17 Landslide Control and Stabilisation Measures for Mountain Roads: A Case Study of the Arniko Highway, Central Nepal

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This case study is concerned with landslide hazards along the Arniko Highway in central Nepal. Two landslides have been selected for detailed examination. This paper describes their history, their effect on human settlements and infrastructure, investigations and monitoring, the design and construction of hazard mitigation and stabilisation measures, and assessment of the effectiveness of these measures. The integrated approach used in the project for the investigation, design, and construction was found to be successful. The innovative solutions for the landslide stabilisation and river training works such as earth/rock anchors, shallow and deep drains, armoured protection with concrete blocks, and bioengineering measures proved to be highly effective. Finally conclusions are drawn and recommendations made for a more rational approach to landslide control.

Introduction

The present case study is intended as a practical illustration of the landslide-related problems encountered along roads in Nepal using specific examples from the Arniko Highway. The Arniko Highway (Figure 17.1) runs along one of the most fragile river valleys in Nepal. Since its completion in 1967, it has been severely affected by a number of hydrological disasters, including glacial lake outburst floods (GLOFs) in 1964 and 1981, and a flood in 1987. The associated landslide problems are typical of those faced in many Hindu Kush-Himalayan river valleys.

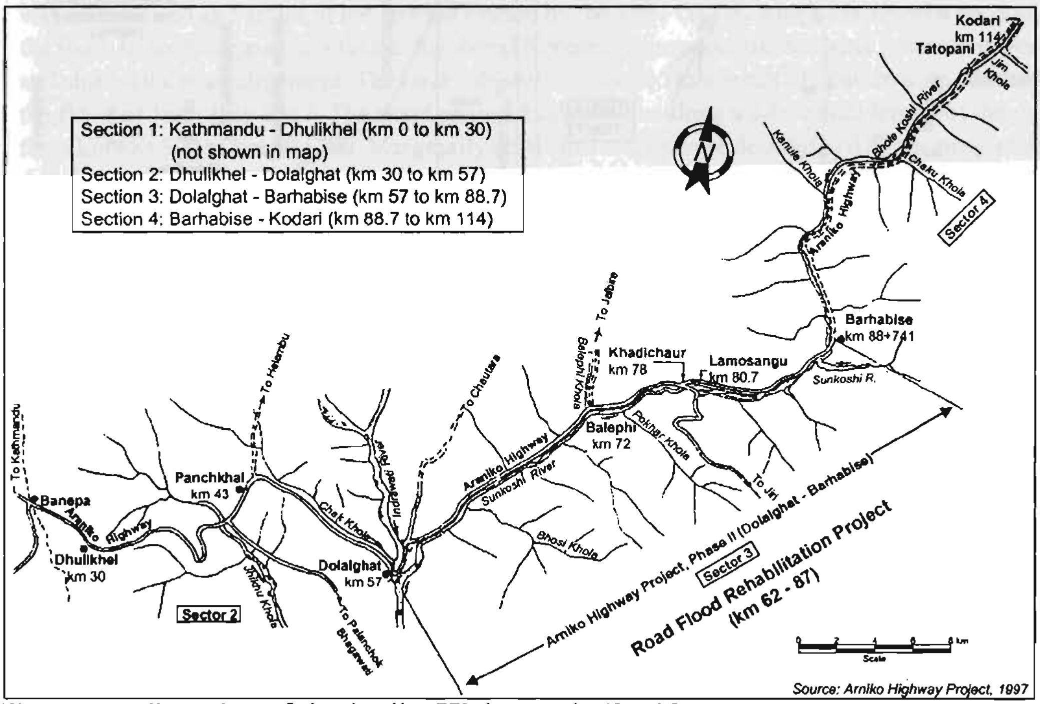


Figure 17.1: Location of the Arniko Highway (AHP 1997)

The Arniko Highway is one of Nepal's most important highways. It is the only road link to China and is classified as ' a strategic national highway linking the capital city to an international border'. In 1990 the traffic volume was about 200 vehicles per day.

The highway is an all-weather metalled road up to Barhabise (km 88 from Kathmandu) and an all-weather gravel road from there to the Chinese border (km 114 from Kathmandu). The Arniko Highway was constructed in the mid 1960s to provide the shortest link between Kathmandu and the border point at Kodari. For this reason, the alignment mostly follows the banks of the Bhotekoshi, Sunkoshi, and Chak Khola watercourses. The alignment runs across a large number of cross drains, fan deposits, landslides, and sharp rock wedges and is susceptible to bank erosion. The road is 114 km long with a width of between 3.75 and 5.5m.

The highway, has experienced a number of major hydrological disasters owing as a result of both glacial lake outbursts and incessant heavy (monsoon) rainfall leading to floods and debris flows. The stretch of highway most affected by these disasters, as indicated by the frequency of landslides and bank erosion, lies between kms 62 and 114 along the banks of the Sunkoshi and Bhotekoshi rivers. Major GLOFs occurred along the Bhotekoshi river in 1964 and 1981. In 1964 the magnitude of devastation was less as the valley was only in the initial stages of infrastructural development. The GLOF incident of 1981 had a source discharge of about 16,000 m^3/s and washed out many sections of the highway and two major bridges. The presence of glacial lakes in the Bhotekoshi catchment area is shown in Figure 17.2. The discharge attenuation along the river from the GLOF in 1981 is shown in Figure 17.3.

The highway experienced another catastrophic flood in July 1987 that resulted in disastrous washouts and major erosion along the Sunkoshi, Charnawati, and Tungbhadra rivers. More than a hundred human casualties and heavy loss of cattle, houses, and cultivated land were reported. A number of suspension bridges, and a substantial length of the Arniko Highway and Lamosangu-Jiri road were damaged or washed out.

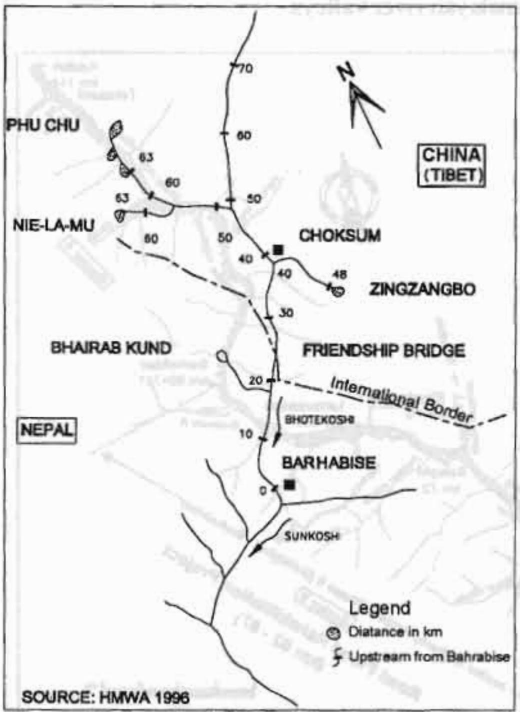


Figure 17.2: **Glacial lakes in the Bhotekoshi catchment (ITECO 1997)**

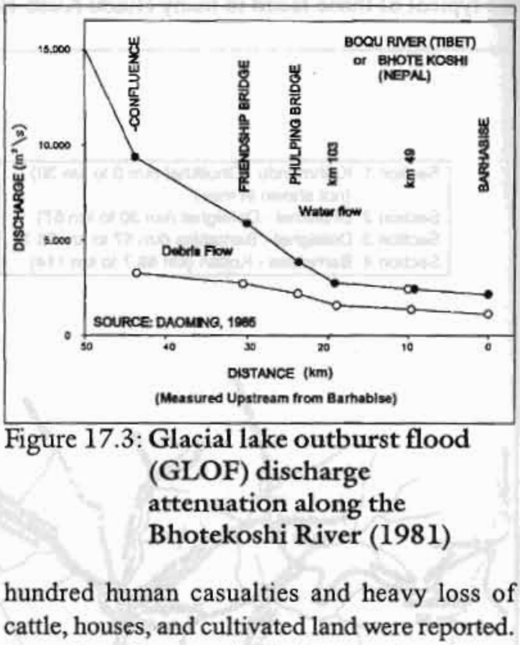


Figure 17.3: **Glacial lake outburst flood (GLOF) discharge attenuation along the Bhotekoshi River (1981)**

The Landslides

Following the catastrophic cloudburst and subsequent debris flow along the Sunkoshi valley on June 30th to July 1st 1987, the Department of Roads (DoR), with assistance from the Swiss Development Co-operation (SDC), carried out a flood disaster appraisal along the most severely affected sector of the Arniko Highway with ITECO as its consultant. It was found that more than a quarter of the road between kms 62 and 87 had been either totally washed out or partially damaged. HMGN entered into an agreement with the International Development Agency (IDA) in 1989 to rehabilitate the same stretch under the Road Flood Rehabilitation Project (RFRP) in parallel with the Swiss funded DoR investigation of the slope conditions. The landslides were monitored by engineering geological and geophysical surveys, exploratory drilling, geotechnical testing, and piezometric monitoring under the Arniko Highway Rehabilitation Project. In turn RFRP carried out surveys, investigation, and design work in 1990, construction work in 1991 to 1993, and defect liability work in 1994.

Detailed description

Of the ten problematic sites included under the RFRP, seven had landslide problems. The landslides at km 69+000 (AH-1) and km 72+500 (AH-4) were chosen for this case study.

Landslide at km 69+000 (AH-1)

This landslide is located between km 68+900 and km 69+200 of the Arniko Highway. It faces south-east and is bounded by the Sunkoshi River at its toe, by a small torrent to the north, and by settlements on its western and southern sides. Figure 17.4 shows a plan of the landslide and Figure 17.5 an aerial view.

Before 1981, the landslide area was mostly occupied by paddy fields and grazing land. The slope was destabilised as a result of toe erosion caused by the 1981 GLOF, which destroyed a section of the road. In an emergency operation, the Royal Nepalese Army constructed bank protection works and shifted the road alignment. The landslide was reactivated as a result of bank erosion caused by the flood of June/July 1987. The flood caused bank erosion along a substantial length of the road from km 68+500 to km 69+200. Marginally stable hill slopes were destabilised by the progressive development of this landslide.

After the 1987 flood, the landslide was monitored and stabilised through extensive drainage works, bioengineering, and a few structural measures. The initial development of the landslide had been rotational, which was followed by a series of slumps, tension cracks, and translational slips. Though the reactivated depth of the slide was shallow, there was a potential for a deep-seated failure.

The landslide area was made up of colluvial material with a sandy-silty matrix with isolated boulders of up to 4m across. The colluvial slope extended from the Sunkoshi River bank to 600m above it. The landslide involved more than 100,000 m^3 of colluvial material. It covered an area of 300m along and 400m above the road and was destabilising paddy fields, dry crops, and settlements. Minor symptoms of instability and tension cracks extended a further 200m.

