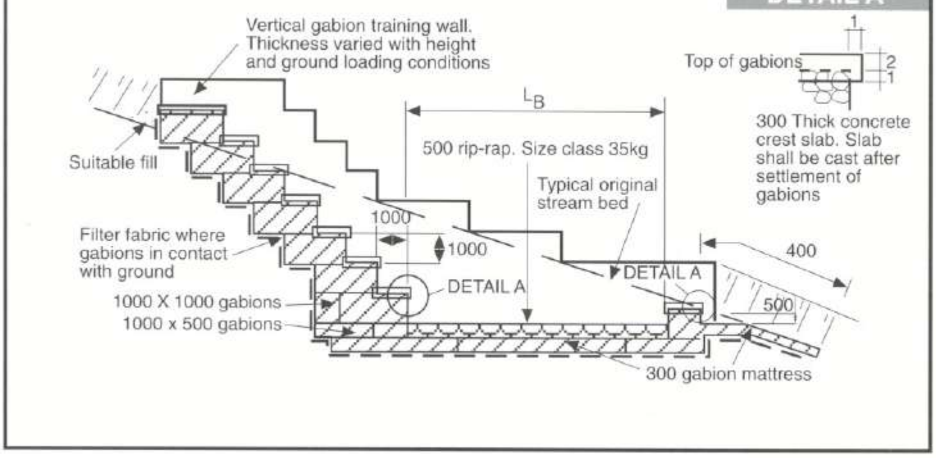
(see Table 2 for dimensions)

Culvert size	'B'	L_B
0·6 dia	2.0	4.5
0·9 dia	3.0	6.5
1·2 dia	4.0	8.5

TABLE 2

SECTION 4-4

DETAIL A



NOTES

1 Gabion cascade type A is for use where ground slope is too steep to accommodate a stilling basin. Gabion cascade type B is for use on shallower slopes where there is room for a stilling basin. It is always important to consider energy dissipation and bed protection downstream of cascades to accommodate the over-design discharge which can be anticipated during the life of the structure.

2 Gabion cascades type A and B shall be used only for watercourses draining through road culverts no larger than the size quoted in Tables 1 and 2 respectively and shall be constructed to the dimensions shown in the tables. At discharges from larger culverts and over-design discharge the cascade will not provide full energy dissipation, and as flow increases will function as a chute.

3 The maximum drop for a type A gabion cascade is 7m as shown in Table 1.

4 The drop / number of steps for a type B gabion cascade may be varied to suit the stream and ground slope.

5 If the dimension L_x is greater than the value of L_{sh} shown in Table 1, then protection work shall be provided to a distance L_x from the end of the cascade. If the value of L_{sh} is greater than L_x then a double row of gabion boxes shall be placed at the point where the rip-rap protection meets the original stream bed (i.e. at a distance L_x from the end of the cascade).

6 Concrete to cascade steps class 250/20

7 Gabion mesh size 80 x 100 mm
Mattress mesh size 60 x 80 mm

8 Foundation bearing surfaces shall be horizontal. Unsuitable material shall be replaced with suitable material.

Figure 10.6 Typical gabion cascade detail

over steps in the cascade, which then performs hydraulically in a similar manner to a straight chute. Very steep cascades, where the rise is much greater than the step, are ineffective for all but the smallest discharges.

10.28 Masonry cascades are recommended only where the channel is formed in relatively intact rock and good founding conditions are available, ie the channel conditions that usually least require protection. Masonry cascades can withstand the erosive effect of sediment-laden flow to a greater extent than gabion, although the latter can sometimes be protected by laying mortared slabs of flat rock on the cascade steps.

10.29 Gabion cascades are preferred where channel foundation conditions are poor, and where some flexibility in the structure is required. They also allow dissipation of hydrostatic pressure from behind the structure. The main disadvantage with gabion cascades is that they are permeable, and seepage erosion beneath the structure can lead to significant deformation. The provision of an impermeable membrane and filter fabric can control this effect.

10.30 The design capacity of a cascade must not be less than the capacity of the waterway in which it is installed. In confined stream courses this may mean that the sides of the waterway have to be cut back in order to preserve a sufficient cross-sectional area. Cascades should always be constructed with side walls that protect the channel banks against side splash erosion. Ideally, the channel banks should be trimmed back so that the side walls can be constructed flush with the original channel. It is important that side walls do not protrude above the height of the channel banks, as this will cause erosion between the bank and the side wall. The steps that form the cascade should be small enough to climb up and down for inspection purposes. Cascades should be wired or keyed into the adjacent road retaining walls on either side and extended down-stream to a strong point in the natural channel. If a strong point does not exist, the cascade should terminate in a stilling basin or plunge pool, possibly with channel protection below.

Channel linings

10.31 A channel bed lining is constructed in situations where the channel is at risk from scour, but where the bed is not steep enough to warrant a drop structure, such as a check dam or cascade. The lining can also be extended up the banks to prevent lateral erosion. When constructing a channel lining it is important to reproduce, as a minimum, the dimensions of the original channel. A dish-shaped cross-section to the bed lining is most preferable. The main disadvantage with channel linings is that a lower channel roughness leads to an increase in flow velocity and hence an increase in scour potential further downstream. In the case of masonry aprons, or gabion mattresses with masonry screeds, some reduction in velocity can be achieved by cementing protruding stones into the surface.

10.32 Channels can be lined with gabion, masonry, dry stone pitching, rip-rap or vegetation. Gabion mattresses can be constructed satisfactorily on channel beds with gradients of up to 30° as long as they are adequately secured at the top and pegged firmly into the channel bed. However, they cannot be made to fit the channel bed as closely as masonry. Their chief advantage is that they are sufficiently flexible to tolerate settlement caused by seepage erosion and scour from beneath. Gabions used for control of water must always be laid over a geotextile or gravel filter whenever the founding materials are potentially erodible. As with all gabion applications, abrasion by stream sediment can rapidly break through the gabion mesh, unless some form of screed is applied. However, a screed is not compatible with a gabions's flexibility, and the design must be based on site conditions.

10.33 Masonry linings can be constructed to fit the stream bed much more closely than gabion. They are also less easily abraded, but they cannot tolerate significant settlements, loss of support by seepage erosion or high groundwater pressure. Dry stone pitching is usually only suitable where discharges are lower than 1 m'/sec per metre width, and where sediment load is relatively fine-grained.

10.34 Channel rip-rap can be used to armour the bed and increase flow resistance, thus decreasing flow velocity. Rip-rap can take the form of boulders or cast-in-place tetrapods, whose size will depend upon the expected flow velocities and scour depths. These can be judged from field evidence of channel scour and transported sediment sizes in association with flow roughness and scour depth computations (Chapter 8). The main drawback with using rip-rap is the thickness of the layer to be constructed, which must be at least 1.5 times the size of the largest stone, and which may require excavation of the bed and banks to be accommodated. Large rip-rap is, therefore, only used to emergency cases, or as a bank armour where flow is concentrated, as on the concave bank of meander bends, or where a channel sharply changes direction. The use of rip-rap as flood plain protection is discussed in Chapter 13.

10.35 Grass, shrub and bamboo planting can provide some resistance to channel erosion and may be used where flow velocities are not expected to be too high. The introduction of vegetation to the channel bed and banks will also tend to reduce flow velocities, although channel vegetation should not be so widespread as to inhibit or divert flow, which could lead to bank scour. Shrubs and bamboo are likely to be the most effective, although the latter can usually only be cultivated in damp sites in warm climates. Where immediate effective protection is required, a structural solution is preferable to a vegetative one.

10.36 The winning of boulders and cobbles from gully beds for road construction materials can reduce the armouring effect provided by coarse material. If the bed material appears to be weathered and static for much of the time, then its removal could expose more erodible sediments beneath. In such cases, extraction from the channel bed should be discouraged or prohibited. Conversely, where the entire bed deposit is fresh and evidently mobile. the removal of material may not have a significant effect on channel stability, especially if the quantities concerned are small compared to the volume of bed load.

Drainage of hairpin stacks

10.37 The disposal of water from hairpin stacks is unquestionably a major hazard, both for the road and for the surrounding hill slopes. It is imperative to ensure that water is discharged into channels that are fully protected. As much effort should go into the protection of these channels as into the protection of the road itself.

10.38 Side drain runoff at hairpin bends is often conveyed via contour drains to discharge into an adjacent catchment, frequently causing severe erosion problems on the slopes and to the channels below. It is preferable, therefore, to contain all water within the stack system itself, thus avoiding the construction of drainage structures remote from the road whose inspection and maintenance might otherwise be overlooked.

10.39 This approach presents two alternatives for design. One is to construct a large reinforced side drain around the outside of each hairpin bend, the other is to install a relief culvert beneath the carriageway at each bend to take runoff into the inside side drain below. The main problem with the former option is the fact that drain failure will lead to erosion of the hill slope below and eventual undermining of the hairpin bend. Although the latter is the preferred option, its main drawback is one of awkward geometry. Side drain runoff is forced to make two ninety degree turns, and the reduced gradient between the inlet and outfall restricts the size of culvert that can be utilized. There is also the cost of these extra culverts and the required greater side drain capacity to be considered.

10.40 Where there is no choice but to discharge water onto a hillslope, the following sites should be sought in order of preference:

- gently sloping or terraced ground
- a slope formed in strong bedrock
- a concave soil slope to assist in energy dissipation.

A standard detail for side drain turnout flow dissipation and erosion protection is illustrated in Figure 10.7. Most turnouts curve in plan, which throws high flows to one side, thus concentrating discharge and increasing erosive power. The flow can be more evenly distributed by providing a flared and baffled outlet.

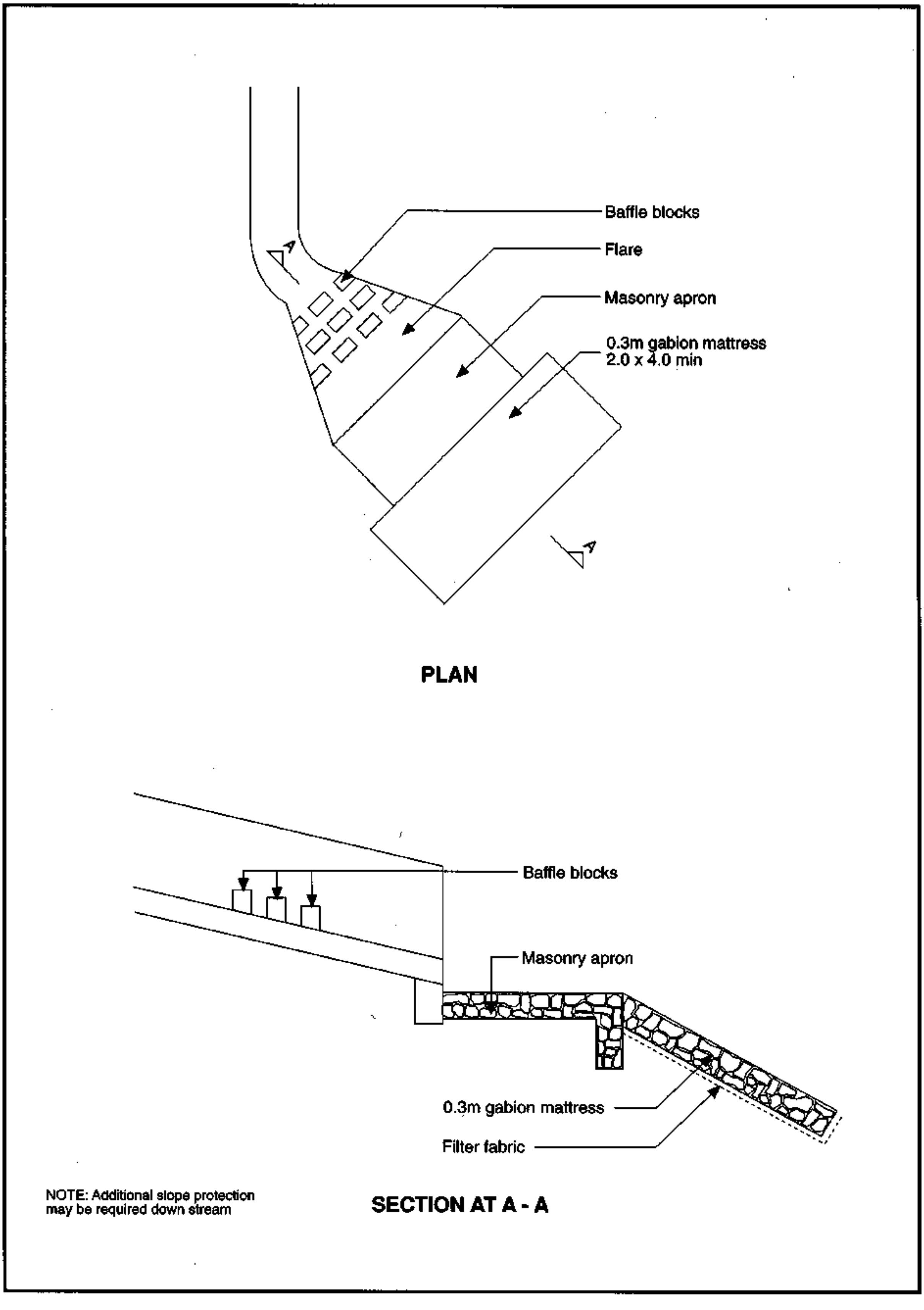


Figure 10.7 Suggested apron details for side drain turnout

11 ROAD RETAINING WALLS

INTRODUCTION

11.1 Retaining walls area common feature of road construction in mountainous regions and can account for 20% of total construction costs. For low cost roads, walls constructed to masonry or gabion are by far the most common, since the bulk of the materials can usually be obtained locally, leaving only cement or gabion wire to be brought from other sources. They are constructed for the following situations:

- to support a road either wholly or partly on fill when the ground profile is too steep (usually greater than 30 degrees) to allow an embankment slope
- to support the toe of a slope that has failed or is likely to fail
- to support cut slopes that would otherwise require a low, uneconomic angle of cut
- when there are constraints on the permissible plan extent of earthworks, as on hairpin bends, or hairpin stacks, and in densely populated areas
- as revetments to prevent erosion on steeply sloping cut faces (Chapter 12) as part of a slope stabilisation scheme.

11.2 Problems of access in steep terrain can make it impracticable to carry out sufficient foundation investigation during the pre-implementation stages of a project. This may prevent the compilation of definitive designs or even delay decisions on appropriate wall types. These circumstances lead to the following recommendations regarding the general approach to retaining wall design:

- it is often better not to design walls in great detail in advance of construction, but instead to provide standard designs and to make adequate provision in construction contracts for detailing designs at that stage;
- standard designs should cater for the range in the type of wall and cross-section that can be adopted under varying topographical conditions, foundation bearing capacity, and to suit locally available materials and expertise, thus encouraging competitive pricing.

