Flood Disaster Rehabilitation, Charnawati, Nepal: A Case Study

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After a brief mention of the principles of design applied to the construction of the Lamosangu-Jiri Road in the central region of Nepal, the authors describe the effort undertaken to rehabilitate a road section heavily affected by a flood in the early monsoon season of 1987. Innovative techniques for the construction and maintenance of roads in the Himalayan region, such as anchoring, drilled subsurface drains, and flexible river protection, were introduced. In view of design options for low-volume roads in hilly terrain, practical suggestions such as low initial investment and event-related rehabilitation are discussed.

Extreme difficulty of access is one of the dominant characteristics of mountain communities and is a formidable constraint to the effective implementation of essential programs of rural development. This is the case throughout the hills and high mountain districts of Nepal. For this country, situated between India and China and dominated by the Himalayan range, road transportation is vital for the development of industries and commercial agriculture because other types of transportation (railway, ropeway, or air) are only complementary to road travel and do not contribute substantially to the transport of goods or passengers. The present road network in Nepal consists of approximately 7000 km, resulting in a density of 1 km per 21 km^2 (*1*). Most of these are gravel or dirt roads, and a large number of them can only be used in fair weather. Construction of all-weather roads through the mountains to improve access to remote areas is extremely costly and technologically challenging, particularly on steep and unstable slopes or across mountain rivers. Most road failures occur during the monsoon season (July to September) because of floods or slides, which sometimes are triggered by earthquakes.

LOCATION OF THE ROAD AND PROJECT AREA

The road considered here is situated in the central region of Nepal. It starts from Lamosangu at the Arniko Highway (linking Nepal to China) and after 110 km, reaches Jiri, the district headquarters and starting point for porter and trekking services to central and eastern Nepal (Figure 1). The road was constructed between 1975 and 1985 with contributions from the Swiss Development Cooperation.

The area requiring extensive stabilization work, described later, is located approximately midway (Km 45), at an elevation of 1800 m where the road crosses the Charnawati River.

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ROAD STANDARDS

The Lamosangu-Jiri Road was constructed to a single-lane standard with an overall width of 5.4 to 6.1 m and bituminous surfacing (*2,3*). The design speed was 30 km/hr, and the maximum longitudinal gradient was limited to 12 percent. It was designed considering local conditions as much as possible, and the following guidelines for implementation were followed:

- Close cooperation with local administration,
- Consideration of ecological constraints in order to prevent adverse environmental impacts and minimize maintenance costs,
- Use of labor-intensive methods to create job opportunities for the local population and maximize the financial input into the region,
- Training of local people, and
- Close cooperation with other development projects in the region.

In summary, the road was constructed with a low-cost approach and labor-intensive methods. The use of equipment was reduced to the lowest level that ensured acceptable quality and rate of progress. This put a considerable constraint on the applicable engineering methods to cope with the frequent problems of floods and slides. All main structures were executed in gabions. Despite the advantages of their flexibility and excellent drainage capabilities, the reasonable height of retaining walls or dams constructed with gabions was, in general, limited to approximately 10 m. In addition, gabion wires exposed to tractive forces due to soil movement or to abrasion in rivers represent problems for maintenance.

Although the estimated (and actual) traffic was rather low, the road base was designed with a bituminous surface course (either a 5-cm penetration macadam or a 3.5-cm premixed asphalt layer using bitumen emulsion) on a width of 2.9 m. This solution was chosen mainly because of the steep gradients of the road in the hilly region and to improve drainage of the road body during monsoon season. The somewhat higher cost of a blacktopped road is more than balanced by easier maintenance of the road surface.

TRAFFIC

The Lamosangu-Jiri Road is a Class II feeder road according to Nepal Road Standards (*4*). Traffic forecasts used for road design in 1975 were of the order of 10 trucks per day in each direction, with a 5 percent increase per year. Observations

FIGURE 1 Location map with main road network of Nepal.

during the construction phase of the Charnawati Rehabilitation Project showed the following average traffic: 7 buses per day in each direction and 10 trucks per day in each direction. Maximum axle loads of trucks were estimated to be 13 tons, according to an axle survey conducted on Nepalese roads in 1983 (*5*).

FLOOD DISASTER AND CONSECUTIVE SLIDES

The Charnawati River has a gradient of 12 percent as it crosses the road. Gradients up to 25 percent can be observed further downstream. Its catchment upstream of the road crossing covers an area of 13 km^2 and is almost semicircular in shape. It is confined by very steep cliffs with a difference in elevation of approximately 1000 m. This morphology is responsible for the microclimate in this area, provoking very heavy monsoon rains in a limited small area. Rainfall and discharge measurements at tributaries of the Charnawati River showed that rainfall intensities vary considerably even within 1 km. The heavy rains often trigger mud and debris flows in the tributaries, disturbing the main flow of the Charnawati.