The reactivation of the landslide in 1987 created havoc in nearby settlements. People had to abandon valuable cultivation land and started to move their houses to safer locations. A 132 kV electric pylon was destroyed. From 1987-1990 landslide activity, characterised by minor slumps, slips, and creeping of soil mass, was moderate.

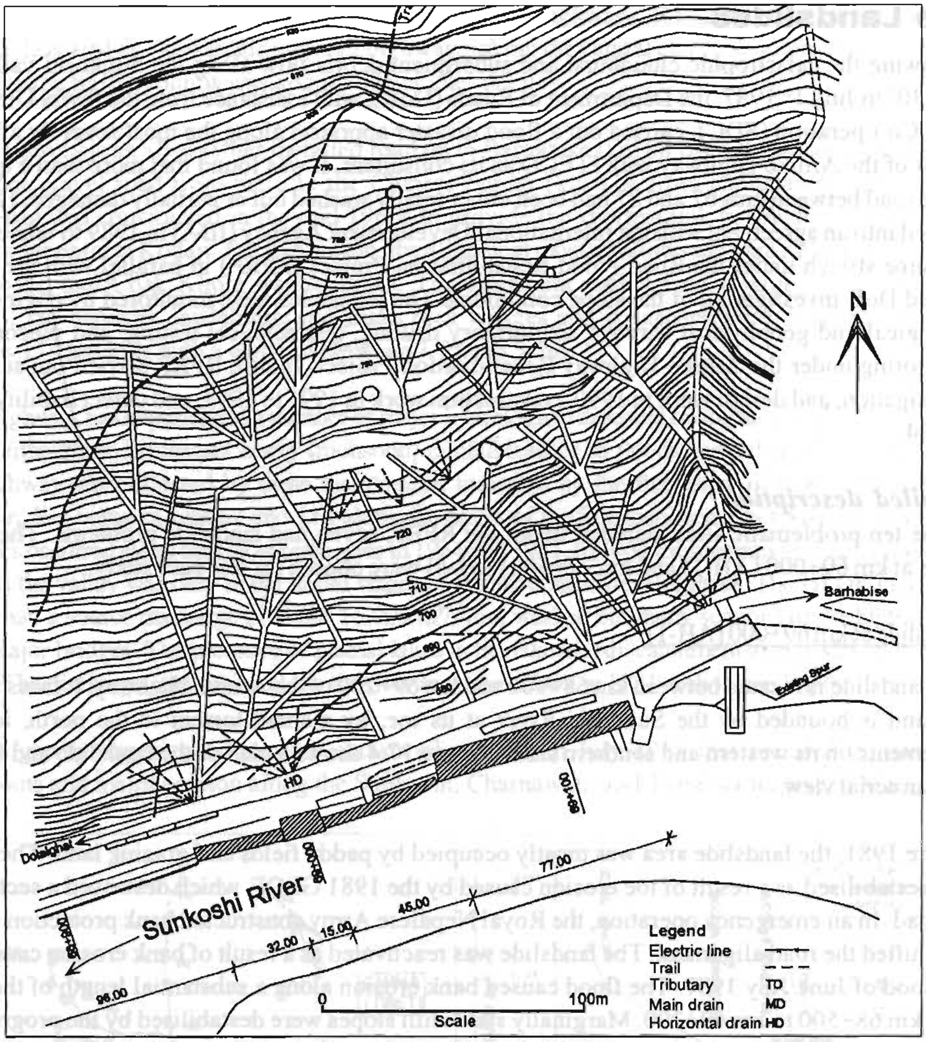


Figure 17.4: Plan of landslide AH-1 (RFRP 1990)

The main triggering factor responsible for the sliding was the bank erosion of the Sunkoshi River. The propagation of the landslide from the riverbank to 400m above the road is attributed to the saturation and high ground water table, which was re-charged by the irrigated paddy fields above and within the unstable area. Photographs taken from a helicopter illustrate the extent of the landslide (Figure 17.5).

Landslide at km 72+500 (AH-4)

Landslide AH-4 was located at km 72+600 of the Arniko Highway near Balephi village. The landslide area was bounded by the Sunkoshi River to the east, Balephi village to the west and south, and a small torrent to the north. The landslide faced due south-east. Figure 17.6 shows the plan of the landslide and Figure 17.7 an aerial view taken from a helicopter.

The massive GLOF of 1981 had extensively eroded the Sunkoshi River bank at this site and destroyed a portion of the road. In an attempt to reopen the road, the alignment had been shifted

Figure 17.5: Aerial photograph of landslide AH-1

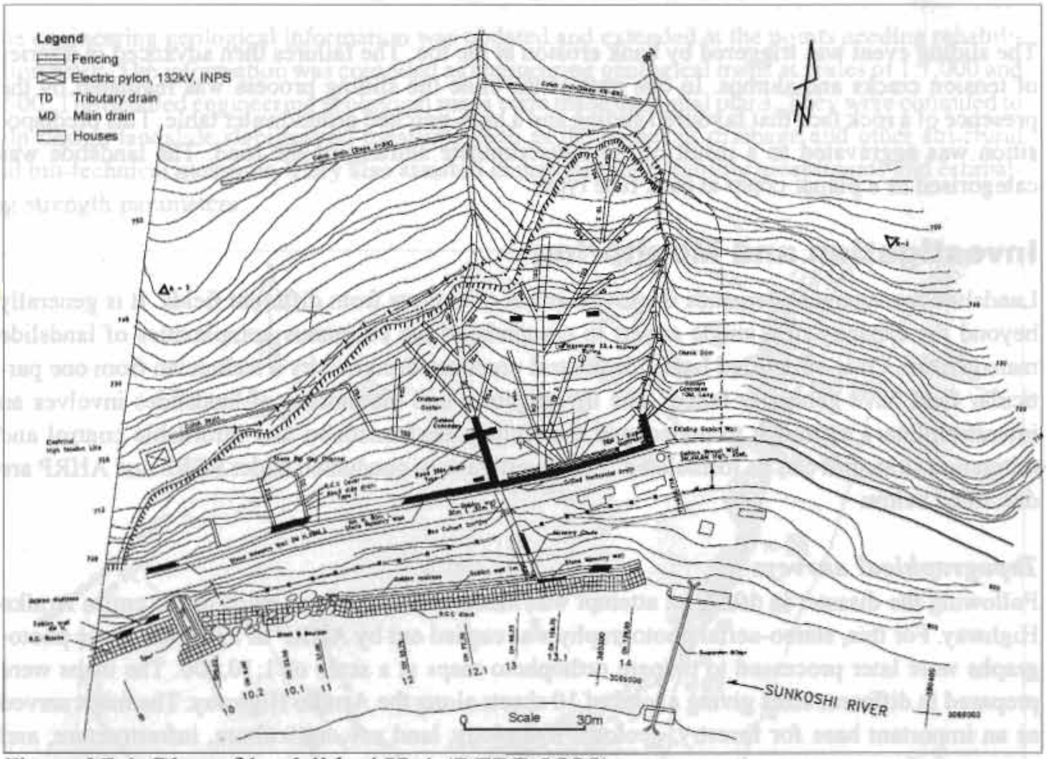


Figure 17.6: Plan of landslide AH-4 (RFRP 1990)

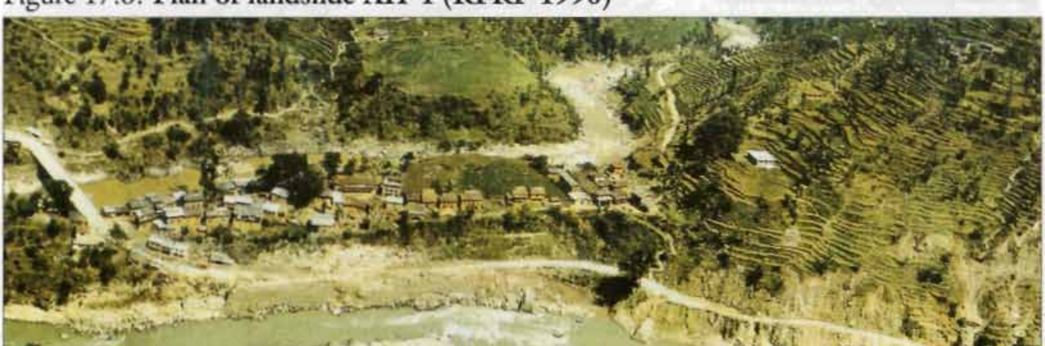


Figure 17.7: Aerial photograph of landslide AH-4 (AHRP 1989)

towards the hillside. However, this destabilised the fragile colluvial slope. The flood event of July 1987 further jeopardised the stability of the slope. Emergency measures such as gabion walls and spurs were constructed in 1988. The landslide was studied by AHRP (1989) and during the RFRP design phase (1990). These studies provided the basis for designing and building mitigation measures, which were completed in 1994.

The landslide consisted of colluvial debris and gravels with a silty to clayey matrix. About 30,000 m^3 of material was involved. The landslide affected an area 150m along and 100m above the road and had destroyed a number of cultivated terraces. Some tension cracks were noted above the crest of the active part of the landslide. The growth of the landslide and bank erosion along the Sunkoshi River had led to the loss of at least two hectares of cultivated land and a few houses at Balephi. An electric pylon was also endangered. The landslide remained active from 1987 to 1989 and was still in a moderately active state during the initial stages of RFRP.

The sliding event was triggered by bank erosion at the toe. The failures then advanced in a series of tension cracks and slumps. In one part of the slide the sliding process was regulated by the presence of a rock face that favoured sliding and a high-perched groundwater table. This predisposition was aggravated as a result of the indiscriminate shifting of the road. The landslide was categorised as a planar creep at rock face type.

Investigation and Monitoring

Landslide management demands the involvement of experts from different fields. It is generally beyond the capacity of a single expert to comprehend the enormous complexities of landslide management. Over-simplified landslide control practices designed by a technician from one particular field have generally failed. The investigation and monitoring of landslides involves an interdisciplinary approach to understand the sliding mechanism so that affordable control and mitigation strategies can be formulated. The investigations conducted under RFRP and AHRP are described below.

Topographical surveys

Following the disaster in 1987, an attempt was made to prepare base maps of the entire Arniko Highway. For this, stereo-aerial photography was carried out by AHRP in 1989, and these photographs were later processed to prepare orthophoto maps at a scale of 1:10,000. The maps were prepared in different sizes giving a total of 10 sheets along the Arniko Highway. The maps served as an important base for forestry, geology, hydrology, land use, agriculture, infrastructure, and geomorphology assessments.

Photographs taken from a helicopter were joined together to give a panoramic view of the road. A precise traverse survey was made from Dhulikhel (km 30 from Kathmandu) to Kodari (km 114) with connection to neighbouring geodetic control points along the highway, to serve as a baseline traverse for subsequent road management interventions.