11.3 There are practical considerations in the design and construction of retaining walls:

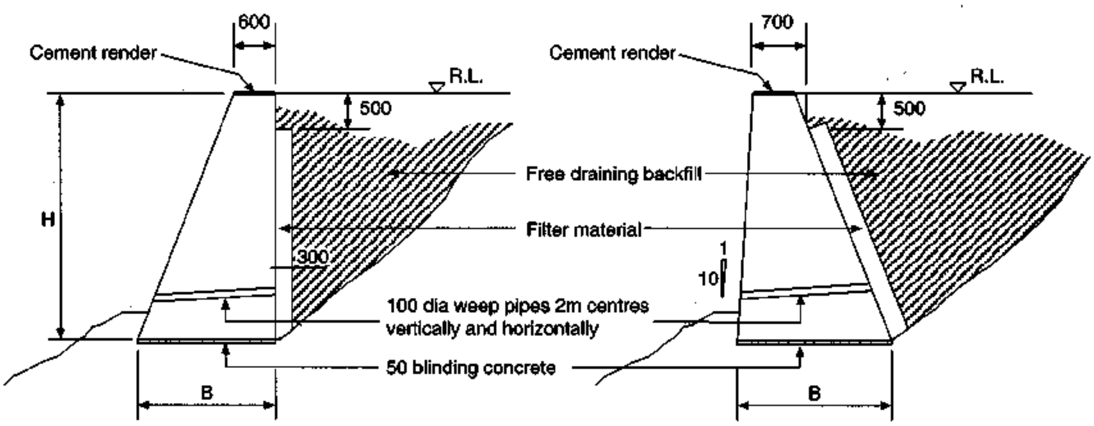
- in the case of road widening and reconstruction, it may be necessary for construction to be corned out while maintaining access for traffic, in which case the extent to which the wall foundation excavations intrude into the roadway must be considered
- construction of retaining walls often takes place early in the sequence of works and porterage of constrution materials, and the availability of trained labour, may affect the choice of wall type
- permanent drainage is always an important consideration, but it may be necessary to consider the provision of temporary drainage measures if wall construction or backfilling will not be completed before the onset of a wet season
- particular care should be exercised in assessing foundation conditions in previously disturbed soils, such as is often the case on road improvement or reconstruction projects where material from earlier slides may have been tipped adjacent to the road. If it is contractually feasible to open up wall excavations before deciding on the form of wall construction, then this should be done
- road retaining walls are usually designed to a standard cross-section on the basis of assumed achievable bearing capacities, horizontal backfill slope, backfill friction, permeability and drained conditions. Toe walls with sloping backfills, and walls constructed to retain failed or failing slopes, do not lend themselves so readily to a standard approach, and usually will require separate design considerations.

WALL TYPES

11.4 Typical details of the most commonly used wall types are shown in Figures 1 1.l and 11.2.

Dry masonry

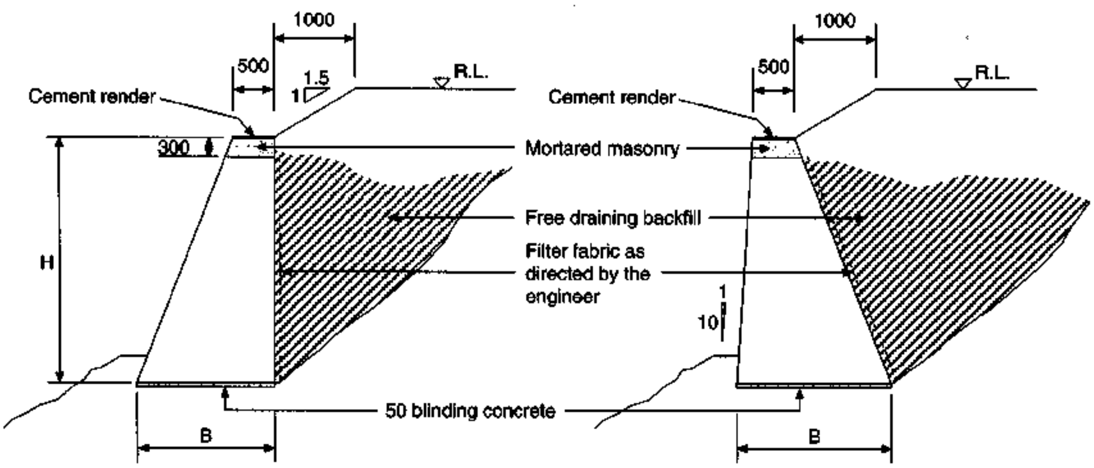
11.5 Dry masonry (unbound masonry or dry stone) walls are usually the cheapest form of walling and are suitable for heights of up to 3-4m. They should not be used as road retaining structures for heights greater than 4m, and preferably not greater than 3m, although there are cases where dry masonry walls have performed competently at heights greater than these. Wall failure is usually caused by lateral loads in the backfill, seepage pressures and vibration from heavy vehicles. In respect of the last factor, dry masonry walls should not be constructed within 1 m of the road edge. The width to height ratio varies between 1:1 and 0.6:1 for wall heights of Imto4m respectively. Skilled masons and suitable stone are required for the construction of quality dry stone walls, because a neat fit and good interlock of the stonework are critical to the wall's strength and stability. The selection of masonry stone should be subject to strict quality control, as should be the construction of the wall itself.



Profile 1 **Profile 2**

BASE WIDTHS AND TYPICAL GROUND PRESSURES									
Wall height (H)m		2	3	4	6	8	10	12	15
Profile 1	Base width (B)m	1.4	1.8	2.2	3.0	3.6	4.6	5.4	6.6
	Ground pressure T/m^2	4.0	6.0	8.0	12.0	16.0	20.0	25.0	31.0
Profile 2	Base width (B)m	1.5	2.0	2.5	3.5	4.5	5.5	-	-
	Ground pressure T/m^2	8.0	13.0	17.0	25.0	33.0	41.0	-	-

Mortared random rubble masonry retaining wall

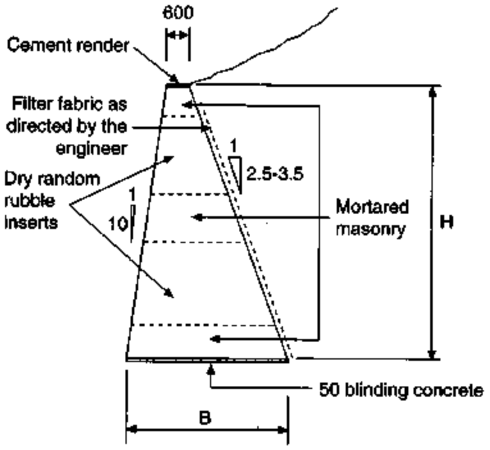


Profile 1 **Profile 2**

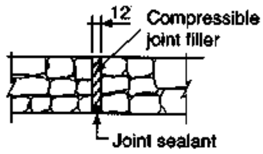
BASE WIDTHS AND TYPICAL GROUND PRESSURES					
Wall height (H)m		1	2	3	4
Profile 1	Base width (B)m	0.9	1.3	1.7	2.1
	Ground pressure T/m^2	2.0	3.5	5.5	7.5
Profile 2	Base width (B)m	1.0	1.5	2.0	2.5
	Ground pressure T/m^2	2.5	4.5	6.5	8.5

Dry random rubble masonry retaining wall

Figure 11.1 Typical details for masonry road retaining walls

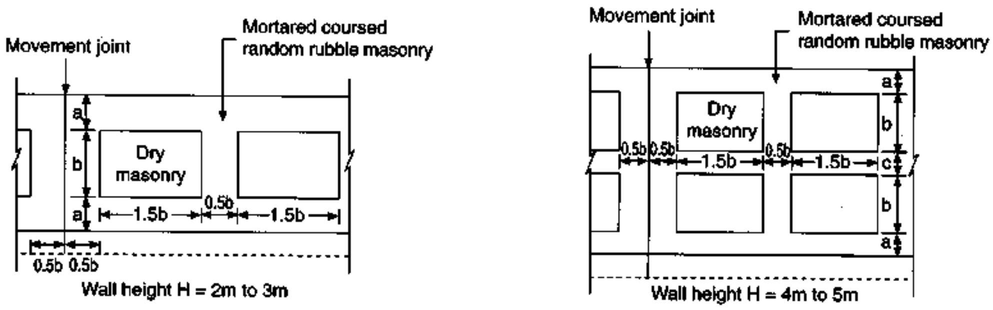


Typical Section (dimensions and ground pressure dependent on grade of slope)



Movement joint detail

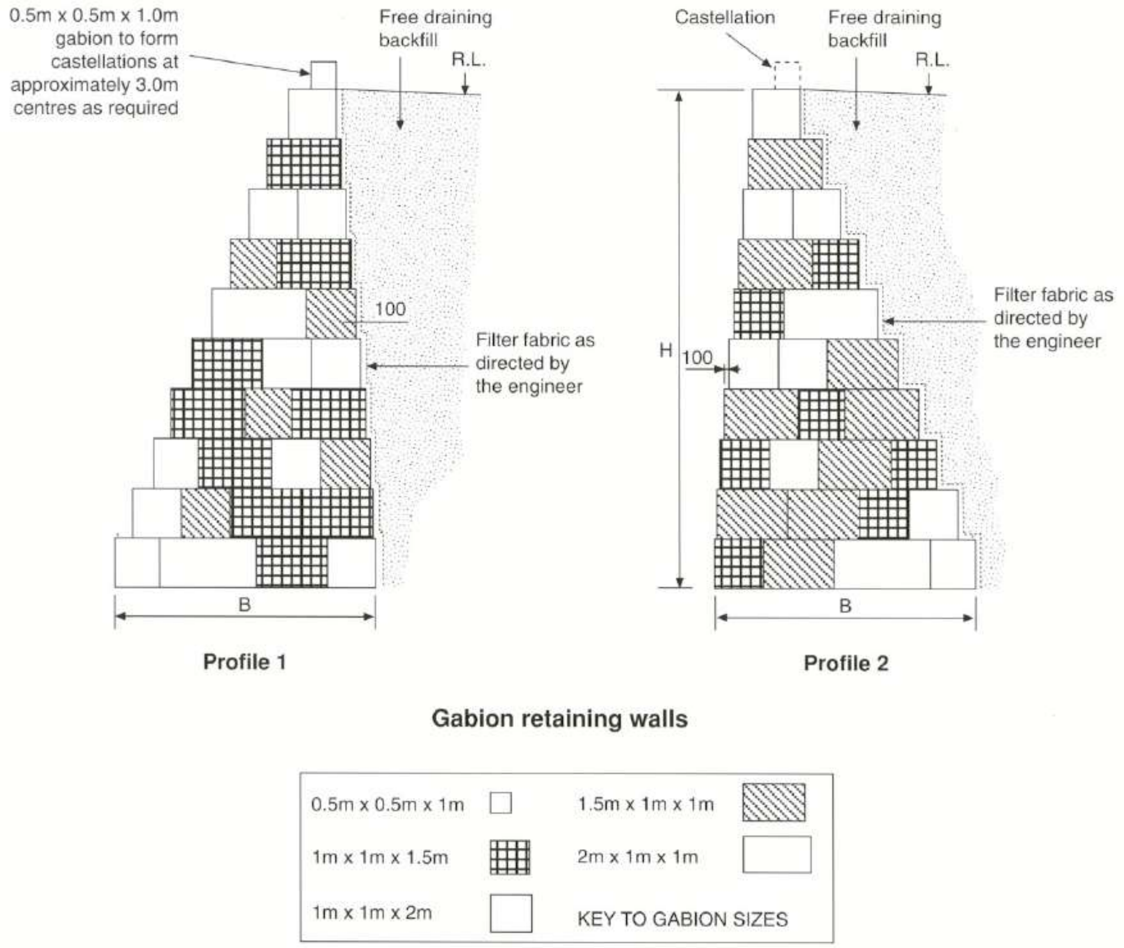
PANEL LAYOUT DIMENSIONS					
Wall height (H)m	2	3	4	5	6
Dimension (a)m	0.6	0.75	0.6	0.6	0.75
Dimension (b)m	0.8	1.5	1.15	1.4	1.75
Dimension (c)m	-	-	0.5	1.0	1.0



Elevations showing typical layout of random rubble panels

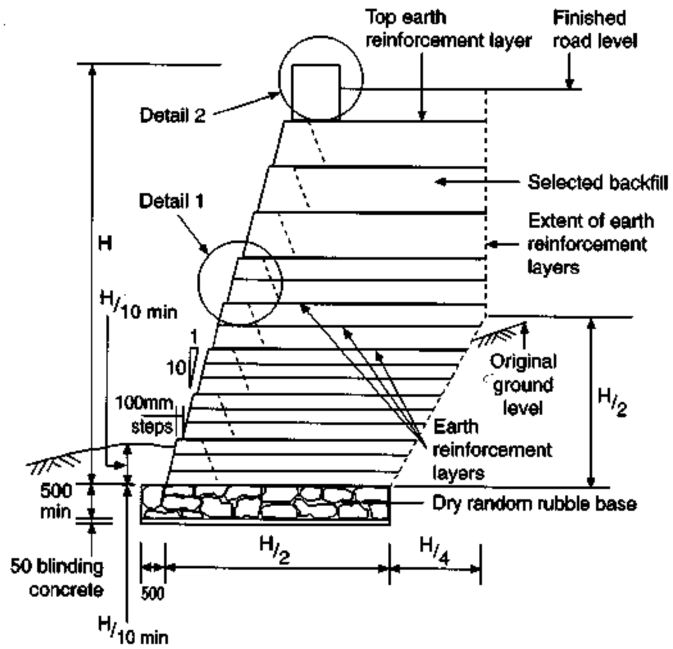
Composite random rubble masonry retaining wall

Figure 11.1 Typical details for masonry road retaining walls (continued)



BASE WIDTHS AND TYPICAL GROUND PRESSURES								
Wall height (H)m		2	3	4	6	8	10	12
Profile 1	Base width (B)m	1.5	2.0	2.5	3.5	4.5	5.5	6.5
	Ground pressure T/m^2	50	70	90	120	150	190	230
Profile 2	Base width (B)m	1.5	2.0	2.5	3.5	4.5	5.5	6.5
	Ground pressure T/m^2	70	110	150	220	290	360	420

Figure 11.2 Typical details for gabion and reinforced earth retaining walls

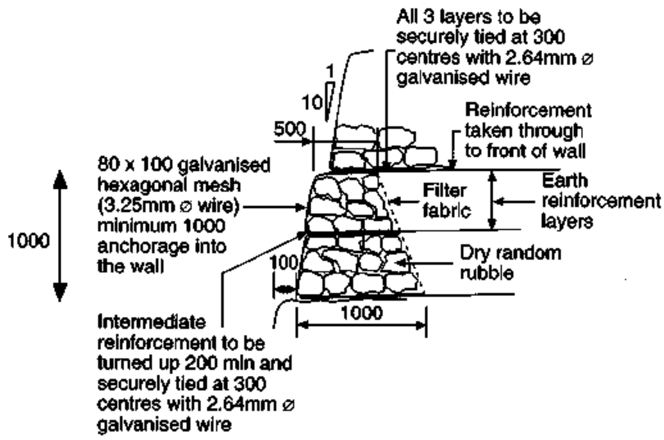


Reinforced earth retaining wall

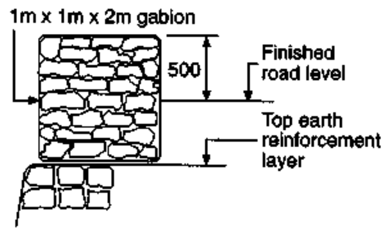
EARTH REINFORCEMENT DETAILS										
Wall height (H)m	5	6	7	8	9	10	11	12	13	14
No. layers at 1000 crs	4	4	4	4	4	4	4	4	4	4
No. layers at 500 crs	2	4	6	6	6	6	6	6	6	6
No. layers at 333crs	-	-	-	3	6	9	12	15	18	18
No. layers at 250 crs	-	-	-	-	-	-	-	-	-	4
Total no. of layers	6	8	10	13	16	19	22	25	28	32

Top of wall

Base of wall



Detail 1



Detail 2

Figure 11.2 Typical details for gabion and reinforced earth retaining walls (continued)

Masonry with dry stone inclusions (composite walls)

11.6 Composite masonry walls are similar to mortared masonry walls except that they have panels of dry masonry about 0.6-1 m square forming a grid on the face with 0.5m division strips. They are stronger than dry masonry walls and, at the same time, maintain the advantage of relatively free drainage. They are used frequently in many mountain regions as slope support and revetment structures in cuttings through weak rocks, but are difficult to specify and construct as road retaining walls in complex and undulating side-long ground. It is doubtful whether they result in any significant savings in construction cost over ordinary masonry walls as they are more time consuming to construct.