A detailed topographical survey was carried out for the sections of the landslides intended for rehabilitation or mitigation (Figures 17.4 and 17.6). This survey was connected to the baseline traverse. The topographical survey marked tension cracks, big boulders, property lines, springs, creeks, gullies, banks, the road and related structures, prominent trees, electric pylons, houses, drains, and public utilities. The extent of the detailed surveys is shown in Table 17.1.

Table 17.1: Extent of the surveys for the two case study landslides				
Landslide	Area covered	Contour interval	Scale of plan	Scale of sections
AH-1	600 x 400	2m	1:1000	1:500 / 1:200
AH-4	600 x 200	2m	1:500	1:500 / 1:200

Engineering geological survey

In 1989, an engineering geological survey of the road corridor between Dhulikhel (km 30) and Kodari (km 114) was conducted as part of the AHRP over the strip out to 50m either side of the road. The survey line was connected to a baseline, which was extended through a compass traverse. The maps included information on rock and soil type, land use, slides, banks, water line, erosion features, exploration pits, bore holes, tension cracks, settlements, slope indicators, and rock structures at a scale of 1:5,000. The engineering geological maps of sites AH-1 and AH-4 are presented in Figures 17.8 and 17.9.

The engineering geological information was updated and extended at the points needing rehabilitation work. The information was compiled as engineering geological maps at scales of 1:1,000 and 1:500. The detailed engineering geological maps were made as spatial plans. They were compiled to help design landslide stabilisation measures such as the layout of drainage and other structural and bio-technical measures. They also assisted in making slope stability assessments and estimating strength parameters.

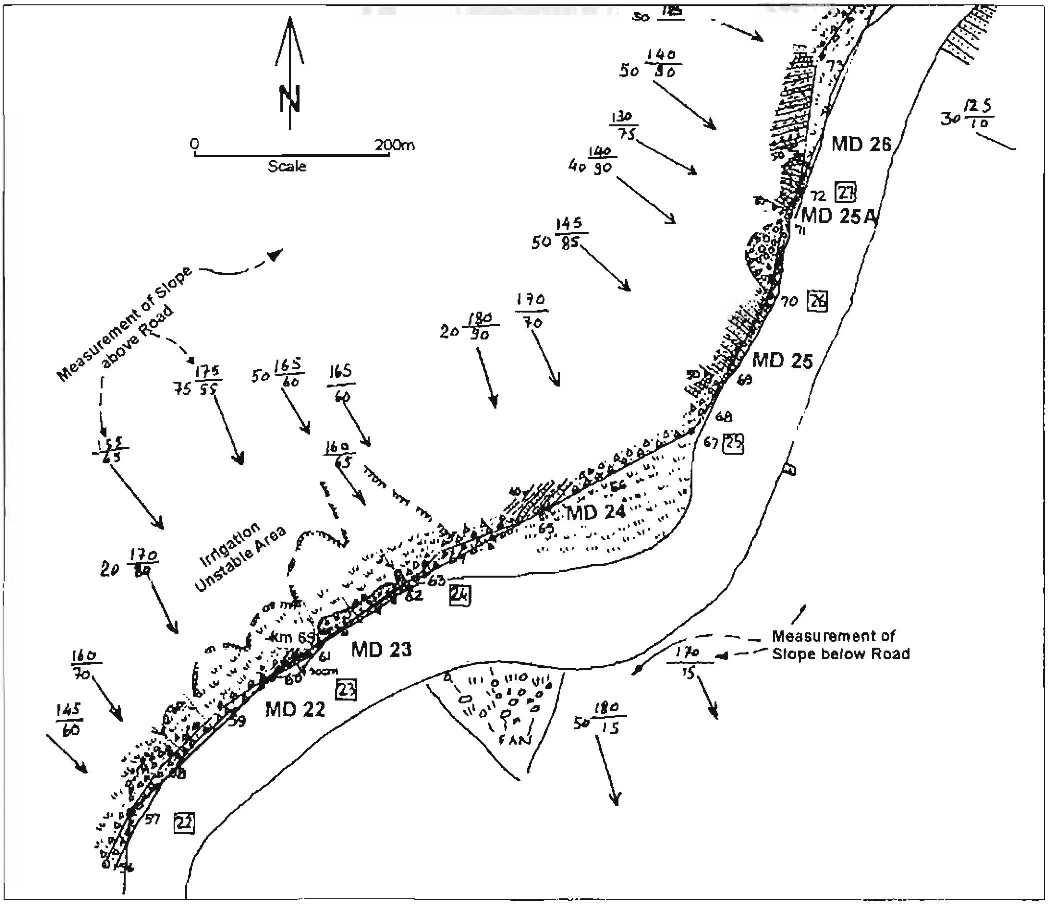


Figure 17.8: **Engineering geology of landslide AH-1 (AHRP 1989)**

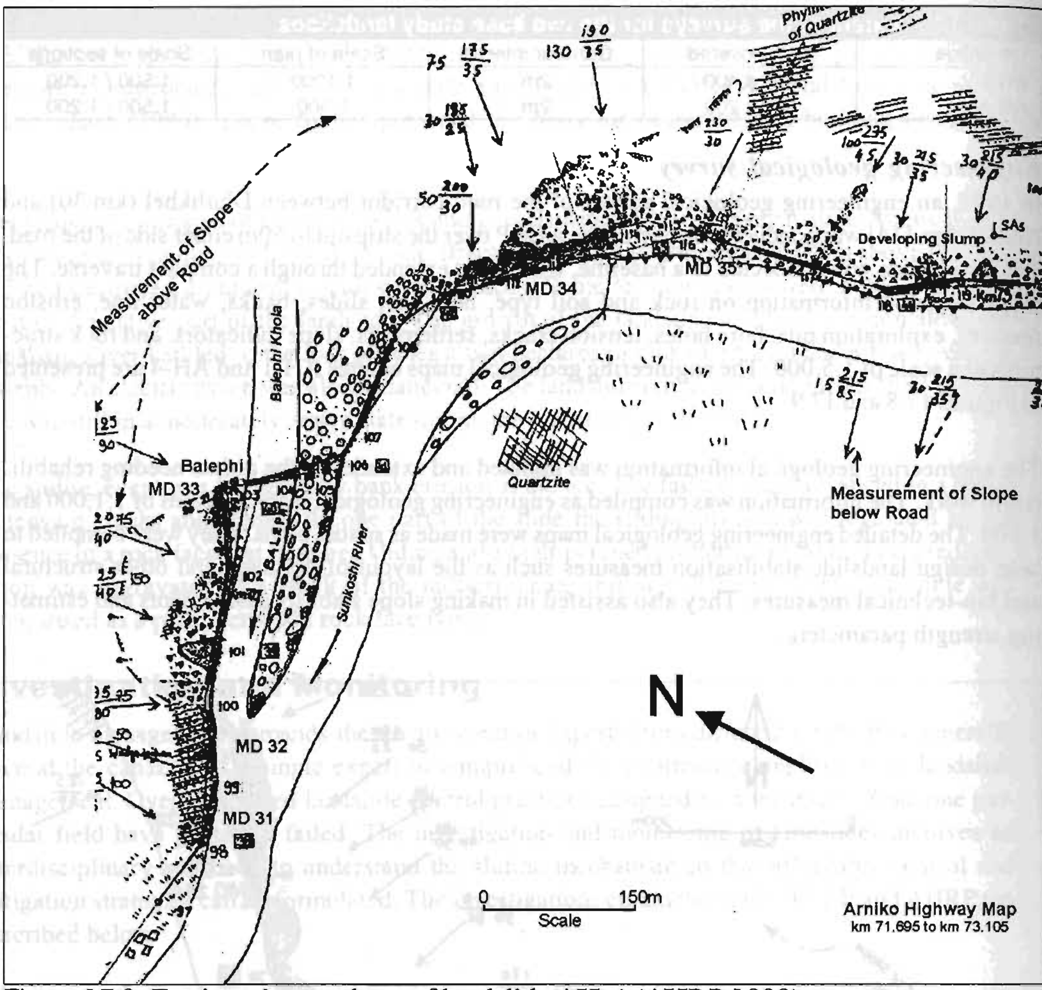


Figure 17.9: Engineering geology of landslide AH-4 (AHRP 1989)

Geophysical investigation

Sub-surface explorations are imperative to understand the mechanism of a landslide. Exploration depths of 20-25m are generally sufficient, but depths of up to 60m are needed for major landslides with deep-seated failures. The geophysical investigation of landslides involves making electrical resistivity profiles and using seismic refraction. A combination of these two approaches has been found to give a reasonable basis for interpretations of sub-surface conditions. The seismic reflection method has been used recently to explore to greater depths with a better level of confidence.

Geophysical investigations were carried out for most of the medium-sized and major landslides along the Arniko Highway. The findings were calibrated using engineering geological and exploratory drillings. The finely interpreted rock and soil strata were used for slope stability analysis. The geophysical sections of landslides AH-1 and AH-4 are presented in Figures 17.10 and Figure 17.11.

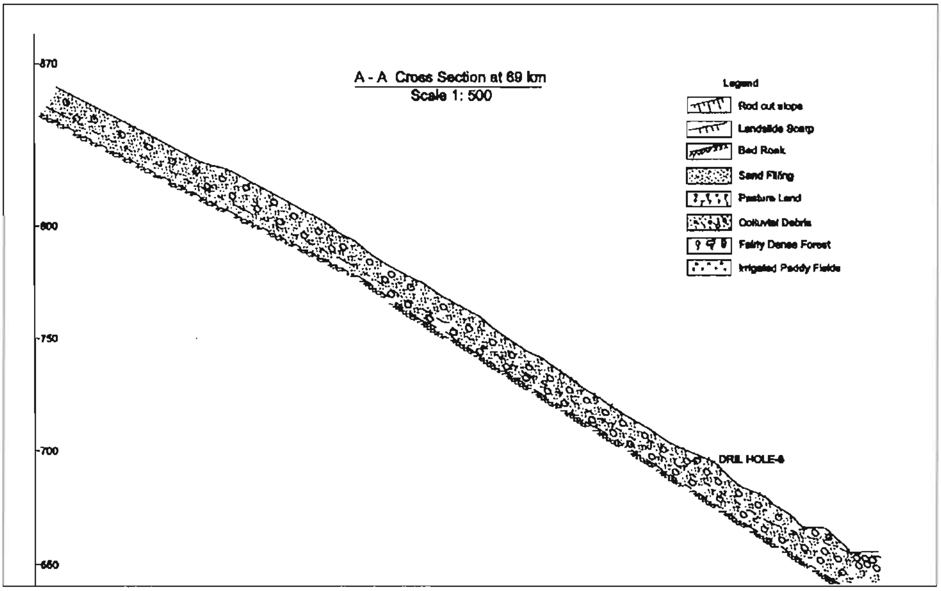


Figure 17.10: **Geophysical section of landslide AH-1** (AHRP 1989)