Masonry walls

11.7 Mortared masonry walls are the most durable of the low cost wall options. They are especially suited to steep rocky ground where foundations are shallow and where the contractor's working area is restricted.

11.8 The base width of these walls is usually between 0.5 and 0.75 times the wall height, and their base should be stepped into a rock foundation or constructed on an unreinforced 300mm thick concrete base, if rock does not comprise the foundation. If the height of the wall varies along its length, (that is, if the base steps down and up to accommodate changes in founding level), the location of vertical joints should reflect the position of foundation steps, and the frequency should be adjusted to minimise differential wall movements.

11.9 Masonry walls are not tolerant of differential settlement. Their lack of structural flexibility and general impermeability, even with freely-draining back fill and weep holes, make these walls inappropriate on wet colluvial slopes and where ground movements are expected, although some cracking in a masonry wall can be tolerated provided it is not accompanied by major deformation. There are many instances where severe cracking and sometimes complete failure have resulted from wall construction across adjoining soil and rock foundation materials. Weep holes should always be connected to gravel interceptor drains in the back-fill, and the lowest weep hole should be no more than 0.2m above final ground level at the toe of the wall.

11.10 The skills required to construct masonry walls are widely available in many populated mountainous areas. They are easier to build than dry stone walls because any lack of fit between the stones is taken up by the mortar. Rounded masonry stone should represent a maximum of a third of the total stone content. The minimum dimension of tabular stone should be at least 50% of the maximum. It is important to ensure that the wall interior is not filled with dry stone rubble as a cost-cutting exercise.

Gabion walls

11.11 Gabion walls are usually preferred under conditions of poor foundation, wet soils, high groundwater and slope movement due to creep, landsliding and seismicity. However, the density of gabion work is approximately 70% that of mortared masonry and hence a gabion wall has to be larger in volume for the same retained height. This increase in size could be a factor in the choice of wall type in situations where cross-sectional space is limited. Gabion walls have the following characteristics:

- they can accommodate settlement without rupture
- they allow free drainage through the wall
- they can be constricted in short sections (2-3m at a time) which minimises the temporary loss of slope support during excavation
- the shape of a gabion structure can easily be varied to accommodate changes in ground conditions across the site
- a gabion structure is less easy than a masonry structure to fit into an irregular foundation because of the standard size and rectangular shape of the boxes.

11.12 Being an unbound structure, the strength of a gabion wall depends upon the mechanical interlock between the stones. The strength of a gabion wall where the boxes have been filled by hand can be far higher than that of one where they have been filled mechanically, because the stones can be packed to a higher density. For high densities to be achieved it is important that as much attention is paid to the packing of the infill stones as to that of the facing, although rigorous site control is necessary for this to be assured. The joints between gabion boxes should be spanned by frequent `stretcher' boxes, as for bonded brickwork, orientated both along the wall and from front to back. These will restrict pulling apart and bulging. Long, flat stones should be orientated from front to bark in order to resist diagonal shear forces that pass through the wall. Where river terrace deposits are likely to form the predominant source of gabion stone, rounded stone should be limited to a third of the total stone content, and evenly distributed throughout each box. All other stone should be angular, or dressed to make it so. Further details of gabion wall construction are given in Box 11.1.

11.13 Where a gabion wall is founded on an uneven bedrock surface, the lower boxes will deform into the hollows, and the sliding resistance will not be as high as for a wall with a masonry or a concrete slab base. If bedrock is encountered at a shallower depth than expected, which prevents the intended design level from being reached with boxes of standard height, the lowest course can be built of mortared masonry up to the level of a course of boxes,

Box 11.1 Construction of Gabions

Gabion structures offer remarkable qualities in terms of strength, flexibility and free drainage. However, they must be built to a high standard if these attributes are to be attained. The following guidelines are recommended

- ensure drainage is provided from the lowest point of the foundations
- use high grade wire with a thick galvansing
- mesh should be either a proprietary welded mesh or a triple-twist hexagonal mesh of 100mm width and 120mm length
- panel frames should be made using 8 swg wire, and mesh should be made using 10 swg wire
- wire all gabion boxes together using additional wire of 10 swg
- during construction, add four or five cross-trusses (of 10 swg wire) per square metre in each horizontal dimension
- ensure that the minimum dimension of all stones is larger than the wire mesh size
- stones should be tabular, of even size, and angularity
- if the boxes are packed by hand, all of the stones should be carefully packed, not just the facings
- wire the lids down with additional wire of 10 swg
- backfill behind the gabion structure using even-sized filter material.

above which construction can proceed in gabion as planned. The top of a gabion wall can be finished with a course of masonry if a seal is required between the road surface and the wall, in order to form an effective road drainage channel.

11.14 The preferred maximum height of a gabion road retaining wall is 10m, although walls of up to 14m have been constructed on occasions with only minor deformations. Under high lateral and vertical loads there is a potential for stone crushing, especially in the toe of the structure. Crushing and compaction of the lower courses of gabion boxes in this way can result in a volume loss of as much as 20/0. If the stones in the toe of the wall crush under the load of those above, the effect on the wall is to settle at the toe, causing the top to tilt outwards and increase the load on the toe. Another effect is that the fill behind the wall will settle and could subject the wall to unpredictable shear stresses.

Reinforced earth

11.15 Reinforced earth walls have not been used to any great extent in developing countries, due primarily to a lack of design expertise and the lack of appropriate backfill compaction plant. The main advantages of reinforced earth walls are their ability to deform without significant loss of serviceability at heights of up to 16m and the fact that they can be constructed with a range of fills. They can be useful in places where stone for gabions is not available within economic haul distance of the site.

11.16 The principal disadvantages with these walls are:

- the cost of, or difficulty of obtaining, the steel strip or geotextile reinforcement
- the need for a high level of compaction, including wall edges, which can only be achieved with mechanical plant
- the requirement for an adequate horizontal clearance between the facing of the wall and the natural ground in order for the reinforcements to develop the required tension resistance. This implies that a) the ground may have to be cut back further than for a conventional retaining wall, and b) there will be additional delay before the space between the wall and the hillside is backfilled, thus increasing the possibility of local failure developing in the hillslope
- a higher cost than masonry for walls less than 10m high.

11.17 Reinforcing elements maybe difficult to obtain, and for most developing countries they would have to be imported. Steel reinforcing strips may be susceptible to corrosion and sufficient sacrificial allowance has to be made in determining strip thickness, depending on the aggressiveness of the back-fill material. Geosynthetics are not subject to corrosion, but they possess low stiffness relative to steel and the amount of deformation required to achieve maximum shear strength is higher. They also require protection from ultra-violet light. Geogrid does not require the use of expensive and unattractive concrete facing panels. Instead, the reinforcing can be wrapped around the outside face of the wall from one layer to the next. The face of the wall can then be covered with soil and vegetated.

Buttress-supported road slabs

11.18 Buttress - supported road slabs (Figure 11.3) have been used very successfully to extend the width of a road beyond the outer face of an existing masonry retaining wall, in places where a cut into the hillside is unfavourable and where good founding material exists upon which to found the buttresses. The buttresses act as piers and carry a reinforced concrete deck. They may be used to heights of 12-15m, although this is dependent on good rock foundations. Designs that obviate the need for ground supported scaffolding as falsework are to be favoured. Buttress-supported road slabs could equally be used in new road construction in steep sidelong ground, where appropriate.

Mass concrete and reinforced concrete walls

11.19 Neither of these wall types are generally suited to road construction in mountainous areas as they require large quantities of cement and crushed aggregate, and are relatively expensive. Reinforced concrete walls also require a greater diversity of technical skills to ensure that a good standard of construction is achieved. Reinforced concrete walls are most appropriate as toe retaining structures to unstable slopes in association with rock anchors, where foundation and anchoring conditions permit. Anchored sheet pile and caisson walls are rarely applied in a low cost situation, except where sheet pile is used as a temporary shoring measure.

STABILITY, CROSS-SECTION AND ARRANGEMENT

11.20 The design of retaining walls is covered in standard texts, some of which are referenced in Chapter 14 (Bibliography). Only those factors especially relevant to wall design on steep, wet and frequently unstable mountain slopes are considered here. Wall design and construction in riverside locations are discussed in Chapter 13.

11.21 Both standard designs and detailed site designs should have a sufficient factor of safety against overturning, sliding and bearing capacity failure. Under normal loads, where a wall is designed to support a horizontal backfill slope, a minimum factor of safety of 2 against overturning, 1.5 against sliding or shearing and 3 against bearing capacity failure is applicable. A minimum internal shear strength of 600kPa is desirable.

11.22 The choice between walls with a horizontal base (upright) or walls with a base slightly inclined into the wall (back-sloping) will require consideration of cost, the availability of fill, stability and ease of construction. Some of the more obvious advantages and disadvantages with various wall cross-sections under different ground conditions are shown in Figure 11.4. Although upright retaining walls are usually taller and therefore more expensive on steeply sloping ground than back-sloping walls, they are generally easier to construct and allow better compaction of backfill. Another advantage is that base pressures are more evenly distributed, and therefore compressive stresses at the toe of the wall will be lower. However, on slopes underlain by hard rock, this advantage may not be significant. Although

Box 11.2 Retaining wall design in Nepal

Road retaining walls in Nepal are generally designed for static loads only, and consequently retaining walls and backfill must be in place soon after slopes are excavated. Stresses induced by seismic or shear loading are not normally catered for. The recently observed effects of seismic acceleration on retaining structures indicates that masonry walls, especially those with foundations less than 1.5m deep, are far more susceptible to damage and failure than gabion walls, because they tend to rotate outwards as rigid structures due to shaking of the wall and settlement of the backfill. By contrast, gabion walls up to 10m high have been observed to remain stable with foundations as shallow as 1-1.5m. Bearing pressures under seismic loading have been calculated to be in excess of 300kPa for masonry walls of 7m height. Foundation depths of 2-2.5m have been recommended for the design of retaining walls in earthquake-prone areas, but the favourable performance of gabion walls in this instance suggests that, for these structures, the recommendation can be relaxed.

Back-sloping gabion walls have an inherently greater resistance to sliding, they have not been favoured in many situations, as the angle of batter makes construction work awkward, and the foundation of the wall needs to be drained in the same way as masonry, thus increasing the cost.

11.23 The type of wall to be built at any one location is selected on the basis of ground conditions at the site and construction cost, although the latter is almost invariably the deciding factor. While there is considerable latitude in matching the type of wall to ground conditions, there are circumstances under which the wrong choice of structure will lead to deformation or even failure of the wall.

11.24 The most suitable form of retaining wall and its cross-sectional area depend upon:

- whether the best design from a geotechnical standpoint provides an adequate roadway width
- available cross-sectional area of the site
- depth and volume of material to be excavated to foundation level
- lateral stress expected to be applied to the wall
- bearing pressure of the wall and bearing capacity of the foundation
- geology and groundwater conditions
- whether a flexible structure is necessary

- extent of subsurface drainage required
- availability of construction materials
- availability of construction skills
- method of construction.

Backfill

11.25 A well compacted backfill is vital to the serviceability of the road above a wall, and possibly to the wall itself. The consequence of poorly-compacted backfill is that, in time, settlement will cause the road to subside and crack, causing unpredictable lateral shear stresses within the wall. Most walls are designed on the basis of a specified backfill Ö' value under drained conditions. It is important to ensure that these design parameters are achievable under site conditions. Ideally, backfilling and compaction should keep pace with the wall as it rises, to give as much space as possible to deliver a good compactive effort. Compaction in layers of 200-300mm is usually specified. Special care is required at the base of the excavation behind the wall, where it is difficult to achieve good compaction because the working area is very restricted.

Foundations and sub-surface drainage

11.26 If the toe of the excavation comes within about 0.5m of the slope surface it is convenient to dig away the upstanding portion of soil to form a continuous step along the front of the excavation. This platform provides a wider working area, and ensures that there is no obstacle to drainage in front of the wall. It is often left to the engineer on site to decide whether a particular wall should be founded at its design foundation level or whether excavation to a deeper, stronger level should take place. Simple hand probing tests, such as the Mackintosh probe and the dynamic cone penetrometer, when suitably calibrated, can provide a quick assessment of bearing capacity, and can augment the more qualitative assessments.

11.27 Where shallow colluvium (1-2m) is encountered, foundations should be excavated down to stable rock head or weathered rock/residual soil materials below the colluvium. Where the depth of colluvium is too great, a gabion wall will probably be the most appropriate choice.

The allowable bearing pressure for any area of wall construction is dependent on:

- form of construction (a higher pressure can be used for settlement-tolerant structures)
- ground conditions at foundation level
- topography of slopes below the base of the wall.

11.28 For high walls (4-7m) the bearing pressures, particularly beneath the front edge of the structure can become considerable, often in excess of 200kPa under static loading conditions. For masonry walls of whatever profile, a toe slab or toe projection at the base should be specified to reduce compressive stress and scour due to runoff over the wall surface.

11.29 A 0.5m gravel and pipe drain should be constructed along the back of a masonry wall foundation to facilitate drainage. In addition, gravel drains (grips) should connect the rear drain to the front of the wall at not more than 20m intervals. This drainage feature is essential for walls with a back-sloping base. Where a wall has a stepped foundation to fit into a topographical depression, water will move along the wall, and accumulate at the lowest point in the footings. A sub-surface drainage system is required here to collect water and deliver it safely into the outfall. Ideally, weep holes should be provided in the concrete base of masonry walls founded on soils to allow drainage. The foundation platform for gabion walls built without a basal slab must be either naturally freely-draining, or have drainage grips installed.

Surface drainage

11.30 The top of the wall is often finished with an upstand that rises to about 0.75m above the road surface, or serves as the base for a series of roadside edging blocks or parapets. This edge prevents water from draining randomly over the edge of the wall where the road has an outward camber. A bituminous fillet can be formed to prevent water entering a gabion structure, and to convey it to the end of the wall. However, even a slight easing of a gabion wall, or cracking of the fillet, will allow water to ingress into the backfill, and it may be appropriate to install a filter drain behind the gabion walls to a maximum depth of lm below the road in order to prevent seepage erosion of the finer materials from the sub-base.

11.31 If the wall is on a sag curve, then water collected at the low point can be conveyed through a gap constructed in the upstand, down to a stream course with erosion protection, as required.

11.32 The slope at the end of the wall is vulnerable to scour from water running off the road behind the wall. The outfall should be protected by a slope revetment formed into a channel, and delivered to a safe discharge point.

11.33 The slope in front of a gabion wall, especially, should be free-draining (all upstanding or poorly-drained material will need to be removed).

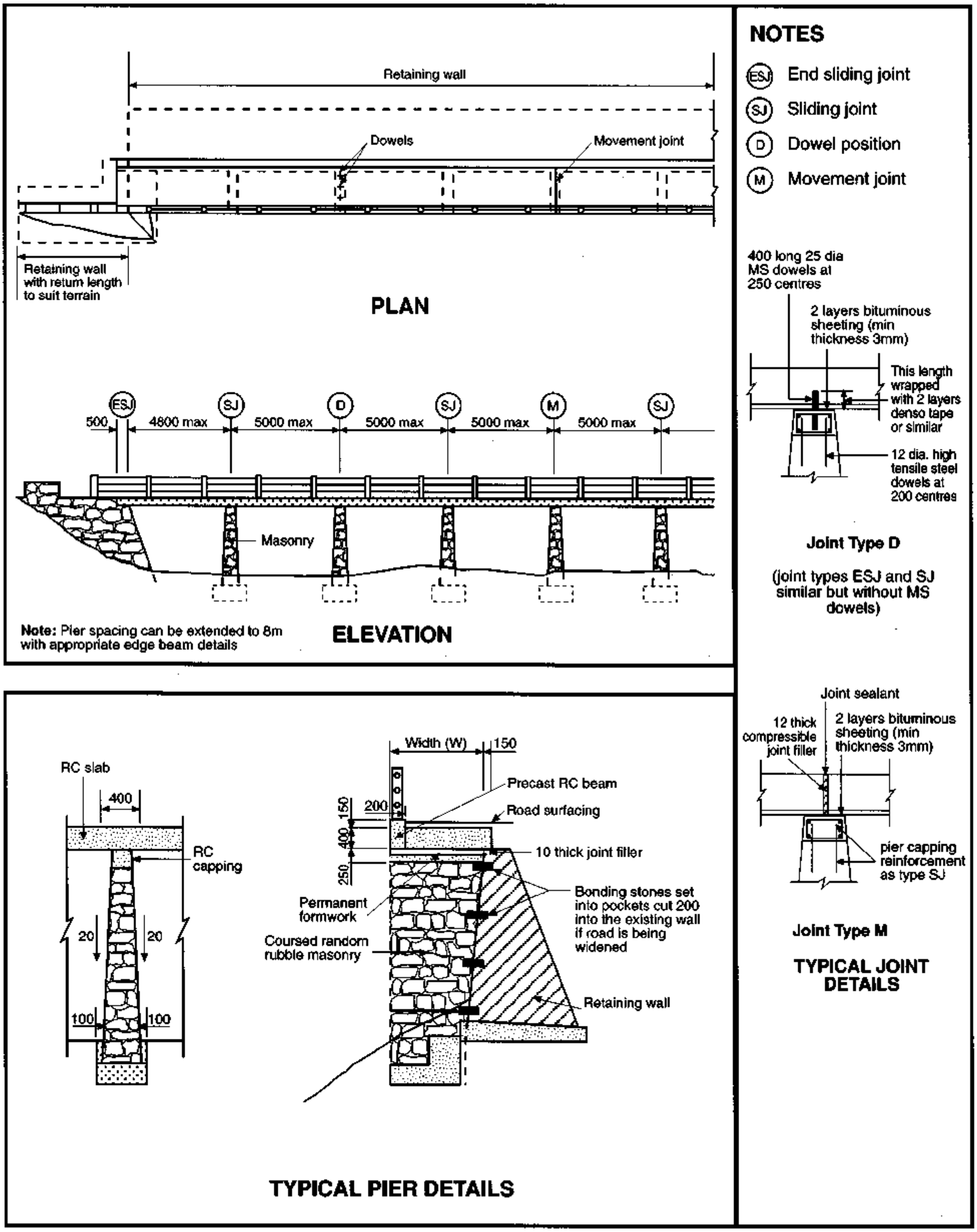


Figure 11.3 Typical details for buttress-supported road slab

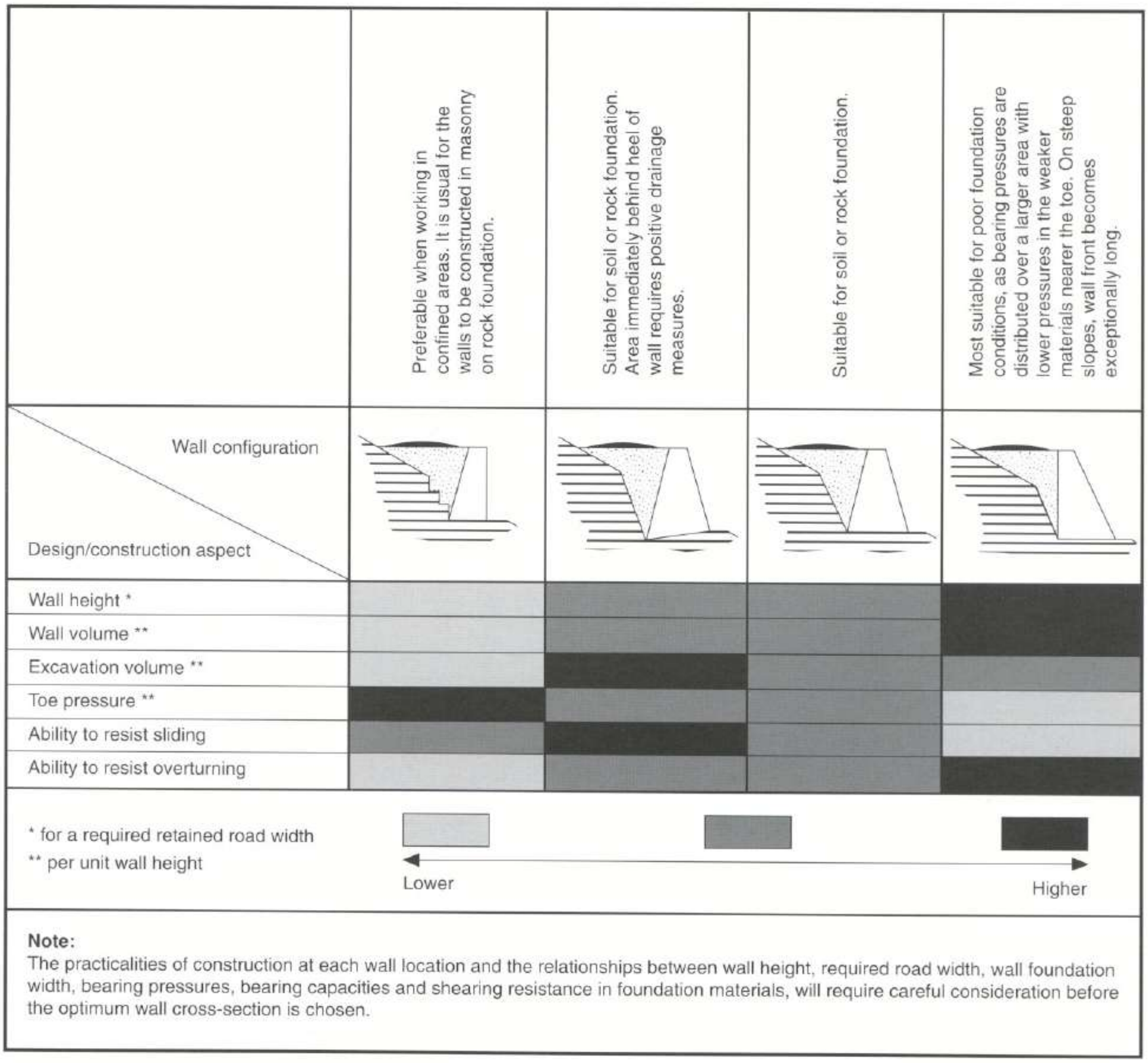


Figure 11.4 Comparison of common retaining wall configurations

12 SLOPE PROTECTION AND SLOPE STABILISATION

INTRODUCTION

12.1 It is important to differentiate between methods of *slope protection* and methods of *slope stabilisation,* as the former are designed specifically to combat slope erosion and shallow slope failure up to a depth of about 0.5m, while the latter are designed to rectify problems of deeper slope movement. Methods of slope protection comprise drainage control and surface treatments for soils and weathered rock that include masonry revetments, the use of vegetation and less conventional slope coverings including geotextiles, bituminous fabrics and gunite/chunam.

12.2 Methods of slope stabilisation normally involve more substantial engineering works, including slope regrading to reduce slope angles, retaining walls and drainage works. Usually, however, it is appropriate to combine elements of protection and stabilisation in order both to give their greatest chance of success and to prevent long term slope deterioration. Toe walls, localised slope grading, shallow drainage and the use of vegetation are the most common measures employed in a combined system.

SLOPE PROTECTION

Embankments

12.3 Embankment erosion is usually initiated as a result of one or more of the following factors:

- the side slope is too steep or too long for the embankment materials to withstand erosion
- embankment materials have not been compacted to specification
- concentrated road runoff is permitted to drain over the shoulder.

12.4 Embankment erosion starts very often at the road shoulder edge, where the level of compaction tends to be relatively low, rather than on the slope surface. Revegetation of embankment slopes is most rapidly achieved by planting with grass slips, by the spreading of collected topsoil containing roots and seeds or by sodding with turves. Turves should be cut from level, fallow fields, or from areas specifically cultivated for the purpose. Grass slip planting is frequently the most effective. A tough grass with a low form and creeping habit should be used, such as *Cynodon dactylon* which is a common pan-tropical species. Erosion of embankment slopes can also be prevented to some extent by careful selection of the material in which the embankment is constructed, if a choice is available. Well-graded soils with some cohesive fraction offer better erosion resistance than single-sized non-cohesive soils. The use of shoulder drains or berms can prevent runoff from discharging over embankment slopes to those areas where erosion has already started, but this can have the effect of concentrating runoff elsewhere.

12.5 One of the most effective ways to control erosion on embankments and on natural slopes below a road is to take reasonable precautions to prevent its initiation. The following guidelines are recommended:

- avoid concentration of runoff wherever possible
- avoid disturbing the natural ground outside the areas to be used for construction
- do not allow construction plant to track natural ground in an uncontrolled manner (a track mark can be sufficient to concentrate flow and start an erosion channel)
- pilot tracks should be constructed only when there is a guarantee that erosion prevention measures will be in place prior to the onset of the next wet season
- every effort should be made to prevent spoil from being dumped outside the limits of designated spoil areas. Spoil is highly erodible; it can smother vegetation and serve to concentrate flow sufficiently to initiate erosion
- do not allow runoff to discharge, either temporarily or permanently, onto unprotected natural ground, other than in pre-existing drainage channels.

Cut slopes

12.6 Table 12.1 illustrates the range of measures regularly used to treat cut slope erosion and failure along mountain roads. Usually, factors of cost, availability of materials and practicality will limit the selection of measures to those that can be applied on a low-technology, labour-intensive basis. It is apparent from Table 12.1 that a number of measures may be required to solve each particular problem. These often involve a combination of slope trimming, slope support, revetments, slope drainage and vegetation applications. Many of these measures will be applicable to the prevention and control of slope erosion as well as to the stabilisation of shallow slope failure.

Bio-engineering

12.7 Where a slope is subject to erosion or very shallow slope failure, bio-engineering methods of slope protection are appropriate. Bio-engineering is the use of living plants, either alone or in conjunction with engineering structures and non-living plant material, to reduce erosion and shallow-seated instability on slopes. In bio-engineering applications there is an element of slope stabilisation as well as slope protection: for convenience in discussing bio-engi -

TREATMENT OPTIONS	MECHANISM OF CUT SLOPE EROSION / FAILURE: Seepage erosion at top of cut slope	Seepage erosion at toe of cut slope	Rilling and incipient gullying on slope	Advanced gullying on slope	Failure of weathered soil at top of cut slope	Soilfall from a steep cut slope	Rockfall from a steep cut slope	Shallow failure of entire cut slope	Deep failure of the entire cut slope in soil	Deep failure of the entire cut slope in rock	Deep failure of the entire hillside
Trim slope to remove irregularities					√	√		√	√		
Remove loose materials	√	√	√	√	√	√	√				
Remove unstable rock masses							√			√	
Remove disturbed materials and replace with dry stone pitching	√	√			√	√					
Trim head scar back to < 45°	√				√	√		√	√		
Apply dry stone pitching or grass planting to head scar	√				√	√					
Apply grass planting and dry stone pitching to seepages	√	√									
Secure gabion netting to the slope							√				
Use bituminised jute netting with grass planting			√		√	√					
Apply mulching and brush mattressing			√								
Apply grass lines and fascines			√								
Dig out seepages and construct surface drain		√	√	√					√		
Protect floors and sides of rills / gullies in masonry			√	√							
Construct small checkdams in rills / gullies			√	√							
Herringbone drains in gabion bolster			√	√							
Trench or herringbone french drains									√		
Drainage management above head scar / cut					√	√			√		
1 - 2m dry stone or composite masonry breast wall				√							
2 - 3m toe wall (composite, masonry, crib or gabion)								√			
Masonry props / dentition to support overhangs							√			√	
Rock bolting / rock anchoring										√	
Gunite / shotcrete (consider alternatives first)							√				
Rock fall trap ditches and walls							√				
Partial removal of slip debris, construct toe wall with backfill									√		
Toe retaining wall with slope grading and trench drainage									√	√	
Ground investigation for design of capital intensive works											√

Table 12.1
Range of methods commonly applied to the treatment of cut slope erosion and failure

neering as a subject, both functions are included here. There is a wide range of techniques associated with bioengineering, which cannot be described here. Gray and Leiser (1982), Gray and Sotir (1996), Coppin and Richards (1990) and Schiechtl and Stem (1996) give good accounts of bio-engineering systems and management.

12.8 Vegetation is a very desirable means of providing slope protection for reasons of availability, relatively low cost, appropriateness of installation techniques and compatibility with a rural environment. It is particularly appropriate in situations where large areas of slope are affected, a common situation on road cuttings and over unstable mountain slopes. The enhancement of road-side vegetation also has a positive effect both visually and in terms of plant diversity.

12.9 Planting schemes should if possible be undertaken in co-operation with local farmers, to keep grazing of newly-planted sites under control and to minimise the effects of soil saturation and runoff from farmland above road cuttings. Concentrated seepage or runoff from irrigated land is a common cause of slope failure, even on slopes that have a well established cover of vegetation.

12.10 The principal advantages of vegetation from a slope protection point of view are:

- the presence of a vegetation cover protects the soil against rainsplash and erosion, and prevents the movement of soil particles down slope under the action of gravity
- vegetation increases the soil infiltration capacity, helping to reduce the volume of runoff
- plant roots bind the soil and can increase resistance to failure, especially in the case of loose, disturbed soils and fills
- plants transpire considerable quantities of water, reducing soil moisture and increasing soil suction

12.11 In view of the above, it is evident that vegetation is important in the control of erosion and shallow forms of instability (1-3m depth at most), but that it plays no significant role in the stabilisation of deeper failures of soils or rock. Vegetation complements engineering structures, protecting them from scour and disruption, and preventing damage to the slope generally.

12.12 The mechanisms of failure on soil slopes for which bio-engineering protection is appropriate are:

- erosion. Washing of soil particles over the slope surface. This process may be regarded as the removal of a skin of material only, to distinguish it from the deeper forms of slope failure that are given below, though where runoff becomes concentrated it can rapidly lead to deep gullying in erodible soils
- soil flowage. The uppermost soil layer can become saturated during periods of heavy rain, causing it to liquefy and flow. This process initiates within the top 50mm or so of the soil, but probably rarely progresses deeper than about 150mm over a whole slope surface before the heavy rain in a storm event ceases, or before a rill is developed and flow becomes channelled. The effect of soil flowage can be very destructive to minor engineering structures and young or even mature plants.
- localised translational shear failure of a soil slope. A depth of about 0.3m is perhaps the maximum practicable limit for effective physical restraint of a slope by plants. The total influence of plants, especially trees, on a slope goes much deeper. It is certain that the hydrological effects, both positive and negative, of plants go to at least one metre and probably to several metres as a matter of course.