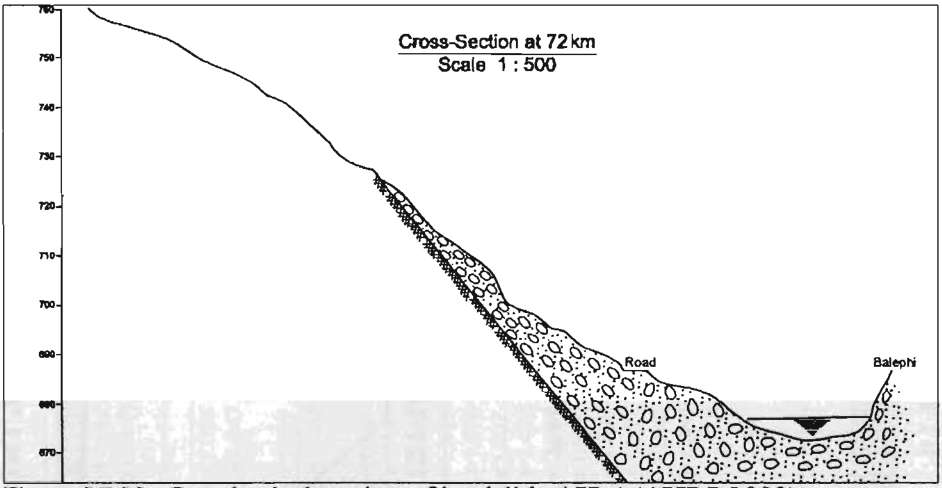
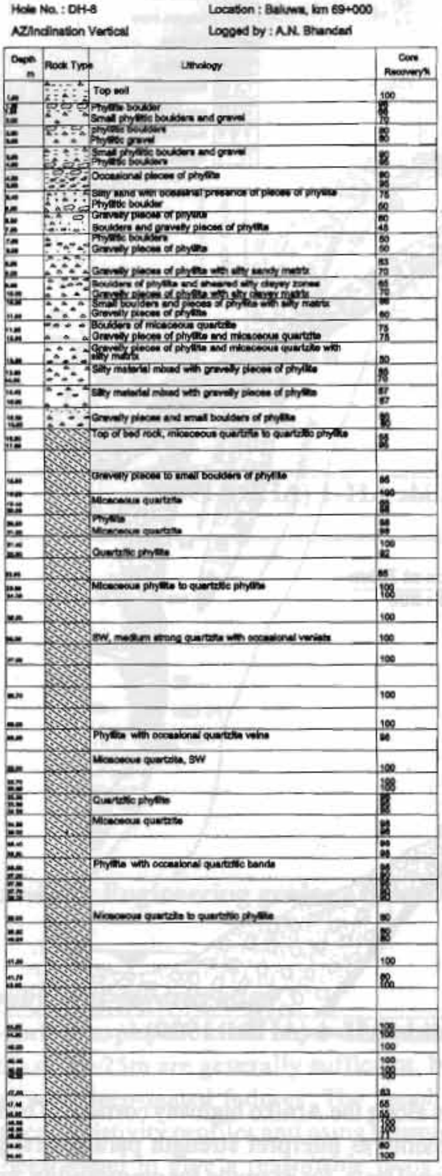


Figure 17.11: **Geophysical section of landslide AH-4** (AHRP 1989)

In 1989 bore holes were drilled at all major landslides along the Arniko highway corridor. The bore logs proved very useful for slope stability assessments to interpret strength parameters, pore pressure ratios, and groundwater table levels. Additional tests, for example for permeability, grain-size analysis, and measurement of groundwater levels, were also undertaken. The bore-hole data for landslides AH-1 and AH-4 are presented in Figures 17.12 and 13.

Geo-technical investigations

Geo-technical investigations are essential to characterise in-situ soil and rock materials. The geotechnical investigations carried out at the landslides included digging test pits on the landslides to a depth of 1.5m. Soil samples were collected for laboratory analysis to determine the bulk density, moisture content, unit weight, and grain size distribution of the soil. Field determinations of permeability, SPT values, surface toughness, weathering grade, and friction angle were also



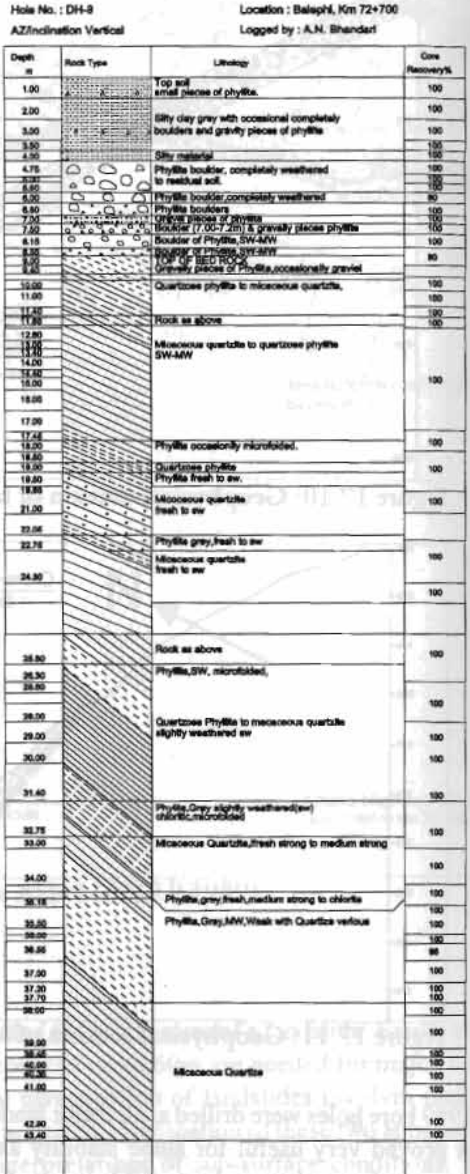


Figure 17.12: **Borehole log for landslide AH-1 (AHRP 1989)**

Figure 17.13: **Borehole log for landslide AH-4 (AHRP 1989)**

carried out. Material samples were also collected from prospective quarry areas to determine their suitability for construction purposes as backfill.

The findings formed the basis for the computation, analysis, and design of landslide mitigation measures. These investigations were carried out at all major landslides. A summary of the findings is given in Table 17.2.

Table 17.2: **Summary of Geotechnical Test Results**

Position (km)	Natural moisture content (%)	Atterberg limits (%)	Unit weight (gm/cm^3)	Specific gravity and void ratio	Sieve analysis (%)	Hydrometer analysis (%)	Gradation ratios	Strength para-meters (kg/cm^2, deg)	Field Permeability (cm/s)
Landslide materials									
69	ω_n=11.1	LL=19	γ=2.053 γ_d=1.847		G=65 S=27.5 F=7.5	M=98.1 C=1.9	D10=0.07 C_u=200 C_c=1.02		k=0.0066
69	ω_n=17.5		γ=1.707 γ_d=1.452	G_s =2.63 e=0.811	G=58.5 S=35.1 F=6.4	M=96.8 C=3.2	D10=0.08 C_u=102.5 C_c=0.644	ϕ=32.5	k=0.0641
69	ω_n=17.7	LL=26 PL=19 PI=7	γ=1.748 γ_d=1.485 γ_s=2.085					C=0.05 ϕ=28.6	
69	ω_n=15.9		γ=1.943 γ_d=1.676						
70.2	ω_n=21.3	LL=24 PL=18 PI=6	γ=1.707 γ_d=1.407 γ_s=2.054	G_s =2.62 e=0.864	G=24.2 S=50.7 F=25.1	M=88.4 C=11.6	D10=0.03 C_u=10.6 C_c=0.487	ϕ=30.5	k=0.579
72	ω_n=13.1	LL=26 PL=20 PI=6	γ=1.511 γ_d=1.335 γ_s=1.787	G_s =2.61 e=0.954		M=97.7 C=2.3			k=0.239
72	ω_n=22.6	LL=25 PL=19 PI=6	γ=1.699 γ_d=1.385		G=80.0 S=17.8 F=2.2		D10=0.37 C_u=216.2 C_c=1.34		k=0.041
72.3								ϕ=35	k=0.011

Cont't ...

Table 17.2: **Summary of Geotechnical Test Results (cont'd)**

Position (km)	Natural moisture content (%)	Atterberg limits (%)	Unit weight (gm/cm³)	Specific gravity and void ratio	Sieve analysis (%)	Hydrometer analysis (%)	Gradation ratios	Strength para-meters (kg/cm², deg)	Field Permeability (cm/s)
72.3		LL=23 PL=19 PI=4			G=55.0 S=36.6 F=8.4	M=94.1 C=5.9	D10=0.07 C_u=71.4 C_c=0.686		k=0.077
72.3				G_s =2.60	G=74.9 S=20.9 F=4.2		D10=0.14 C_u=72.1 C_c=7.701	ϕ=36.5	
72.3	ω_n=13.2	LL=28 PL=21 PI=7	γ=2.011 γ_d=1.776 γ_s=2.177			M=98.7 C=1.3			
Backfill materials									
62.3			γ_m=2.153 OMC=6.4		G=78.2 S=21.8 F=0		D10=0.25 C_u=96 C_c=5.226	C=0 ϕ=29	
66.3			γ_m=2.153 OMC=6.4		G=68.8 S=21.1 F=10.1		D10=0.06 C_u=301.5 C_c=3.341	C=0 ϕ=33.2	

<u>Legends</u>

- ω_n = natural moisture content (%)
- LL = liquid limit (%)
- PL = plastic limit (%)
- PI = plasticity index (%)
- γ = field density (gm/cm³)
- γ_d = dry unit weight (gm/cm³)
- γ_s = saturated unit weight (gm/cm³)
- G_s = specific gravity
- e = void ratio
- G = gravel (%)
- S = sand (%)
- F = fines (%)
- M = silt (%)
- C = clay (%)
- D10 = sieve passing 10% material
- C_u = coefficient of uniformity
- C_c = coefficient of convergence
- C = cohesion (kg/cm²)
- ϕ = friction angle (deg)
- k = permeability (cm/s)
- γ_m = max. dry unit weight (gm/cm³)
- OMC = optimum moisture content (%)

Hydrological investigations

Most of the disasters along the Arniko Highway have occurred as a result of GLOFs or periods of prolonged heavy rainfall. All available meteorological and hydrological data for the Sunkoshi catchment were therefore collected and analysed. The maps showing snow coverage, isohyetals, longitudinal profiles and bed slopes, the catchment area, and the locations of meteorological and hydrological stations are shown in Figures 17.14 to 17.18. The probability of major floods happening over return periods of from two to a hundred years were calculated for landslides AH-1 and AH-4. The probable flows for the project sites were computed by correlation with the gauging stations 610, 620, and 630 (Table 17.3).