12.13 Although vegetation cannot be designed or built to engineering specification in the conventional sense, it can be selected and arranged on the slope to perform a specific engineering function. This function should be identified as part of the process of bio-engineering design. The engineering functions of vegetative treatments are listed in Table 12.2, together with those of civil engineering structures for comparison.

12.14 The general characteristics required of bio-engineenng plants are:

- they should be adapted to the growing conditions of the general environment, as well as those of the individual site
- rapid growth
- long-living
- substantial root system
- easy to propagate and yielding abundant propagation material

appropriate to secondary considerations of socio-economic factors, eg a selected species should not be invasive or poisonous to livestock.

12.15 Ideally, a mixture of plant types should be introduced so as to give a range of rooting depths. This will tend to prevent continuous shear planes from developing in the upper soil layers, discouraging shearing from taking place. It is appropriate to use local species rather than imported material because native plants are more likely to be adapted to grow in the hostile conditions found on bare sites and be resistant to local diseases. Also, if a nursery industry is to be developed to serve the road project, this can be implemented more easily if the species are familiar to the nurserymen.

Table 12.2 Engineering functions of treatments

Engineering function	Civil engineering treatments	Vegetative treatments
REDUCE STRESSES	Remove load; reduce slope angle	Not applicable, but weight of vegetation on slope should be minimised
CATCH material moving over surface	Tightly-pegged wire netting; cable lashing of boulders; catch wall	Stout grass; broad-based shrubs; bamboos
ARMOUR slope against rain splash and erosion	Revetment; surface rendering; stone pitching; jute netting	Grass mat; aerial canopy of plant community
SUPPORT slope from below	Retaining wall; prop wall; fence	Trees; large shrubs; bamboos
ANCHOR slope by pinning through to layer below	Rock anchors	Anchoring effect of individual tree root systems cannot be guaranteed, though this mode of operation is often assumed
REINFORCE soil by increasing its shear strength	Reinforced earth; soil nailing; soil-filled fabric cells	Strong, dense rooting systems of grasses, shrubs and trees
DRAIN slope	a) Surface drains	a) Vertical or diagonal planting to direct water down the slope
	b) Sub-surface drains	b) Root systems carry water down into soil as well as drawing it out by transpira tion; live fascine drains

12.16 Revegetation should be carried out as early as possible in the rainy season, to maximise the time available for the plants to become established. Grasses planted late in the rainy season may not become well enough established to survive the following dry season. Grasses benefit from an increased growing season by being planted shortly before the rains and watered until the start of the wet season. Sites should be protected from grazing until the plants are large enough to tolerate some browsing.

12.17 Despite the apparently rapid growth of plants in the tropics and sub-tropics, large grasses and woody plants take at least three years to become well established. In subtropical countries with monsoonal rainfall regimes the dry season is undoubtedly a major factor causing this long period to maturity. However, grasses in particular can begin to contribute to slope protection within the first season.

Grasses

12.18 Grasses should preferably be a mixture of creeping and clumping types. Creeping grasses form a continuous root system (a root mat), whereas clumping grasses leave gaps between the plants that can be subject to erosion. The advantage of clumping grasses is that they can grow very large, with deep roots.

12.19 Grass can be established by hydro-seeding, turfing or by manual planting. Hydro-seeding is by far the quickest method of grass establishment on highway cut slopes and embankments. It is often sprayed onto a bio-degradable geotextile pegged to the slope surface, which helps to hold the seed mix in place until established. However, note that:

- hydro-seeding requires specialist equipment and application technology
- it is not possible to apply hydro-seeding at sites located far above or below a road, beyond the reach of spraying equipment

- hydro-seeding is not appropriate for forms of slope failure involving soil flowage or deep mass movement.

12.20 Turfing is rarely adopted in mountainous areas because usually there is so little grassland available that can be used as a source of turf without causing erosion or loss of grazing land. In remote, low cost road applications, planting or seeding of vegetation by hand is often the only practicable method.

12.21 It is not really known whether it is better to plant grass randomly or in lines on any particular site. Research on the subject is continuing: Figure 12.1 and Table 12.3 give some suggestions. Horizontal grass lines are efficient at catching moving debris, but the lines tend to retain water on the slope and increase the risk of soil saturation and flowage. Vertical (down-slope) grass lines release the water but suffer from erosion of the roots and death from drought. Thus, vertical lines cannot normally be recommended for slope protection. A compromise exists in diagonal grass lines, which retard the rate of water flow down the slope while encouraging enough infiltration for growth.

12.22 Another area of uncertainty in grass planting is whether to seed the slope or plant it with grass slips. (Slips are small complete grass plants divided off from a parent clump). The choice depends to some extent on the tendency of the species to spread by seed or division. Seeded slopes give a random planting configuration, and seeding has the advantage of speed of sowing. Grass slips have the advantage of establishing quickly and of forming definite lines from the start, which enables them to perform a function as physical barriers on the slope as soon as they are planted. Grass established from seed has little or no barrier effect until the second growing season.

12.23 Where grass is to be established by hand broadcasting of seeds the slope should be covered with a layer of mulch (cut plant material). Although this increases costs, mulch creates a protective environment in which the seeds can become established, and offers some protection to erosion-susceptible soils during the early stages of the wet season when revegetation is still in progress. If mulch is not provided, grass seeds can simply be washed from the slope. Mulching also protects very young, newly germinated grass plants against sun scorch. This can be a problem on exposed slopes in higher altitudes and tropical locations; it can also be damaging to early growth from some slurry-seed mixes.

12.24 Bamboo species can be very advantageous in places where ground conditions are suitable, and they are welcomed by farmers as having many uses. They grow best on damp, shady sites on fine soils. Bamboos are relatively light in weight and develop extensive and dense root systems. They can also tolerate erosion and adapt to changing stability conditions. Bamboos require careful propagation and raising in a nursery, and take about five years to reach maturity.

Trees and shrubs

12.25 Trees and shrubs are placed as individual plants positioned at random on the slope. Trees should be of low height, or capable of good recovery after being lopped. A maximum height of about 5m is appropriate for trees on slopes, sufficient for them to achieve good stature without becoming too susceptible to leverage by wind. Trees and shrubs originate as woody cuttings taken from mature plants, as seedlings grown in tubular polythene bags (`poly pots'), or as seedlings lifted and brought with bare roots from a nursery. Cuttings are planted on site, and require no preparation other than to be planted without delay after cutting.

12.26 Cuttings can be rather slow to take. Although they may put out a good show of leafy branches, root development tends to lag well behind. Seedlings, although apparently less sturdy when first planted, produce a more even balance of roots and shoots. Also, although small they are fairly tolerant of disturbance.

Species selection

12.27 The choice of vegetation species is dependent upon both the function which the plants have to perform and the local site characteristics. It is most important to choose a species that will thrive on the site. For this reason, and because of the wide variety of micro-sites usually found in mountainous areas, environmental factors may have to take precedence over engineering attributes of plants. However, in most tropical, sub-tropical and temperate areas there is normally a sufficiently wide range of plants available so that all criteria can be met without serious compromise. The selection criteria are:

Engineering considerations

- engineering function of the system (Table 12.2)
- rooting type for the function required (ie deep or spreading)
- aerial structure. The main purpose should be to assist the engineering function, but the aerial part of the plant is also important for the ecological and social compatibility requirements listed below.

Environmental considerations

- climatic conditions (temperature and moisture at the micro-site level)
- physical site conditions (stoniness, permeability and fertility of the slope material; see also Table 12.3)

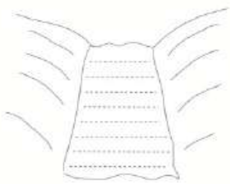
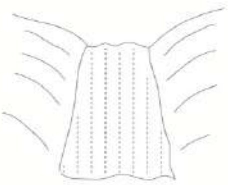
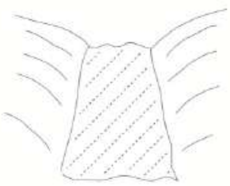
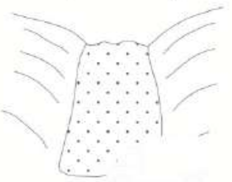
Configuration	Description/ critical slope	Normal spacings	Main advantages	Main limitations
Horizontal (contour) 	Planting of grass slips (or sprigs) in geometric lines across the slope or along the contour Slopes ≤ 65°	Plants at 100mm centres within rows. Row spacings: Slope <30°: 1000mm Slope 30-45°: 500mm Slope >45°: 250mm	Traps material moving down slope. Retards runoff on impermeable materials	On permeable materials, can increase infiltration rate to the point of flowage
Vertical (downslope) 	Planting of grass slips (or sprigs) in geometric lines down the slope or towards drainage lines Slopes ≤ 65°	Plants at 100mm centres within rows. Rows at 500mm spacing	Maximises surface drainage while protecting against erosion. Minimises infiltration	On impermeable materials, erosion occurs between plant lines. Plants can suffer from drought
Diagonal 	Planting of grass slips (or sprigs) in geometric lines diagonally across the slope, usually at 45° to the contour Slopes ≤ 65°	Plants at 100mm centres within rows. Rows at 500mm spacing	Appears to combine the best features of both horizontal and vertical planting in most cases	Should not be used where the specific advantages of horizontal or vertical planting patterns are critical (see Table 12.3)
Manual seeding 	Grass seeds are spread evenly over the surface and usually covered with mulch Slope ≤ 50°	Most species require a seeding rate of $25 g/m^2$. Mulch, if applied, should be at a rate of $0.05 m^3/m^2$	Can be used to create an even cover over all surfaces	None of the structural advantages of grass slip planting. Plants take longer to develop from seeds than slips
Hydro-seeding 	Grass seeds are sprayed over the surface in a slurry mixture containing mulch Slopes ≤ 65°	Application varies, depending upon system and site	Very rapid cover of large areas. Resistant to rain splash and minor surface flow	None of the structural advantages of grass slip planting. Requires costly specialist equipment
Turfing 	Turf cut from elsewhere is placed on surface and pegged if necessary Slopes ≤ 35°	Complete cover. Pegs should be placed at about 250mm centres on slopes >15°	Instant and complete surface cover. Immediate and good resistance to erosion. Requires an area of cut turf equal to that of the surface to be treated - possible bare areas remaining	Relatively costly. Turf may never extend roots into underlying material, leaving a permanent potential sliding surface

Figure 12.1 Grass planting systems

Table 12.3 Selection of vegetative slope protection techniques

Slope angle	Slope length	Material drainage'	Site moisture 2	Optimal technique
START	→	→	→	
>50°	>15 metres	Good	Damp	Diagonal crass lines
			Dry	Contour grass lines
		Poor	Damp	1 Downslope grass lines and strengthened rills or 2 Chevron grass lines and strengthened rills
			Dry	Diagonal grass lines
	<15 metres	Good	Any	Jute netting and planted grass
		Poor	Damp	1 Downslope grass lines or 2 Diagonal grass lines
			Dry	1 Jute netting and planted grass or 2 Contour grass lines or 3 Diagonal grass lines
35-50°	>15 metres	Good	Any	1 Horizontal bolster cylinders and tree planting or 2 Downslope grass lines and strengthened rills or 3 Grass seeding, mulch and wide mesh jute netting
		Poor	Any	Herringbone bolster cylinders and tree planting
	<15 metres	Good	Any	1 Brush mattresses of woody cuttings or 2 Contour grass lines or 3 Grass seeding, mulch and wide mesh jute netting
		Poor	Any	1 Diagonal grass lines or 2 Herringbone fascines and tree planting or 3 Herringbone bolster cylinders and tree planting
<35°	Any	Good	Any	1 Contour strips of grass and trees or 2 Tree planting
		Poor	Any	1 Diagonal lines of grass and trees or 2 Tree planting
Any	Any	Any rocky material[3]		Direct seeding of shrubs or small trees

Notes
1. Material drainage is related to the permeability of soils and the likelihood of their reaching saturation and losing cohesion under intense rainfall, thereby starting to flow.
2. Assessment of site moisture should include a consideration of overall slope moisture. This is an environmental dryness factor related to the sum of local topographical and climatic variables. Rain shadow effects can be significant in this, although aspect is usually the most prominent factor, especially outside the tropics.
3. "Any rocky material" is defined as material into which rooted plants cannot be planted but seeds can be inserted in holes made with a steel bar

This table was supplied by the Geo-Environmental Unit, Department of Roads, Kathmandu

- adequate availability of seed or plant stock without the risk of depleting natural reserves
- appropriate propagation and planting method
- ecological compatibility between plants on the site
- compatibility of the chosen species with the community of local people (eg positively useful plants or undesirable plants such as weeds).

12.28 The selection of a vegetative technique for the protection of a particular slope depends on many factors relating to the site. It is necessary to consider these in as much detail as time and skill allow in order to arrive at the most appropriate technique. It is not practicable to quantify the many variables involved, so assessments rely principally upon experience. Table 12.3 gives a simplified guide to the selection of techniques on earthworks slopes in seasonally wet mountainous areas, and Figure 12.2 shows examples of bio-engineering systems.

12.29 It is normally possible to draw up a list of local plants that fit most of the above requirements for bio-engineering. The department of botany in a university may be able to give advice on the characteristics and distribution of species within the region. Departments of agriculture and forestry have been found less able to help because their expertise lies with economic species, which plants useful for bio-engineering purposes frequently are not. However, these institutions are excellent sources of advice on setting up and running nurseries, and on plant propagation and care.

12.30 The relation between the length of life of engineering structures and the time that a plant takes to grow to maturity is relevant to the question of combining plants and civil engineering structures effectively. If a treatment such as jute netting is to protect seedlings until they are established, the treatment must last long enough before decaying to be able to perform this function. If not, the slope will be at risk of being damaged before the plants are capable of protecting it.

12.31 Engineering structures made of dead organic products such as jute, wood or bamboo can be regarded as being `temporary' - defined here as having a serviceable life not exceeding five years. Materials such as jute netting and bamboo last only one or two seasons in a tropical environment, but a coat of bitumen will increase their life to three years or even more. 'Permanent' materials such as stone and cement-bound products have the potential to last much longer than five years, so the distinction between temporary and permanent structures is usually obvious.

Slope fences

12.32 Slope fences are traditionally made of woven wattles (flexible stems) or split bamboo and placed horizontally across a slope. Fences made of organic material have a life of only two or three years at most in a wet sub-tropical environment before rotting. Wire fences, made with steel posts and wire rails and mesh panels, last for 8-10 years.

12.33 Fences are intended to support placed soil or moving debris, providing a stable environment in which plants can grow. However, during heavy rain, water can build up in the soil behind these structures, causing the soil to flow out beneath. Otherwise, if the fence is weakened by decay, the pressure of wet soil may push it over. Because of this, and the relatively small amount of soil they retain, fences can be regarded as having limited application for slope stabilisation in hot, wet climatic environments.