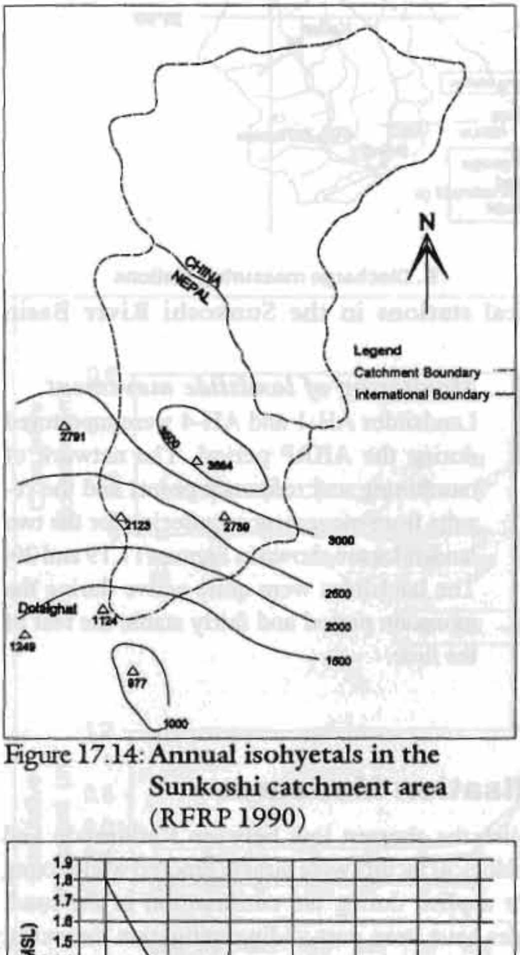


Figure 17.14: Annual isohyetals in the Sunkoshi catchment area (RFRP 1990)

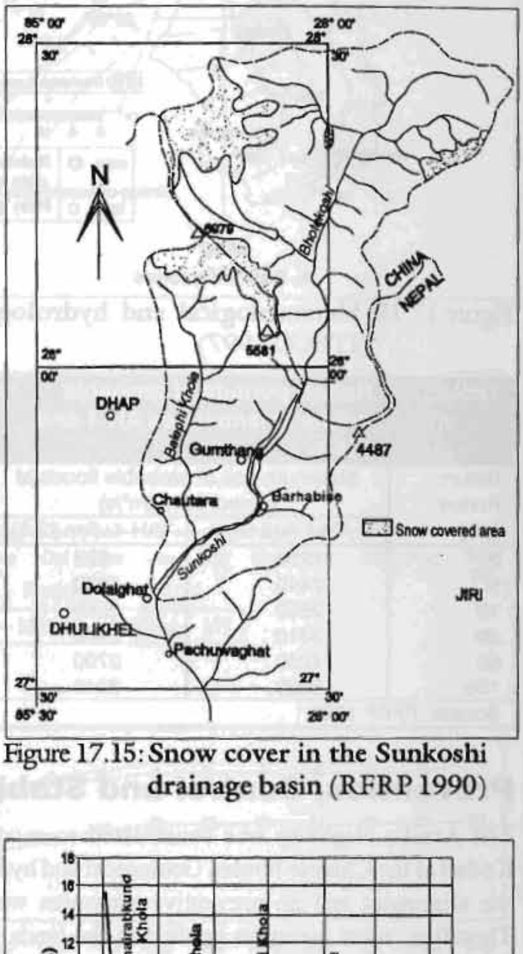


Figure 17.15: Snow cover in the Sunkoshi drainage basin (RFRP 1990)

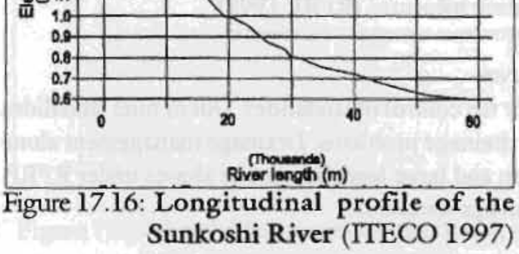


Figure 17.16: Longitudinal profile of the Sunkoshi River (ITECO 1997)

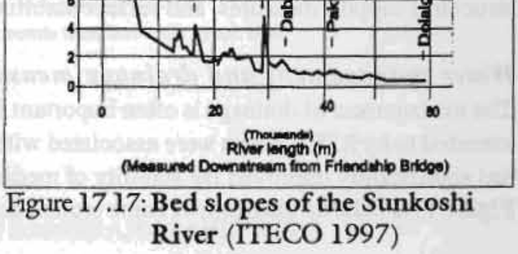


Figure 17.17: Bed slopes of the Sunkoshi River (ITECO 1997)

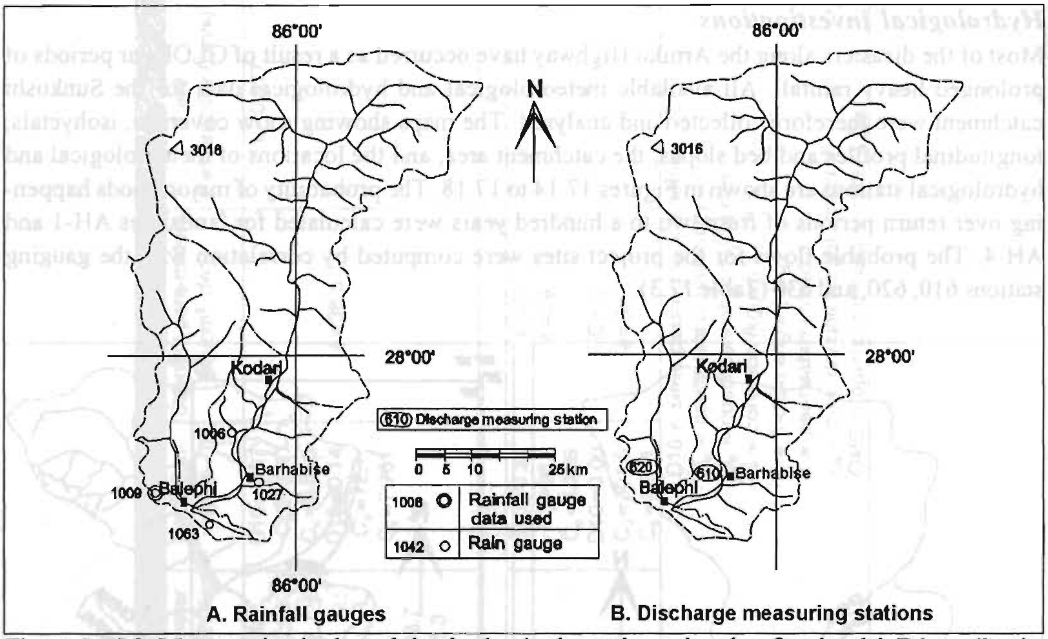


Figure 17.18: Meteorological and hydrological stations in the Sunkoshi River Basin (ITECO 1997)

Table 17.3: **Return periods of probable floods at landslides AH-1 and AH-4**

Return Period (yrs)	Maximum size of probable floods at project sites (m^3/s)	
	AH-1 (km 69)	AH-4 (km 72.5)
2	1250	923
5	2440	1930
10	2850	2260
20	3310	2400
50	4230	2700
100	5000	3340

Source: RFRP 1990

Monitoring of landslide movement

Landslides AH-1 and AH-4 were monitored during the AHRP period. The network of monitoring and reference points and the results from movement monitoring for the two landslides are shown in Figures 17.19 and 20. The landslides were quite active during the monsoon period and fairly stable the rest of the time.

Prevention, Control and Stabilisation Measures

The Arniko Highway was constructed to establish the shortest link between Kathmandu and Kodari at the Chinese border. Geological and hydrological factors were largely ignored while fixing the alignment and no preventive measures were applied during the construction of the road. Therefore, most measures applied to the landslides have been post-sliding mitigation measures. This has been the case for most mountain roads built in Nepal during the 1960s and 1970s. This section therefore mainly covers post-sliding measures including water management and drainage, structural support measures, and surface stabilisation measures (RFRP 1993)

Water management and drainage measures

The management of drainage is often important for the control of landslides. Out of nine landslides attended to by RFRP, seven were associated with drainage problems. Drainage management alone has significantly improved the stability of medium and large landslide-prone slopes under RFRP. Figure 17.21 shows diagrams of some typical drainage measures.

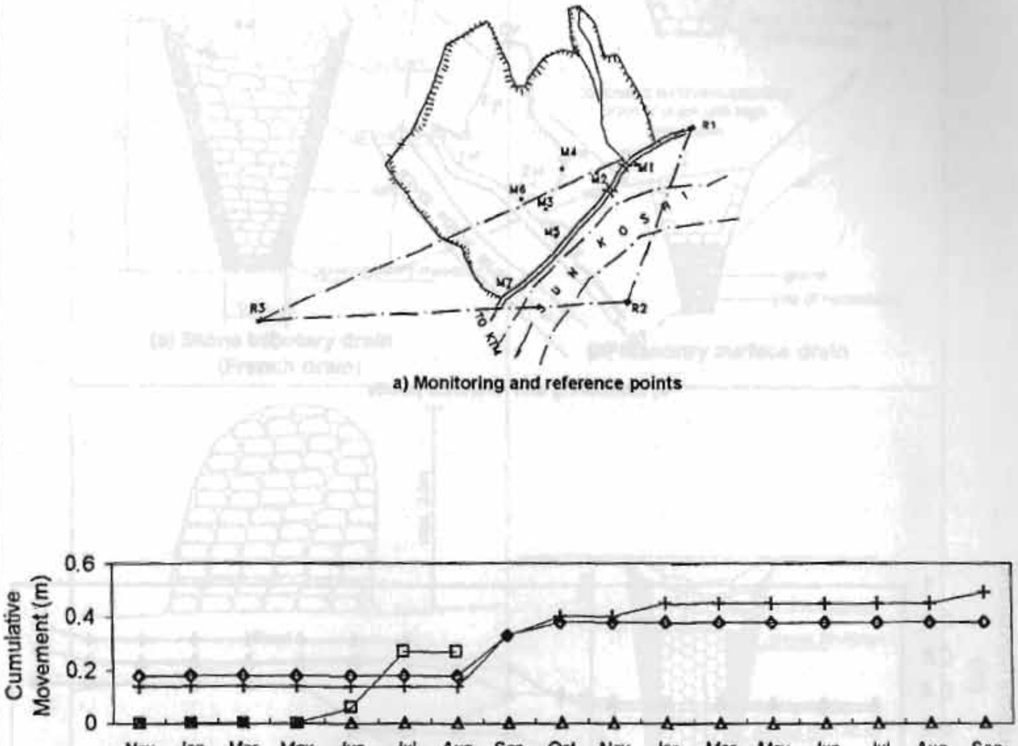


Figure 17.19: Landslide movement monitoring at landslide AH-1 (AHRP 1990)

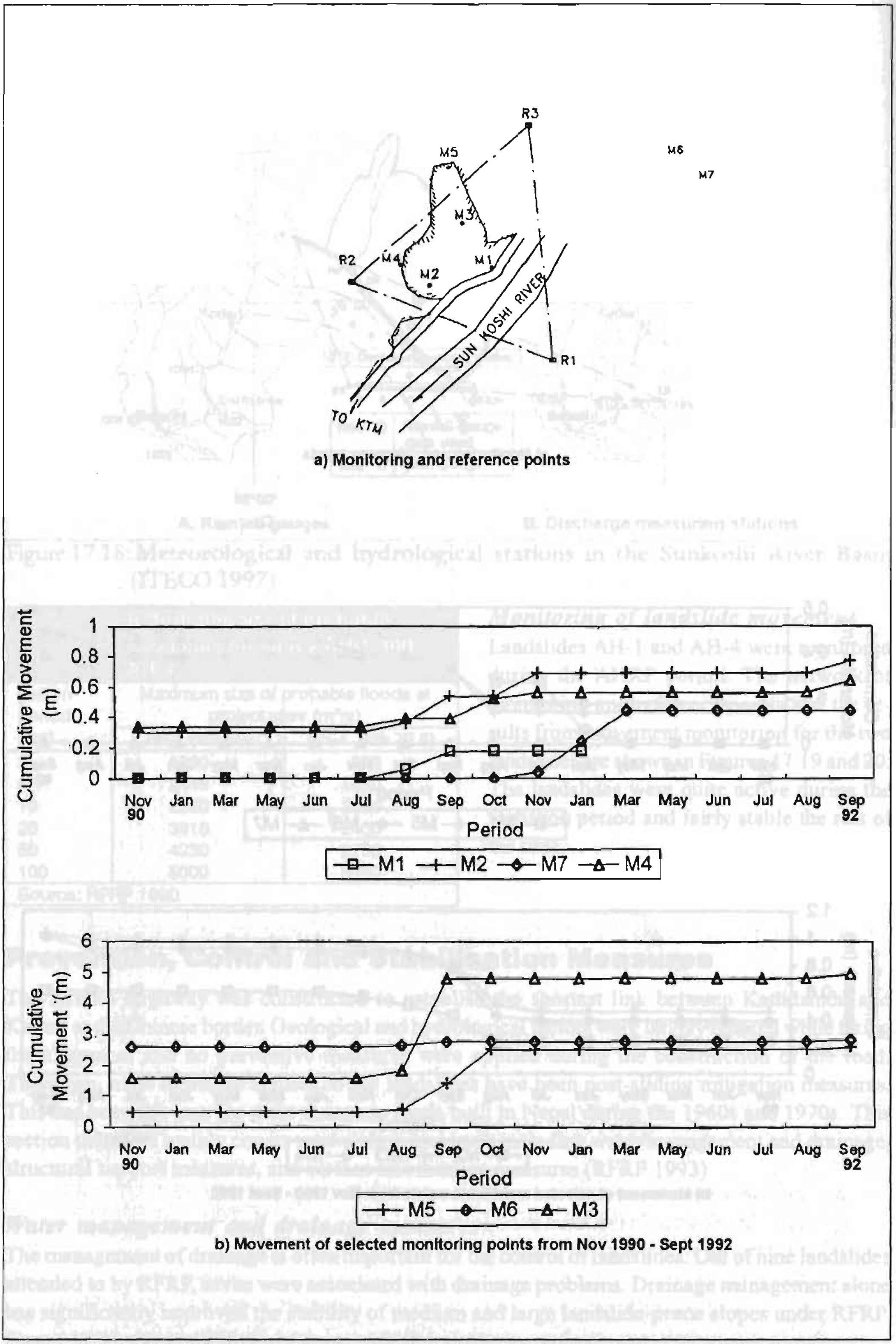


Figure 17.20: **Landslide movement monitoring at landslide AH-4** (AHRP 1990)

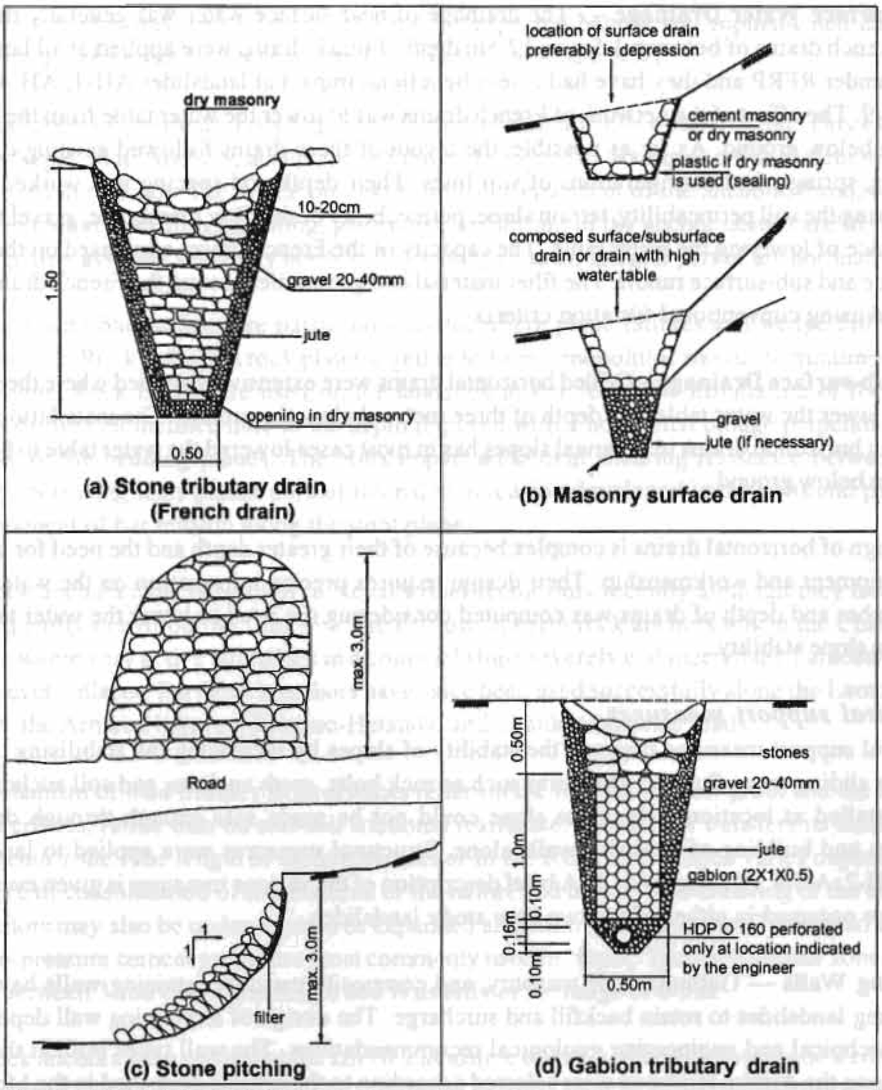


Figure 17.21: Typical drainage measures

Surface Water Drainage – The surface runoff at landslides AH-1 and AH-4 was drained off by installing trapezoidal catch drains lined with stone masonry. The capacity of the catch drains was designed to accommodate surface runoff for a precipitation intensity of a once in 25 years flood event. Catch drains and out-fall gullies were positioned according to the recommendations of an engineering geologist. Catch drains were installed at seven of the nine landslides. The improvement of surface drainage largely mitigated rill and gully erosion.

Another important aspect was drainage management within the area of the landslides. The size of the landslides ranged from between one and ten hectares. The corresponding surface runoff within the landslides was up to 1 m³/sec. To manage this level of surface runoff safely, a network of riprap channels was constructed. The installation of cascading lined catch drains, to deal with surface runoff was not usually appropriate due to the continued movement within most of the landslides. The riprap channels were aligned and constructed along existing rills, gullies, depressions, and watercourses to harmonise with the micro-hydrology within the landslide.

Near-Surface Water Drainage — The drainage of near surface water was generally managed using French drains of between 1.5m and 2.5m depth. French drains were applied at all landslides treated under RFRP and they have had a very beneficial impact at landslides AH-1, AH-4, AH-8 and AH-9. The effect of the network of French drains was to lower the water table from the surface to 1-2m below ground. As far as possible, the layout of these drains followed existing drainage channels, springs, and configurations of slip lines. Their depth and spacing was worked out by considering the soil permeability, terrain slope, permeability of the filter membrane, gravel fill, and importance of lowering the water table. The capacity of the French drains was based on the levels of surface and sub-surface runoff. The filter material and geotextiles around the French drains were designed using conventional filtration criteria.

Deep Sub-surface Drainage – Drilled horizontal drains were extensively applied where there was a need to lower the water table to a depth of three metres below the ground. The installation of 15-20m long horizontal drains in colluvial slopes has in most cases lowered the water table to between 4 and 6m below ground.

The design of horizontal drains is complex because of their greater depth and the need for specialised equipment and workmanship. Their design requires precise information on the water table. The number and depth of drains was computed considering the need to lower the water table for optimum slope stability.

Structural support measures

Structural support measures improve the stability of slopes by increasing the stabilising component of a sliding mass. Support structures such as rock bolts, earth anchors, and soil anchors have been installed at locations where the slope could not be made safe enough through drainage measures and building of retaining walls alone. Structural measures were applied to landslides AH1, AH-2, AH-4, AH-5 and AH-8. A brief description of the various measures is given even when they were not used in either of the two case study landslides.

Retaining Walls — Gabion, stone masonry, and composite masonry retaining walls have been built along landslides to retain backfill and surcharge. The design of a retaining wall depends on the geotechnical and engineering geological recommendations. The wall types built at the landslides along the Arniko Highway were selected according to the criteria described in the Mountain Risk Engineering Handbook (Deoja et al. 1991). Gabion walls were designed according to the guidelines in Agostini et al. (1989). Most gabion walls used in the project were the rear-stepped type with front batters (inward inclination) of 1:10. Stone masonry was designed with different types of front batters according to site conditions.

Retaining walls were designed and constructed at sites AH-1, AH-4, AH-8, and AH-2 by calculating the selected safety factor against overturning, base sliding, structural adequacy and overall stability. Attention was also paid to providing adequate drainage to ensure the safety and economy of structures. The LARIS-SM and RETAIN computer software were used whilst designing the walls.

Anchored Structures – Some of the instabilities identified by RFRP could not be stabilised using conventional stabilisation measures. Examples were the landslides at km 70.2, 73.5, and 81.8 (but not AH-1 or AH-4). These landslides were stabilised using anchored structures and rock dowels (RCC panels or RCC slices-cum-gabion walls and passive anchors with bar tendons). Stabilisation of some landslides was deemed too costly for the expected benefit and sophisticated measures were not proposed at such sites (e.g., the Kothe landslide at km 74.4).