Jute net

12.34 Woven jute netting can provide useful temporary protection and a stable environment for grass establishment in landslide back scars and on eroding cut slopes. It is sometimes possible to have the net made locally from jute grown in the region. Jute netting can reduce the velocity of surface flow, and is capable of retaining small volumes of soil debris and a degree of soil moisture necessary for the growth of grass slips in drier areas. It has been applied to cut slopes as steep as 80°. Jute netting is unsuitable where soils are wet or are undergoing active shallow failure.

12.35 Netting should be secured to a smooth slope with staples, live pegs or split bamboo. Live pegs offer advantages of low cost and the capability to grow. The life of the fabric can be extended from one to at least three years if it is soaked in a bitumen solution prior to fixing.

Palisades, brush mattresses and fascines

12.36 Palisades, brush mattresses and fascines are made from live woody cuttings. Palisades are fences consisting of closely-spaced upright cuttings, the line of cuttings being placed horizontally across a slope. The continuity of the fence tends to break down over time through death of the cuttings by overcrowding and their subsequent decay. Brush mattresses and fascines are more durable, though less able to support debris immediately from the time of installation onwards.

12.37 Brush mattresses are formed from hardwood cuttings. The cuttings are laid with their lower end in a shallow trench and the aerial part sticking out above ground. For extra security they are often wired to wooden stakes driven into the ground at 1 m centres. An advantage of brush mattresses is that they allow excess debris to roll over them with minimal damage. This helps them to survive long enough to take root and grow into strong shrubs.

12.38 Fascines are bundles of live cuttings laid buried in soil. The cuttings can be placed in shallow cross-slope trenches or in earth supported by wattle fences. Fascines are capable of rapid shoot and root development and eventually

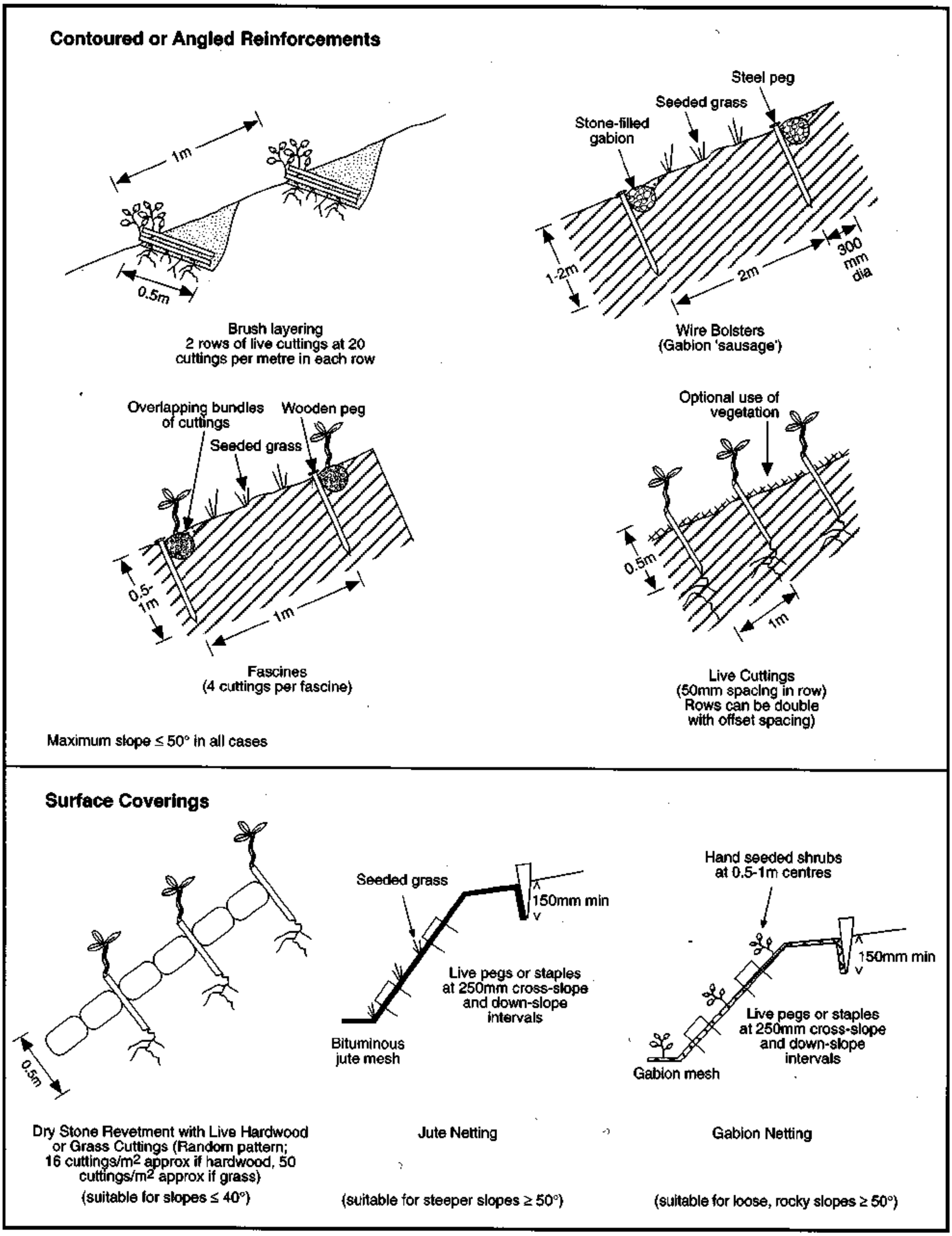


Figure 12.2 Examples of bio-engineering slope protection systems

form a dense shrubby barrier. Their purpose is to catch moving debris and provide support for the slope through the interlocked root systems.

Impermeable membranes and surface coatings

12.39 Gunite has been used widely and successfully in Hong Kong and Malaysia for the prevention of erosion and rockfall from weathered or fractured rock slopes under conditions where immediate protection is required. It comprises cement-stabilised aggregate (shotcrete) sprayed onto a wire mesh slope covering. Although it is standard practice to incorporate weep holes in the sprayed surface, the technique is generally inappropriate under conditions of groundwater seepage. Its application to low cost mountain roads is limited by its expense and the predominance of colluvial soils and complex drainage conditions in many cut slope excavations.

12.40 Chunam is a lime-based screed which is usually applied to a soil slope by trowel. Again, it has been used successfully in Hong Kong to prevent erosion of slopes formed in residual soils. As with gunite, the use of chunam under conditions of groundwater seepage or high soil moisture content can have an adverse effect on stability, despite the incorporation of weepholes into the covering. In addition, where a chunam cover becomes cracked it can lead to concentrated runoff and rapid erosion of the slope beneath the cover. Although chunam probably has a greater potential use on low cost mountain roads than gunite because it can be applied by hand, it requires a maintenance commitment which usually cannot be relied upon. Also, on environmental grounds, it is a less preferable option than vegetation.

Revetments

12.41 Revetments are designed as slope protection rather than slope support structures (Figure 12.3). They are most frequently constructed in dry stone, mortared masonry, composite dry/mortared masonry and gabion. Their construction at the toe of a cut slope can be of particular benefit to the reduction in seepage erosion and softening of materials that would otherwise lead to progressive erosion and failure of the entire cut slope.

12.42 Where see pages are encountered, it is preferable to employ freely-draining revetment structures, such as dry stone pitching and composite dry/mortared masonry grids. Mortared masonry revetments are frequently employed where slopes formed in weathered rock require protection against erosion induced by surface runoff rather than failure of the surface materials. Masonry buttresses and dentition work are used on steeper sections of cut slope to support overhanging portions of rock or boulders, and to fill in any cavities in the slope surface formed by localised failure and seepage erosion. Gabion toe or breast walls are usually 1 or 2m in height and are designed to protect the base of a cut slope from fretting, or to provide a small retaining capacity to loose and usually failed materials.

Surface drainage

12.43 Surface drains (Figure 12.4) are frequently constructed on hillsides, for purposes of slope stabilisation, or across the top of a cutting to prevent surface runoff from eroding freshly exposed materials. In the latter case, they are usually constructed between 3 and 5m behind the top of the excavation, and on slopes up to 35°: above this angle construction becomes impracticable. They are normally built as a trench, lined with either mortared masonry or dry stone. Polythene-lined drains are cheaper and more flexible than masonry but they are often vandalised and punctured, especially when adjacent to footpaths and housing areas.

12.44 At sites where some seepage from the drain is allowable, earth bunds can be considered. These can be unlined and grassed, or stone pitched, and can be further reinforced with vegetation on the downslope side. Generally, bunds require more careful construction than trenched surface drains because they need to be accurately positioned in order to maintain a continuous longitudinal fall.

12.45 Obviously, surface drains are only effective when surface runoff rates are significant. Surface runoff can be expected only during high intensity rainfall on moderate to steeply-inclined slopes, on slopes of low permeability where vegetation is patchy, or where runoff from agricultural land becomes concentrated onto unvegetated soil slopes. If surface runoff is substantial, and there is a clear threat of erosion or slope failure further downslope, the use of surface drains is justifiable. However, they do represent a hazard in the sense that they are easily damaged or blocked by debris or leaf litter, and are often not seen and therefore not cleaned on a regular basis. In addition, differential settlement or ground movement will dislocate masonry drains, leading to concentrated seepage, if they are constructed without polythene lining. If there is any doubt about their effectiveness, or whether they can be maintained in the long term, it is better not to build them than have them become forgotten and allowed to fall into disrepair. making drainage and instability problems worse. Some further considerations of surface drain construction are listed in Box 12.1.

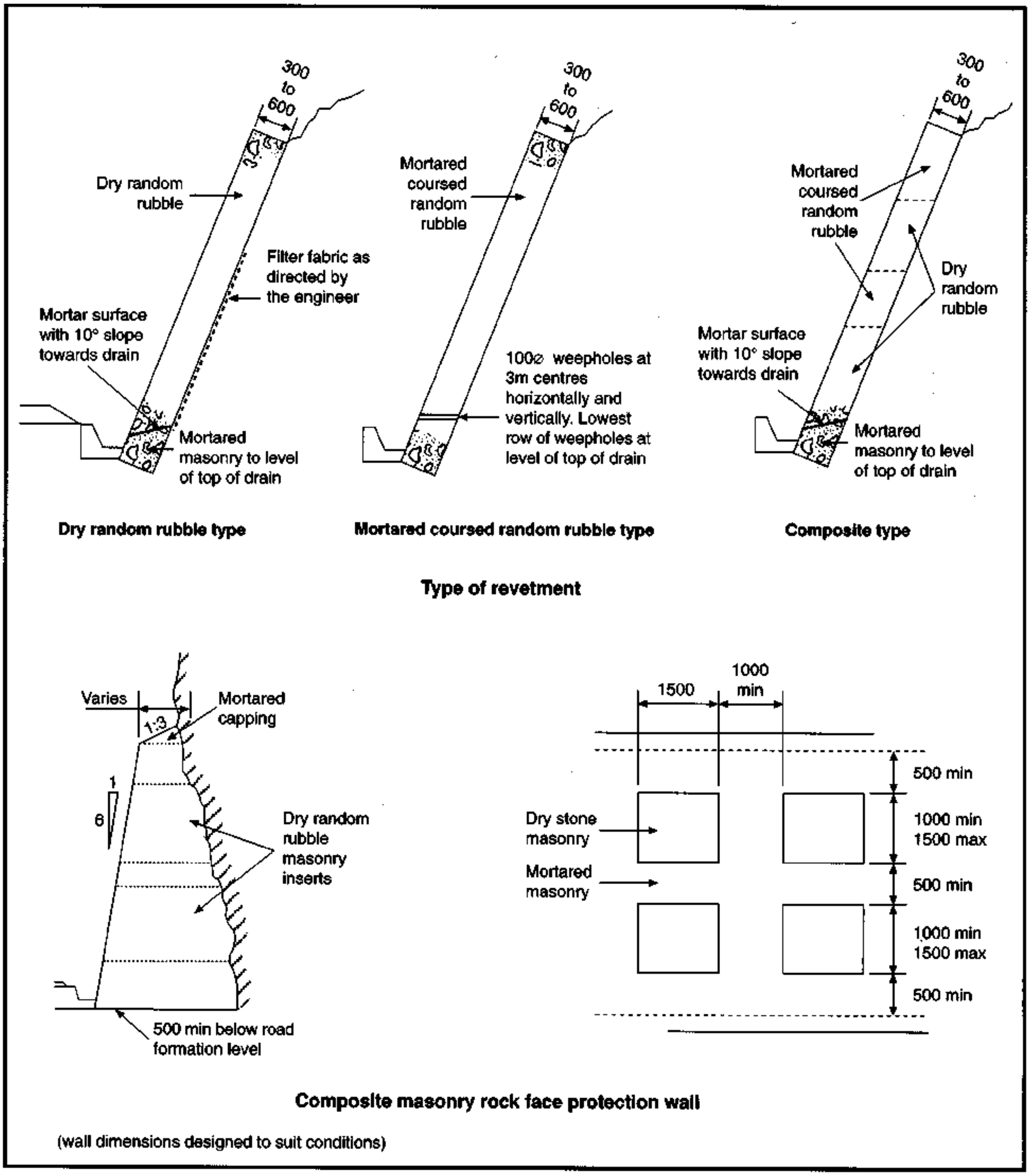
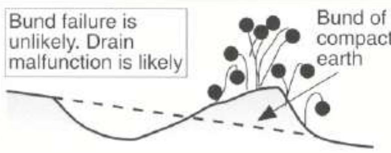
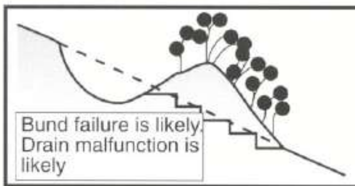
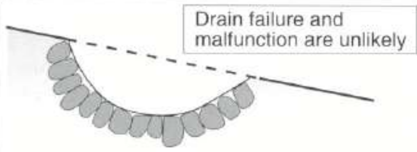
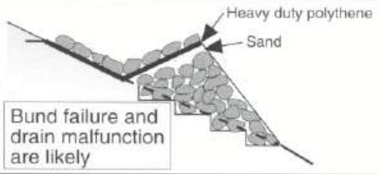
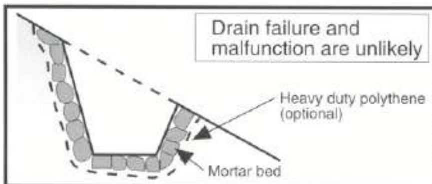
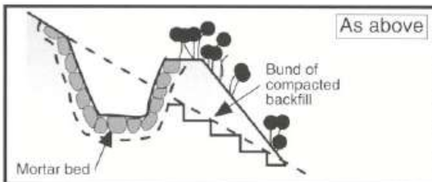
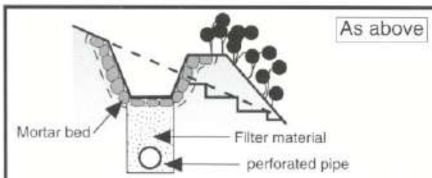


Figure 12.3 Revetment structures

	DRAIN TYPE	APPLICATION	PROBLEMS
	Cut and fill earth bund Low capacity	Limited application to slopes where minor overland flow is occurring on sandy soils	Difficult to construct with constant gradient. Difficult to maintain. Will lead to concentrated seepage
	Cut and fill earth bund with fill benched into steeper slope for increased stability. Low capacity.	Limited application as above Interceptor drain on low-permeability soil slopes	As above. Bund collapse will lead to concentrated runoff and erosion
	Full cut pitched dry stone drain. Moderate capacity.	Carrier rather than collector drain, used as part of small scale slope drainage works	Blockage or seepage will cause concentrated erosion spoil disposal
	Full fill dry stone bund. Low capacity.	Interceptor drain on low-permeability slopes	Collapse of rubble retaining wall will lead to concentrated seepage. Polythene may be stolen or damaged
	Full cut masonry lined channel. Moderate capacity.	Cut-off drain above cut slope or as part of wider slope drainage scheme	Blockage, seepage or failure may lead to concentrated erosion and slope failure spoil disposal
	Cut and fill masonry lined channel with compacted earth bund. High capacity	As part of an intergrated slope drainage scheme where drainage control is paramount to slope stability	Comparatively expensive to construct. Blockage, seepage or failure may lead to concentrated erosion and slope failure
	Cut and fill masonry lined channel with compacted earth bund and 1 - 1.5m deep filter drain. High capacity.	As above and where the stability of surface soils (0-2m) can be improved by sub-surface drainage and water-table draw-down	As above. Clogging of pipe is a frequent outcome unless it is wrapped in filter fabric

NB. All of these slope drains will lead to concentrated erosion and instability if they are not properly maintained. Their ability to function in the intended manner will require careful consideration.