The installation of in-situ anchors requires substantial skill and close supervision. This expertise exists in Nepal. With anchors, acceptance of the end product by testing is not sufficient to ensure the safety of the anchor. The whole process from the preparation of the anchors, through equipment preparation to drilling, grouting, placement, and fitting of the anchor needs careful supervision. A quality assurance strategy needs to be developed as an integral part of anchor installation.

Rock bolts with bar tendons are particularly useful where plane failures and wedge failures are likely to occur. Rock bolts join rock plates together to form a monolithic mosaic to minimise future rock failures. Rock bolts were used on the landslide at km 70.2. The installation of rock bolts includes drilling an inclined hole to the depth required with a horizontal plunge perpendicular to the strike of the bedding planes. The bolts impart additional shearing resistance between rock joints by increasing inter-planar normal force due to tension developed in the bolts and partly by the component of bar tension along the joint plane.

Earth/rock anchors have been used in Nepal's road sector only recently although they have been used frequently in hydropower tunnels. The first use of earth/rock anchors was in the Charnawati left bank where very active landslides in a colluvial slope severely endangered the Lamosangu-Jiri road at several places. Earth/rock anchors have since been used successfully along the Lamosangu-Jiri road, the Arniko Highway, Bhainse-Hetauda, and Thankot-Naubise roads.

The mechanism of load transfer from anchors relies on the bond at the soil-grout and the tendon-grout interfaces, rather than on soil-soil frictional resistance. The anchor transfers its tension to a fixed anchor zone. The length of earth/rock anchor in the fixed anchor zone varies depending on the degree of consolidation of soil material or the nature and degree of weathering of the bedrock. The anchors may also be under-reamed or expanded at the fixed anchor zone. Cylindrical anchors with non-pressure cement grouts are most commonly used in Nepal. The fixed anchor zone in rock may be between 3 and 6m, while that in soil is usually in the range of 6-9m.

Earth/rock anchors were applied at the km 70.2 landslide on the Arniko Highway to tie a composite retaining wall to weathered bed rock at a depth of 9-10m. Typical anchorage measures used for stabilisation of the landslide AH-2 are shown in Figure 17.22.

The most commonly used anchor tendons are continuously threaded MACALLOY or DYWIDAG anchor steel 32 mm ϕ with a minimum yield strength of 830–1030 N/mm^2. A working load of 250 kN on each anchor has been found suitable for the geotechnical conditions of landslides in the Sunkoshi Valley.

Surface treatment measures

The Arniko Highway corridor experiences heavy precipitation (>1,500 mm per year), which induces flash runoffs along the fragile slopes, creating rill, gully, splash, and sheet erosion. It was therefore necessary to design and apply surface protection measures on all natural and artificial bare slopes. Such measures consisted of bioengineering and skin protection measures. The long-term stability of the slopes in the landslides depends on the effectiveness of sub-surface drainage and bioengineering measures.

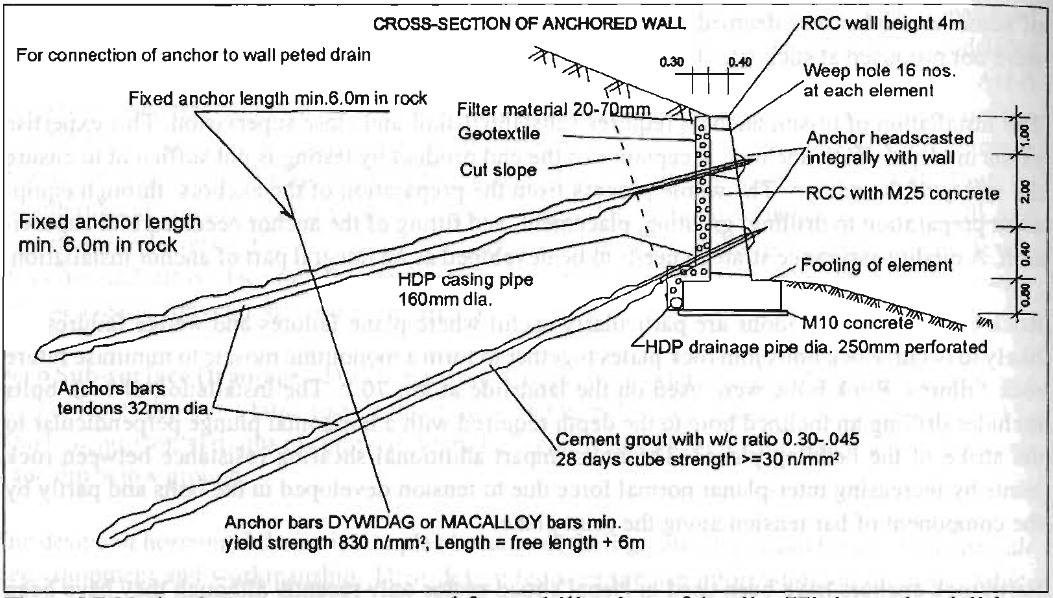


Figure 17.22: **Anchorage measures used for stabilisation of Arniko Highway landslide AH-2 (AHP 1997)**

Bioengineering — Bioengineering was used in RFRP to re-establish vegetation cover on bare landslides and artificial slopes and thus protect them against splash and sheet erosion. Bioengineering reduces rainfall erosion through intercepting rainfall and reducing evapotranspiration. However, these measures usually increase the infiltration rate and thus proper drainage of the root zone is essential. Plants increase the shearing resistance of soils at the root zone, but trees may increase the surcharge load and their presence can be counter-productive. The final choice of species was guided by both the need and the limited time available. The overall project duration was 18 months, which is generally insufficient for the successful application of bioengineering measures. Bioengineering is a long-term solution and relies for its success on repeat inputs and maintenance by the executing agency. The nine most common types of bioengineering measures are illustrated in Figure 17.23.

Both intensive and extensive treatments were applied for surface stabilisation.

Intensive treatment was applied generally on backfill slopes, spoil slopes towards the hillside of the road, and on most of the artificial slopes towards the valley side. The main emphasis was to establish a grass cover rapidly to protect the surface. The combinations of the following were used: terracing; mulching; planting grass; planting trees; stone arching; gabion-netting; edge rounding; spreading top soil; reseeding; and other complementary measures such as palisades, live stakes, and hedge layers.

Extensive measures were applied on less critical and less degraded soil slopes. The measures consisted of planting grass and trees, reseeding grass at lower densities, and making brushwood check dams.

The grass and tree species used for bioengineering were as follow.

- Tree species – utis (*Alnus nepalensis*), salix (*Salix tetrasperma*), poplar, sissoo (*Dalbergia sissoo*), kavro (*Ficus lacor)*, khirro (*Sepium insegne*), simali (*Vitex negundo*), badahar (*Artocarpus lakoocha*)

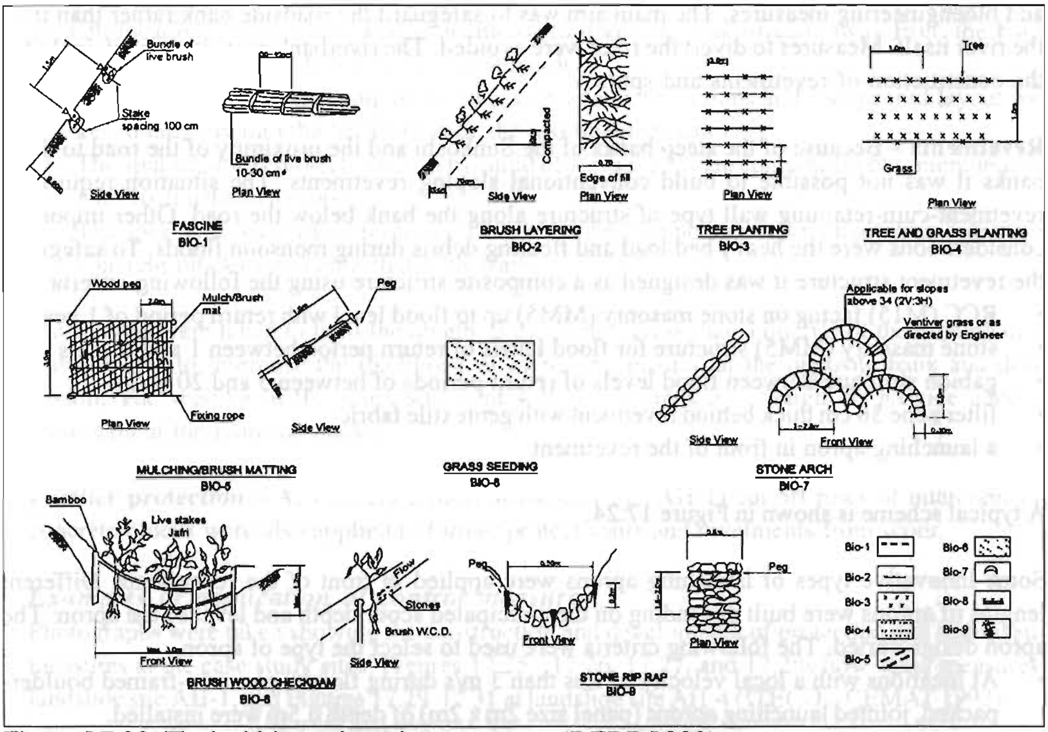


Figure 17.23: **Typical bioengineering measures** (RFRP 1990)

- Grass species – vetiver (*Vetiver zizanioides*), rye grass, paspalum, khar (*Cymbopogon microtheca*), kikiyu (*Pennisetum clandestinum*), kudzu (*Pueraria lobata*), clover (*Trifolium* spp.), Napier grass (*Pennisetum purpureum*)
- Mulch material - straw, banmara (*Eupatorium adenopherum*), titepati (*Artemesia vulgans*), and jute netting

Three nurseries were established along the road to produce tree seedlings, plants, and grass slips. Locally trained labourers were used to apply the bioengineering measures.

These bioengineering measures were the first to be applied along the Arniko highway corridor and there was little experience to build upon. Many of the plants did not establish well, but utis, sissoo, simali, vetiver grass, paspalum, and Napier grass all did well. The grass species kikiyu was highly successful at Charnawati at 1800m altitude but unsuccessful lower down at 600–1000m.

Stone Pitching — In addition to structural, drainage, and bioengineering measures, certain bare soil surfaces within landslides needed immediate surface strengthening to prevent sheet, rill, and gully erosion. In such areas, depending on the availability of supports, light superficial non-structural measures, such as thin gabion mattresses, stone pitching, stone arches, and riprap channels were applied. Such measures are cost effective and often unavoidable because bio-measures take a few years to establish. Such applications provide temporary surface protection, which are then reinforced by the bioengineering measures.

River training measures

The main triggering factor for the landslides was the flooding of the Sunkoshi River. Therefore stabilisation of the landslides required riverbank protection in addition to the drainage, structural, and bioengineering measures. The main aim was to safeguard the roadside bank rather than train the river itself. Measures to divert the river were avoided. The riverbank protection work included the construction of revetments and spurs.