Figure 12.4 Common types of contour and cut off drains and their application

Box 12.1 Factors to be considered in the design of surface drains

- water collected by the drain must be discharged safely, in a manner that will not initiate erosion elsewhere
- construction of masonry-lined drains should be limited to undisturbed slope materials. Differential settlement, which frequently occurs in made ground and particularly at the interface between natural ground and fill, will lead to rupture
- drain gradients should not exceed 10°
- stepped drain outlets should be provided with a cascade down to the collection point
- drains should discharge into a stream channel wherever possible, and preferably into channels that already convey a sizeable flow in comparison to the drain discharge
- low points in the drain system should be designed against overtopping by widening or raising the side walls
- lengths of drain should be kept short by the construction of frequent outlets in order to reduce erosion potential should drain failure occur
- where it is not practicable to discharge cut-off drainage into an adjacent stream channel, cascades can be constructed down the cut slope to convey water into the side drain. However, these structures are often vulnerable to the effects of side splash, undermining by seepage erosion and concentrated runoff along their margins. They must be designed to contain the water, and their margins must be protected with vegetation or stone pitching.

SLOPE STABILISATION

Introduction

12.46 Large (greater than 50,000m3), deep-seated and active landslides on natural slopes are relatively rare due to the frequency of small failures and erosion processes that tend to remove weathered and unstable materials as soon as they develop. However, where large landslides are encountered, they can lead to repeated road loss or deformation. These consequences must be weighed against the likely costs, practicalities and long-term success of capital-intensive stabilisation works. In most cases of this nature, a flexible gravel pavement across the zone of movement with appropriate seasonal maintenance, is by far the best solution until the landslide becomes dormant. Generally, the costs and technical back-up required for investigating, analysing and remedying these landslides are not compatible with the low cost design philosophy of mountain roads, especially in remote areas where access for drilling and excavation machinery is often difficult (Chapter 7).

12.47 The following precautions and actions should be taken, in approximate order of preference, to avoid or minimise the engineering damage and traffic disruption caused by large landslide movements:

- avoid the landslide by prudent choice of alignment wherever possible
- minimise length of alignment affected and cross at most stable location
- design earthworks cross-section, ie proportions of cut and fill, so as to improve stability or minimise slope disturbance (Chapter 9)
- carry out measures to improve temporary stability and accept periodic loss of access and a long-term maintenance commitment
- stabilise locally, using a combination of comparatively low cost drainage, slope support and erosion control measures that, overall, lead to a satisfactory increase in stability in the vicinity of the alignment. (However, it may be necessary to accept that the road suffers further deformation or is sometimes closed)
- stabilise entire slope using capital and machinery-intensive methods.

12.48 As with the prevention or control of erosion, slope stabilisation usually employs a number of methods to remove or reduce the causes of failure that are external to the slope, such as toe erosion protection (Chapter 13), together with measures to improve the stability of the slope itself, including drainage, slope support and reprofiling. Where capital-intensive measures are to be employed, it will usually be necessary to carry out some form of ground investigation (Chapter 7) in order to be able to define an acceptable factor of safety for the design of stabilisation works that are most likely to succeed.

Sub-surface drains

12.49 Drainage is often the most important factor in slope stabilisation, and is usually undertaken by the diversion of water away from the slipped mass by surface drains (discussed above) and by drainage of the slipped mass itself. Drainage design should be based on an appreciation of the natural drainage patterns of the slope, especially the locations of springs and see pages. Filter or interceptor drains can be used independently or in association with surface drains. However, as with surface drains, drain dislocation for whatever reason can make matters significantly worse.

12.50 Herringbone drains tend to be the most common forms of shallow (usually lm deep) sub-surface drainage adopted to tackle instability problems in earthworks and natural slopes along low cost mountain roads. The use of herringbone systems, with diagonal feeder drains wrapped in wire gabion panels and an impermeable membrane on their invert and downslope side, will reduce the tendency for dislocation to develop. Nevertheless, significant movement of the drain will affect its hydraulic conductivity to the point that it may become locally surcharged. Furthermore, the excavation of sub-surface drains should not itself be allowed to disturb the stability of the slope.

12.51 To facilitate rapid extraction of water, a perforated plastic pipe is frequently placed on the floor of the drain and wrapped in filter fabric to prevent clogging by fines. Diagonal feeder drains should be lined on the uphill side with a geotextile filter fabric to prevent clogging. The choice of filter fabric depends upon expected rates of subsurface flow and the particle size distribution of the surrounding soil. Too fine a fabric may prevent free drainage of the slope immediately above the drain; too coarse a fabric will allow fines to enter the gravel core.

12.52 Deeper trench drains (up to 3m in depth) are occasionally used to obtain a greater drawdown of water. Generally, 3m is the maximum depth advisable for drain excavation by manual means. In reality, groundwater inflow, the frequent presence of large boulders and the problems of safety in both shored and unshored excavations mean that most trench drains are excavated to depths much shallower than this. The spacing of trench drains will depend on permeability and the drawdown required, but generally varies between 3m and 10m.

12.53 In large debris masses where periodic mass movement occurs during saturated conditions, deeper trench drains can be used very effectively. Trench drains can be used to stabilise the mass by removing excess water at critical periods, and thereby avoiding the loss of cohesion which comes with saturation. The simplest situation is where erosion of the debris mass has no adverse consequences, and open drains can be used to initiate a semi-natural system of gullies. In this case, unlimited drains can be used, with the option of vegetative protection of drain sides in the second wet season. In some cases, short sections of drain bed lining or drop structures may also be required. Where erosion of the debris is a potential hazard. then the drains must be lined or backfilled with freely-draining material.

12.54 Trench backfilling materials include gravel, rubble or no-fines concrete, with the last providing an additional buttress against slope movement. Perforated pipes can be used to facilitate drainage in the same manner as for herringbone systems. Ideally, trench drains should be excavated at right angles to the slope contours to minimise slope disturbance and facilitate rapid drainage. Backfilling should commence as soon as deep drains are excavated in order to minimise the possibility of trench collapse and induced slope failure.

12.55 Horizontal drains are designed to intercept groundwater and concentrated seepage at depth Their use on low cost mountain roads is comparatively rare. Examination of drill core and logging of water strikes during drilling operations can greatly assist in the identification of see pages, although it is common to find that only a relatively small percentage of the installed drains are successful in lowering pressures within water-bearing strata. Because of the usual uncertainty over their likely performance, their effect in reducing water levels should not assume critical significance in the design factor of safety.

12.56 Installation of horizontal drains into failing soil or rock masses is cautioned as drain dislocation and leakage could lead to accelerated movement. Drains usually comprise 50-100mm diameter slotted PVC pipes inserted into drill holes on a rising gradient of 5-10%. Pipes should be wrapped in filter fabric to prevent clogging although, even with fabric wrapping, pipes may require cleaning by water flush every 5-10 years. Where horizontal drains are successful, drain discharge rates of 1,000 litres per day can occur.

Support for soil slope failures

12.57 Along with drainage, toe retaining structures, constructed in either masonry or gabion, are employed most frequently in soil slope stabilisation. It is necessary to design the wall against overturning, sliding and bearing capacity failure (Chapter 11), as well as to design it as an integral part of the overall slope stabilisation scheme. The success of any retaining wall depends upon the adequacy of its design, construction, foundation stability, key into adjacent slope materials, and scour protection, if positioned in vulnerable river-side locations.

12.58 Other techniques of slope support are occasionally used for which specialist services are required. These are briefly outlined below.

12.59 Anchored reinforced concrete retaining walls. These require a bedrock foundation and may be contemplated when the required restraining forces are such that a gravity retaining wall of sufficient dimension would be impracticable due to foundation or space limitations, and where a stable rock mass behind the failing slope can be used for anchoring purposes. The principal advantage with this design of wall is that it can be constructed in confined spaces, while its main disadvantage is high cost.

12.60 Cast-in-situ contiguous bored-pile walls. These offer a lateral resistance to slope movement while allowing drainage to pass through the space between each pile. They are almost always designed with a key below the slip surface in rock and an anchored reinforced concrete bulkhead across the front face of the wall, anchoring it back into the hillside. Piles constructed at wider centres can induce an arching effect in moving soil masses that tends to increase stability, but the uncertainty over their design and performance, and the need to employ piling equipment in their construction, usually makes them impracticable.

12.61 Soil nailing. Soil nails are designed to transfer the load of an unstable soil mass in shear to the stable material below the failure surface. Soil nails may be up to 5m in length and they are installed either by high pressure injection or by drilling and grouting. Their main advantage is that they can be used to stabilise soil masses under space limitations that restrict the use of more conventional methods. Also, unlike anchors, once installed they require no maintenance. If the necessary equipment can be made available, the technique may have some limited application along low cost mountain roads, although factors of cost, access difficulties for machinery and the general success of more conventional methods of stabilisation will usually prevail.

12.62 Reinforced earth. Reinforced earth retaining walls are discussed in Chapter 11.

Support for rock slope failures

12.63 Retaining structures. Large rock slides are generally not amenable to stabilisation within a low cost project philosophy. Masonry retaining structures can be constructed to some effect if space permits, but the removal of failed rock in order to construct a toe wall may have little overall influence on stability, and might disturb the slope further during excavation. Anchors and anchored bulkheads can provide support against rock creep, dilation and gradual planar failure over relatively small areas (up to $400m^2$), although progressive failure will eventually affect all anchorage systems if they are not maintained.

12.64 Slope re-profiling. The trimming back of steep slopes to more gentle slope angles may have a locally beneficial effect on stability, but it is generally not feasible to do this over large areas. Slope unloading at the head of a landslide, to reduce the driving forces of failure, presents practical difficulties and is very costly in the case of large landslides. Toe weighting, using rock fill berms and road embankments, can significantly increase stability.

12.65 Rock fall containment. Rock fall debris can be contained by means of cut slope benches and slope netting, together with a rock trap ditch, wall or fence at the base of the slope. Rock fall containment measures are only applicable in critical areas. High impact wire nets and flexible post systems, developed in alpine Europe and North America, are relatively costly and require specialist installation and maintenance. Rock trap ditches and walls at the base of a slope take up space but are easy to build and maintain.

13 ROAD CONSTRUCTION ALONG VALLEY FLOORS

INTRODUCTION

13.1 As discussed in Chapter 4, where there is a choice it is usually preferable, on hydrological and stability grounds, to adopt a hillside rather than a valley floor alignment. However, the choice of corridor will depend upon the length and practicality of hillside and valley floor options, and the degree of hazard posed by slope failure and flooding along each. Furthermore, some valley floors are significantly more hazardous than others, and it will be necessary to carefully evaluate the risk implications of these hazards before an alignment is chosen.

13.2 Valley floor and lower valley side alignments can encounter some or all of the following landforms and hazards:

- broad rivers that may rise and fall rapidly by several metres on a regular basis, and by as much as 10m or more in response to high magnitude rainstorms and GLOFs (Chapter 8)
- rivers which are actively meandering and changing their plan-form, which could subsequently encroach on the alignment
- active river flood plains that are likely to flow full at least once a year. The erosive power against the banks of a river in flood is very great
- vigorous tributary streams that are usually highly erosive and capable of transporting large volumes of sediment
- fans from tributary valleys that are either i) high level, with or without active incision, ii) in equilibrium, or iii) rapidly aggrading (building up by accumulating debris)
- flood plain terraces that may be susceptible to river scour on numerous occasions during the wet season, and inundation once every 2-3 years
- higher level terraces that may be subject to scour on a regular basis where they protrude onto the active flood plain
- rock spurs or promontories that project into the flood plain, forming obstacles to river flow and road alignment
- steep, and often eroded, rock slopes on the outside of valley meander bends
- slope instability on the lower valley sides in general.

13.3 These conditions are most common on youthful valley floors, and especially those with gradients steeper than 1 in 20. The rivers that occupy these valley floors dram steep and frequently unstable catchments. Their flood plains will be either so confined and erosive that the development of terrace sequences has not been possible, or will be subject to cyclic erosion and side slope instability over engineering time-scales to an extent that any preserved terrace surfaces cannot be regarded as safe for road alignment. In such situations, valley floor road alignments should be avoided altogether, otherwise frequent loss of significant sections of road will be inevitable.

13.4 Where a valley floor is comparatively mature, and ancient high level terraces are well preserved, then a road alignment located at the back of these terraces, combined with intervening rock cut, may prove satisfactory. If valley side rock mass conditions are not especially adverse to stability, it is usually preferable to construct a road in full cut, or a combination of cut and retained fill through these rocky areas, with a freeboard above the highest anticipated flood level. Where valley side stability conditions are unfavourable, or where river flooding could cause erosion and slope failure to extend far enough upslope to undermine road foundations, it is advisable to examine the practicalities and costs of an alternative alignment altogether.

13.5 The various design considerations associated with road construction in valley floor locations are discussed below.

FREEBOARD

13.6 It is usual to provide the road surface and associated structures with a freeboard of 2m above design flood level to accommodate surface waves and to provide some leeway in the estimation of flood level. The freeboard can be reduced to 1-1.5m in cases where the hydraulic analysis is more reliable. However, the calculation of the design flood (Chapter 8) is a particularly difficult task when rainfall and flow gauging data are limited or non-existent, and where a catchment runoff regime is subject to short term fluctuations brought about by road construction, land use change, extreme rainstorms and cycles of slope instability, channel incision and aggradation. Although widely appreciated, it is important to remember that flood levels can be substantially higher on the outside of meander bends than anywhere else along a given reach.

FLOOD PLAIN SCOUR AND EMBANKMENT PROTECTION

13.7 Flood plain scour, flood plain deposition and valley side instability usually occur at predictable locations. However, external influences, such as tributary fan incursions onto the flood plain, temporary landslide dams and engineering structures, can cause significant short-term modifications to flood plain processes and flow patterns. These should be identified and monitored during the course of construction and maintenance, with appropriate steps taken to protect or locally realign affected sections of road.

13.8 Notes on the calculation of channel scour are given in Chapter 8. It can be assumed that maximum velocities around the concave (outside) banks of river bends and in valley constrictions are between 1.5-2 times greater than average or calculated velocities. On highly active flood plains with mobile bed material, predicted and actual scour depths can frequently exceed 5m, and occasionally 10m. Foundation excavations for road retaining walls and other structures are often impracticable at these depths, given the bouldery nature of the bed material and the requirements for dewatering the excavation.