Revetments – Because of the steep banks of the Sunkoshi and the proximity of the road to these banks it was not possible to build conventional sloping revetments. The situation required a revetment-cum-retaining wall type of structure along the bank below the road. Other important considerations were the heavy bed load and floating debris during monsoon floods. To safeguard the revetment structure it was designed as a composite structure using the following criteria.

- RCC (M15) facing on stone masonry (MM5) up to flood level with return period of 1 year
- stone masonry (MM5) structure for flood levels of return period between 1 and 5 years
- gabion structure between flood levels of return periods of between 5 and 20 years
- filter zone 30 cm thick behind revetment with geotextile fabric
- a launching apron in front of the revetment

A typical scheme is shown in Figure 17.24.

Some innovative types of launching aprons were applied in front of the revetments. Different lengths of aprons were built depending on the anticipated scour depth and level of the apron. The apron design varied. The following criteria were used to select the type of apron.

- At locations with a local velocity of less than 3 m/s during floods, concrete-framed boulder-packed, jointed launching aprons (panel size 2m x 2m) of depth 0.5m were installed.
- At locations with a local speed between 3 and 6 m/s during floods, interconnected 1.5 x 1.5 x 1m concrete blocks tied both ways with 16 mm cable were used.
- **At locations with an abundance of boulders and a local speed between 3 and 6 m/s during floods, interconnected armoured boulders (size >1m) were used.**

Spurs – Along certain stretches of the banks of the Sunkoshi, such as at AH-1 and AH-4, the banks were highly vulnerable and revetments alone could not ensure safety of the roadside bank.

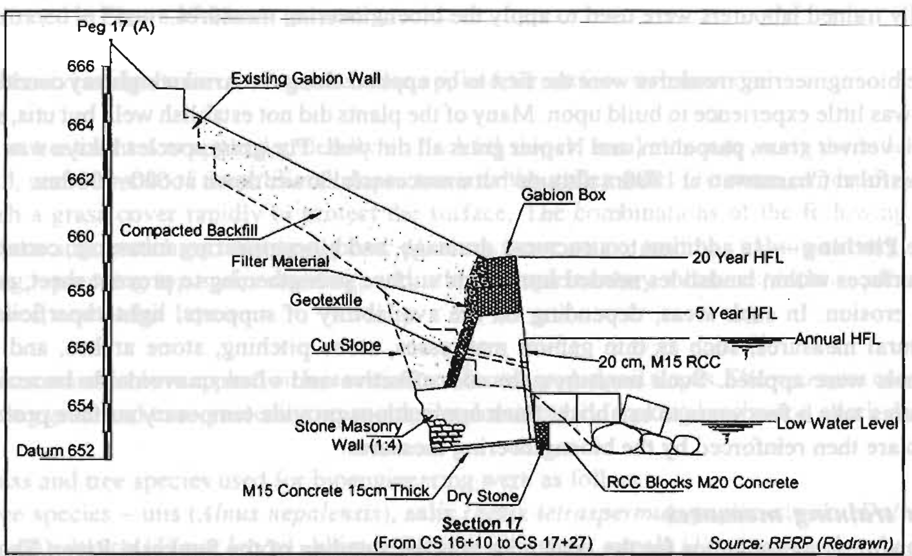


Figure 17.24: Typical river training measures (RFRP 1993)

In such locations, short spurs were constructed to divert the mainstream away from the banks. Composite spurs were installed as described below.

- The main body was made up of gabions with adequate sections and a staggered layout well-keyed laterally into the bank or inside the revetment structure.
- The spur head was protected with a pyramid of 1.5m x 1.5 x 1m concrete blocks interconnected with 16 mm diameter rope.
- The spur head was further protected from scour by a launching apron on all three sides using concrete blocks interconnected both ways.

Precautions were taken to limit the length of the spurs to less than a quarter of the breadth of the water course to minimise the likelihood of increasing erosion at the opposite bank and downstream. The spacing of the spurs was kept within 2.5 times their length to provide adequate protection at the roadside banks.

Further protection – At severely critical stretches (e.g., AH-1) cut off rows of interconnected concrete blocks were also applied to further protect spurs and revetments from scour.

Examples of application of control measures

Photographs were taken showing the construction and development of protection and stabilisation measures at the case study sites. Figures 17.25, 17.26, 17.27, and 17.28s how some measures at landslide site AH-1, and Figures 17.29-17.31 at landslide site AH-4 (ITECO – CEMAT 1992).

Figure 17.25: **Landslide AH-1: a complex network of landslide drainage**

Figure 17.26: **Landslide AH-1: an anchored gabion-retaining structure**

Figure 17.27: **Landslide AH-1: a revetment wall is being constructed with a concrete framed apron**

Figure 17.28: **Landslide AH-1: completed bank protection works**

Figure 17.29: **Landslide AH-4: landslide drainage and bank protection upstream from the landslide**

Figure 17.30: **Landslide AH-4: construction of revetment wall**

Figure 17.31: **Landslide AH-4: completed revetment wall**

Summary and Conclusions

Road planners, consultants and contractors in Nepal now realise the need for an interdisciplinary approach to investigating, designing, and building stabilisation and mitigation measures for landslides along highways. The necessary expertise includes engineering geology, hydrology, geophysics, geotechnics, highway engineering, bioengineering, river engineering, and soil engineering.

The drainage of landslides, through installing networks of surface, near surface, and deep horizontal drains, was used most in the RFRP. Structural measures were given second priority and only applied where drainage alone was considered insufficient to improve slope stability to an acceptable level of safety. Bioengineering measures were considered as essential components of all stabilisation systems and were applied to all landslides. These measures were highly successful on active and moderately active landslides. The performance of French drains and deep-drilled drains was satisfactory as shown by monitoring with piezometers.

The total cost of stabilisation and mitigation of the nine medium-size and medium complex landslides was about NR 37 million (approx. US$ 0.55 million). The average cost per landslide was around NR 4 million (US$ 60,000). This cost is comparable to the cost of maintaining a standby bulldozer for five years for emergency reopening of the road during the monsoon period. Proper stabilisation means improved serviceability of the road with far fewer blockages during the monsoon and alone justifies this expenditure. The methods of stabilisation described here appear to be practical and affordable.

The efficient management of these sites now relies on carrying out routine maintenance, but as yet no efficient and appropriate system for this has been developed. Efforts are being made by the Geo-environmental Unit of the DoR to prepare guidelines for installation of further bioengineering measures and maintenance of that already in place through community participation. In the absence of maintenance, the works at the landslides along the Arniko Highway have already started to degrade.

Lessons learned

Technical and other lessons learned from the planning, investigation, design, implementation, and maintenance of the mitigation measures under RFRP (ITECO –CEMAT, 1993) are summarised below.

- The project had to complete the design of rehabilitation measures along a 25 km stretch of road, including survey and all investigations, within four months. However, certain types of investigation require dry months while others must take place in the monsoon period. A period of six to eight months is needed to enable consultants to plan for and implement works during the most appropriate season.
- A large number of gabion structures lacking foundations and lateral embedment had been applied at most of the sites during the pre-project period (1987-1992). The inclusion of these would have weakened new structures. It is recommended that short-term measures should be limited to reopening roads by clearing the blockage. The use of massive spurs and revetments as emergency measures should be discouraged.
- Using Lacey's general scour equation and scour factors (Z-factors), the scour depth along several stretches of the Sunkoshi was found to be in the range of 6-8m. However, the practical limit for dewatering was found to be only 1.5m below low water level. The founding of structures below the scour depth is not viable in mountain rivers and the safety of bank structures depends on the construction of launching aprons.
- The specifications and items of bioengineering measures should be precisely given to allow contractors to carry out such work. Bioengineering measures are usually started after the completion of structures and drains. Unlike conventional structures, bioengineering measures need intensive maintenance over at least three years. One way of carrying this out is through the participation of local communities.
- The innovative idea of using inter-connected concrete blocks was unsuccessful at the most vulnerable location at Balephi (AH-4), where the blocks from the upper three layers of spur were washed out by a moderate flood of about a 10-year return period. This experience should be taken into account when building such structures at vulnerable locations.
- The deflection of gabion walls was excessive due to high flexibility. For future projects it is recommended that the front batters for rear stepped gabion walls should be in the range of 1:6 to 1:4 instead of 1:10.
- The main reason for the instability and damage at sites AH-1, AH-2, AH-5, AH-8, and AH-10 was insufficient drainage. This emphasises the importance of drainage for slope stability.
- Most of the landslides along the Arniko Highway are triggered by toe erosion and propagate a couple of hundred metres above the Sunkoshi riverbed. The Hazard Mitigation in Sunkoshi/Bhotekoshi Water Catchment Areas (HMWA) project has substantiated this finding. Had the alignment of the Arniko Highway been at least 200m above the riverbed, most of the hazards related to the river would have been eliminated. This emphasises the need for proper selection of highway alignment along hazardous river valleys such as the Sunkoshi.

Mitigation planning

The experience of planning and implementing rehabilitation works for landslides along the Arniko Highway emphasises the need for an integrated approach to landslide hazard management and control. The problems encountered in the Sunkoshi valley are typical of those likely to be encountered in other areas of Nepal and the lessons learnt here can be applied overall. In addition to engineering approaches, social and environmental factors must be considered to make the work socially acceptable and long lasting. Landslide management should be a continuous process in geologically fragile and hazardous mountain regions like the Himalayas. Over-simplified approaches to landslide control and management can lead to conflicts of interest with the local population.

Most of the landslides along the Arniko corridor were caused by hydrological, geo-hydrological and land use factors. More than two-thirds of the landslides were activated due to a lack of drainage. Pressure on marginal lands due to population explosion has further deteriorated slope stability.

The main recommendations for a more rational approach to landslide hazard management and control are as follow.

- Prepare an inventory of all glacial lakes within a catchment area and assess and monitor their condition
- Prepare 1:10,000 scale topographical, hazard and land use maps for all important river valleys that are susceptible to hydrological disasters
- Monitor landslides and potentially unstable areas to warn of impending dangers
- Incorporate risk engineering in civil engineering and geology courses
- Train engineers and geologists how to investigate landslides, and design stabilisation works
- Encourage participation of the local population in landslide stabilisation and bioengineering works
- Introduce legislation to limit the use of high hazard condition areas
- Carry out further research on landslide management
- Improve the level of disaster preparedness in sensitive corridors; emergency measures should be limited to the bare minimum necessary to reopen the infrastructure and should not be applied indiscriminately where long term definitive solutions are envisaged.
- Establish an institution to deal with landslide hazards management and mitigation works
- Equip executing agencies with modern and efficient techniques for slope stabilisation
- Establish regional information networks for landslide hazard management

It may be difficult for planners to implement this long list of activities; but, as suggested from the efficient operation of the Lamosangu-Jiri road where the above measures were largely taken, the extra costs can nearly always be justified.

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