13.9 Mortared masonry walls are more durable than gabion walls in abrasive riverside locations. and they have the potential to arch over small areas of scour, where gabion walls are more likely to deform. Even when heavy duty selvedge wire is used, gabion boxes are easily broken open by debris-laden water flowing at velocities greater than 4m/ sec, which is not unusual.

13.10 Where there is no choice but to construct a retaining wall within the zone of highly erosive floodwaters it is worthwhile extending foundation excavations deeper than the depth required for bearing capacity considerations alone in the expectation that bedrock will be encountered, to obtain a stable foundation for a masonry wall. Alternatively, where the foundation is composed of a significant proportion (usually 50% or greater) of large boulders, the softer materials can be excavated and replaced by concrete to provide a stable foundation for a masonry wall.

13.11 However, it is frequently the case that neither of these foundation conditions are achievable within practicable excavations depths, and especially on the outside of river bends where scoured bedrock and boulders have been replaced by finer-grained materials. The potential for foundation scour in these situations will usually dictate that a flexible gabion structure is adopted, in preference to a more rigid masonry one (Chapter 11), and combined with whatever scour protection works are feasible under the circumstances. Foundation stability can be improved by constructing the retaining wall on a concrete raft, thus reducing differential settlements. Sacrificial walls, double thicknesses of gabion mesh, gabion mattresses (Box 13.1) and stone rip-rap are likely to prove effective during small and medium-sized flood events only, and will require regular repair or replacement. Reinforced concrete rip rap can be fabricated in situ if sufficiently large local stone rip rap is unavailable or cannot be transported to the site, as is often the case. However, the cost of fabricating rip rap to the required dimension (3m in some cases) is usually prohibitive, and it is usual to adopt a compromise solution under conditions of extreme scour potential.

Box 13.1 Gabion mattresses and aprons

When constructing gabion scour protection works it is important to create a smooth and regular surface without protrusions and to extend the measures beyond the zone of immediate vulnerability. Gabion mattresses (up to 0.3m thick) are designed to deform into scour holes, thus providing some protection against scour progressing to the wall foundation. The inner edge should be anchored or lapped beneath the wall or slope, and the outer edge can be buried and anchored well below the flood level. A mattress can also be laid on fill slopes up to the design flood level.

Gabion aprons are generally 0.5-1m thick and, although snore robust, are not capable of deforming effectively into scour holes. Where possible, they should be constructed with their top surface buried beneath the level of the flood plain, as close as possible to the expected scour depth, and if not to this depth, then at least no higher than the flood plain surface. All gabion structures built within reach of floodwaters should be laid upon or backed by filter fabric, or a graded gravel filter, to prevent fines from being washed out through the stones, and the structure undermined.

13.12 At some sites, reinforced concrete may be the only means of providing effective scour protection. This expensive option should only be contemplated at high risk sites, and where dewatered excavation for foundation can be advanced to maximum anticipated scour depths. If this is not achieved, basal scour and undermining may take place without detection from the surface, until large scale failure occurs.

RIVER TRAINING

13.13 Groynes, guide walls and other such structures will have a comparatively short life on active flood plains. However, training works can have some beneficial effects during low-moderate flow conditions and, even on hostile flood plains, they may survive for 4-5 years. Their serviceability at vulnerable sites for periods much greater than this is doubtful, although the life of a river training structure can often be extended by the use of rip rap protection, grouting and wide gabion aprons on the upstream side.

13.14 The siting and spacing of groynes should be determined by the flood flow regime identified from field evidence, although the practicalities of construction and excavation will often be deciding factors. If groynes are regarded as sacrificial, rather than permanent measures, it will probably be more worthwhile in the longer term to use capital resources to protect the road itself better.

13.15 River training can also be attempted by large-scale earth-moving on a flood plain to construct linear flood channels and boulder barrages, and to steepen the flood plain cross-gradient away from the road alignment. Such practices require heavy earth-moving plant and may lead to unpredictable redistributions of flood plain scour and side-slope erosion in the short term. In the longer term, the river will recreate its original flood flow pattern.

CROSS DRAINAGE AND TRIBUTARY FAN CROSSINGS

13.16 Where alignments are located on the lower slopes of steep valley sides, cut slopes can truncate drainage channels with the result that, during heavy rain, sediment and water may overshoot culvert inlets and discharge directly onto the road surface. This is usually remedied by constructing a large catchwall between the culvert inlet and the road edge, or by providing a dished concrete causeway. On occasions, concentrated runoff and slope failures will erode new gullies that will require some form of culverting or other drainage provision. A causeway is likely to be the only practicable remedy.

13.17 Tributary fans present a range of problems for road alignments. It is useful to differentiate between:

- mature and stable high level fans
- equilibrium fans
- immature and unstable fans.

13.18 Mature and stable fan surfaces are usually preserved as old, high level landforms that have become incised by rejuvenated stream channels. The principal problem for road construction on high level fans, other than alignment constraints, is the choice of a suitable site to cross the incised channel, bearing in mind that its banks will be composed of unconsolidated and erodible materials.

13.19 In the case of equilibrium fan surfaces, all sediment supplied to the fan is transported out of the catchment by one or a number of well defined channels. Equilibrium fans, and their wide flood plains immediately upstream, will usually comprise a number of distributary flow channels with only one or two of them occupied during normal flow conditions. Other channels may become occupied every 2-3 years or so, in response to floods or landslide - generated debris flows from further upstream. These fans are usually associated with terrace sequences on the adjacent valley floor and, therefore, represent a stage of drainage development between mature high level fans and immature, active fans on flood plains (described below).

13.20 A thorough understanding of the flow patterns across equilibrium fan surfaces is required before a road alignment and bridging structures are designed. Artificially increased channelisation and bank protection of the normal flow channel may increase its definition and capacity, in order to allow the design of a road crossing that consists of:

- a relatively short span bridge
- approach embankments
- vented causeways and drifts (multi-culverted embankments, Figure 13.1) to cater for other distributary channels.

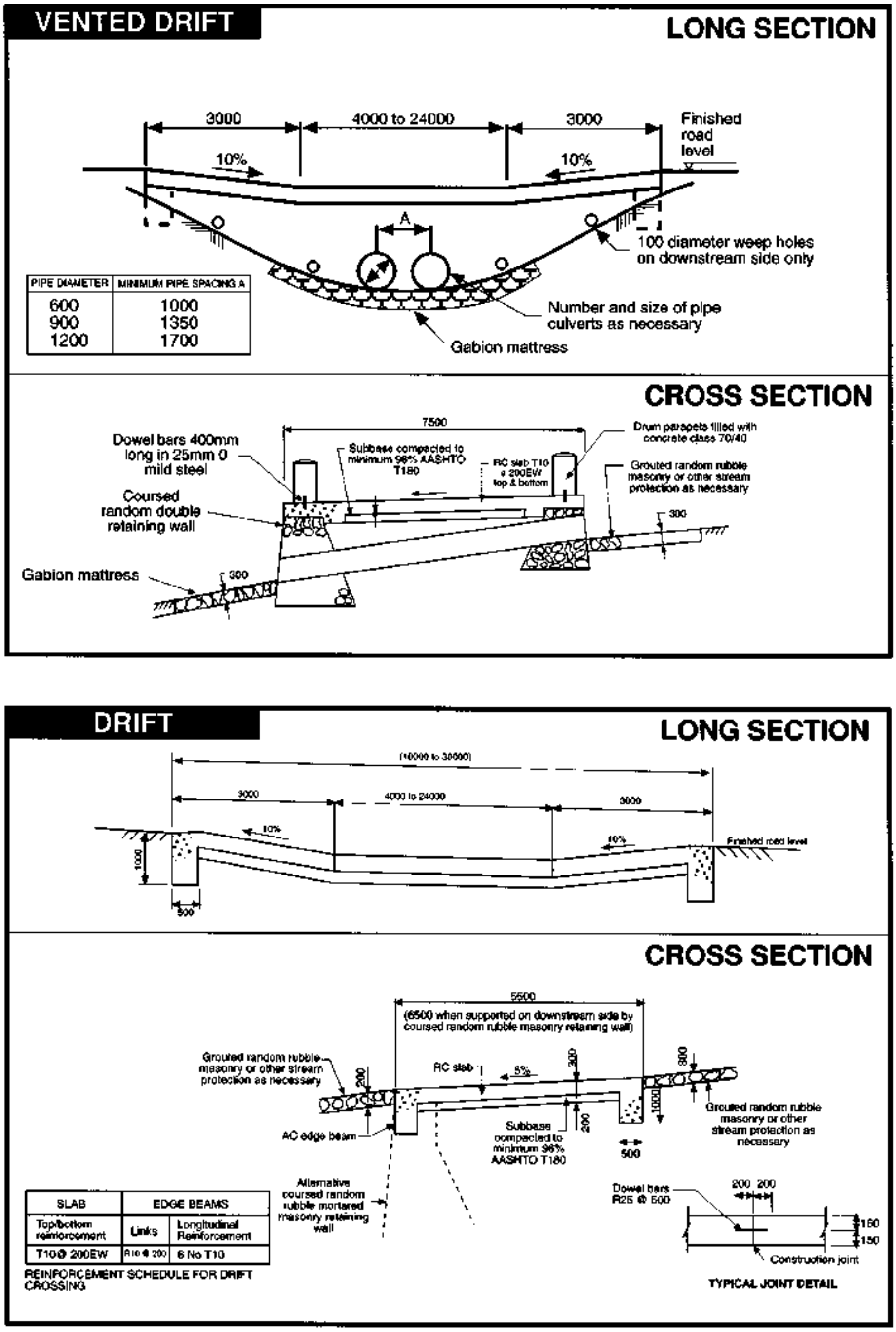
13.21 The above combination will usually be cheaper to construct and represents a more convenient solution than a multi-span bridge or a lengthy detour into the tributary to find a suitable shorter span bridging site with stable abutment and pier foundations. However, river training and erosion protection of bridge abutments, piers and approach embankments will prove costly and difficult to maintain in the long term. In addition, vented causeways should only be considered where sediment loads are likely to be low.

13.22 Immature and unstable fan surfaces are usually characterised by cyclical regimes of erosion and deposition across the whole fan surface during the course of individual storms, and a general process of fan aggradation from one year to the next. The crossing of these fans presents severe problems to valley floor alignments. Sediments will accumulate wherever flow velocity decreases as a result of an abrupt concavity in the channel profile or a sudden increase in channel width. Ideally, valley floor alignments should cross fans immediately upstream of these concavities, and if possible at the fan apex.

13.23 Bridge clearance of at least 5-7m should be provided above the existing fan surface level wherever a fan is actively aggrading and its tributary catchment is unstable. However, it is not always possible to conform with this recommendation due to alignment constraints, unstable valley sides adjacent to the fan apex, or the fact that the apex is poorly defined and the fan itself extends upstream into the tributary valley. Under these circumstances, a combination of the following may be the only viable option:

- gabion checkdams in the stream channel above the fan and erosion control in the catchment above, to control the stream bed level
- river training and scour protection works upstream and downstream of the bridge, to control the stream course
- a commitment to regular maintenance and waterway clearing operations, to keep flow within the channel and to provide room for the accumulation of debris during fan-building episodes.

13.24 Design strategies for crossing unstable fan surfaces are summarised in Box 13.2.



NOTES

1 Where water flowing over the road surface will drop to the stream bed downstream it is essential that consideration is given to scour protection and energy dissipation as described elsewhere in this document. Such considerations can result in the culvert being the preferred solution, particularly on steep ground.

2 Concrete classes

Drift slabs 250/20
Drum parapets 70/40

3 Cover to reinforcement 50

4 All laps between adjacent bars shall be a minimum of 50 bar diameters

5 Max dimension of slab without joint not to exceed 7500

6 Shape of drift crossing approach slabs to suit road geometry. Crossfall of road to be adjusted locally to match crossfall of drift crossing.

Figure 13.1 Typical details for drift crossings

x 13.2 *Options for crossing unstable fan surfaces*

A. Where rates of fan deposition are low, and where the flow path across the fan is reasonably consistent, a road can be formed on a causeway, preferably constructed in reinforced concrete or gabion. If gabion construction is used, no wire baskets should be left exposed to abrasion by passing rocks. They should be protected with a mortar rendering or equivalent durable surface

B. Cross the fan via a track that is re-cut after every aggrading or eroding storm flow. This approach will require the following considerations:

vehicular access will be prohibited during, and for a few hours after, each flood

as the fan surface builds up over time, temporary access will have to be cut deeper into the fan surface and may eventually become waterlogged and impassable during the entire wet season

an alternative to the above would be an ever-enlarging detour downstream across the fan which would increase access problems. Eventually, this option would come to an end when the detour reached the floodplain at the base of the fan

flood flows will tend to run down either side of the fan and erode the finished road on the fan approaches.

C. Select a relatively narrow channel across or, preferably, to one side of the fan surface (depending on the drainage pattern). Use river training gabions and excavation to concentrate flow through this channel. Construct a bridge over the entire width (if possible) of the fan with at least 7m clearance. This approach will require the following considerations:

river training gabions will tend to be scoured and undermined towards the fan apex

deposition of fine-grained material towards the end of each storm may bury the river training gabions to the extent that during the following storm a new channel will be formed and the existing gabions may be outflanked or destroyed

if the bridge does not extend the full width of the fan, there is a risk that it may be outflanked by changes in flow pattern across the fan surface, leading to bridge redundancy and erosion of the approach embankments.

D. Extending the concept of river training further, construct a continuous masonry or concrete spillway from the fan apex to a point downstream of the bridge. The slope and cross-section of this structure would have to be such that flow velocities were sufficient to transport bedload from the apex of the fan to the flood plain downstream. This approach will require the following considerations:

flow along the external margins of the structure, leading to undermining, is likely to occur unless its inlet is adequately keyed into both banks of the upstream channel

during peak runoff the floor of the spillway may be scoured and eventually destroyed by passing boulders

during the latter stages of storm runoff, the channel might still become blocked by fine-grained sediment which would require clearing. The shape of the channel should be made to facilitate clearing, and access to it provided for machinery.

E. Install checkdams in the gully upstream of the fan apex to retain sediment. This approach will require the following considerations:

the checkdams may be destroyed during the first few storms in a channel where aggradation of the fan itself is rapid

the volume of sediment that can be trapped behind a checkdam system is usually insignificant in comparison to the volume of transported and transportable material

artificially raising the channel bed upstream of the fan apex could easily lead to increased aggradation rates.

F. Attempt to stabilise the eroding catchment. This approach will require the following considerations:

catchments delivering large volumes of sediment to active fan surfaces cannot normally be stabilised unless the underlying cause can be identified and remedied. This is, almost without exception, not a viable option. Even if it is, the period to stabilisation will be several to many years

the catchment will extend far beyond the right of way of the road. A stabilisation programme will probably have to be integrated with a programme of agricutural extension applied to the catchment, involving the co-operation of the local community.

G. Encase the road in a concrete or masonry tunnel across the fan surface and allow flood flows and sediment to pass over the top. This option is almost certainly too expensive and unworkable on low cost roads. It would suffer the following problems:

earth pressures and water pressures behind the structure might cause it to fail unless sufficient through drainage is provided

the top surface of the structure might become damaged by boulder impact

the tunnel headroom required for vehicles would, in the early stages of fan aggradation at least, cause the structure to stand proud of the fan surface and deflect runoff to either side, resulting in scour of the approach embankments.

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Principles of low cost road engineering in mountainous regions

OS-G