During the night on June 30, 1987, a heavy flood of the Charnawati River destroyed the original 10-m-span concrete bridge on the road and triggered slides all along its banks, particularly disastrous to the road for a length of approximately 1 km on the left-bank bridge approach. The river partially created a new bed in a process of erosion and sedimentation. Riverbed erosion to a depth of 5 m (locally even 10 m) and recent local sediment deposits up to a thickness of 3 m could be observed after the flood.

Hydrological estimates based on flood traces revealed that the discharge during the flood event was approximately 150 m^3/sec. A hydrological study using regression analysis and run-off formulas concluded in the following maximum discharge estimates: Q_2 = 40 m^3/sec, Q_{10} = 95 m^3/sec, Q_{25} = 120 m^3/sec, Q_{50} = 140 m^3/sec, and Q_{100} = 160 m^3/sec.

PROJECT HISTORY

At the end of the monsoon season in 1987, studies for the rehabilitation of the Charnawati crossing were undertaken and implementation of the Charnawati Rehabilitation Project (Phase I) started in January 1988. Due to time constraints, construction had to be limited to a road stretch of 300 m on both sides of the river crossing.

Simultaneously with the implementation of the first phase of the project, realignment alternatives were studied to bypass the critical section. It was found that realignments were either too long and thus too costly or passed through highly unstable areas. Thus the problems would have been shifted rather than avoided. A conclusion was therefore reached to maintain the alignment by trying to stabilize the slides at and below the road.

The target of keeping the road open during monsoon season in 1988 was achieved. However, four slides triggered by the 1987 flood, which were 100 m below the road, developed with unexpected speed during monsoon season 1988 and two of them reached the road by the end of it. This development called for substantial additional stabilization measures to save the road and the stabilizing structures, which constituted the primary investment. A second phase of the project was thus implemented from November 1988 to June 1989.

At the time of this writing, stabilization work was still in progress (Phase III) and is expected to be complete by the end of 1991.

Figure 2 shows an overview of the project area with the Charnawati crossing in the foreground.

GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

The Charnawati area is located in the lesser Himalaya, in the so-called midhills that are incised by many rivers and are continuously reshaped by active erosion processes.

The whole region between Km 42 and 47 of the road has been evaluated by hazard mapping. The technique of soil hazard mapping has been developed based on the principles of rock and debris slide risk mapping (6,7) adapted to slide hazards in soil material. By this method a weighting factor is attributed to all relevant parameters governing the soil stability, such as

- Slope angle and type of soil (residual, colluvial, alluvial, or glacial),
- Hydrological and hydrogeological condition (perennial-seasonal streams, springs, water table),
- Hydrodynamic condition (floods, glacial lake outburst, slide dam breakthrough),
- Past and present slide and erosion activity,
- Seismic activity and presence of faults, and
- Land use (vegetation, forest, irrigation).

The combination of all parameters results in a soil hazard map (Figure 3) showing areas of low, moderate, and high instability. This map is used to design possible realignment alternatives and to define the locations where further detailed investigations are necessary. The map clearly shows the high-hazard area on the left bank of the Charnawati River (Km 45 to 46 and Km 47).

In this area the rock is predominantly gneissic and covered by a thick soil layer at most locations. The top soil layers generally consist of colluvium with a silty-sandy matrix and gneissic boulders (2 to 5 percent). Slopes are rather steep with typical slope angles between 30 and 35 degrees.

Detailed geophysical investigations, exploratory drilling, piezometers, and soil tests revealed the following subsurface conditions:

- The internal angle of friction of the noncohesive colluvial matrix is 33 degrees (consolidated undrained triaxial shear tests with measurement of pore-water pressure), classified as SM-ML with very low plasticity;
- The thickness of the soil cover above the bedrock in the critical (sliding) areas is greater than 20 m (resistivity measurements);
- The water table is situated at a depth between 3 and 9 m during dry season and is partially rising to the surface during monsoon;
- Seismic refraction tests showed a distinct change in wave velocity at a depth of between 3 and 7 m; this was attributed to the boundary between upper loose (sliding) and lower more compacted soil material.

FIGURE 2 Aerial photograph of project area, looking south; Charnawati crossing in the foreground.

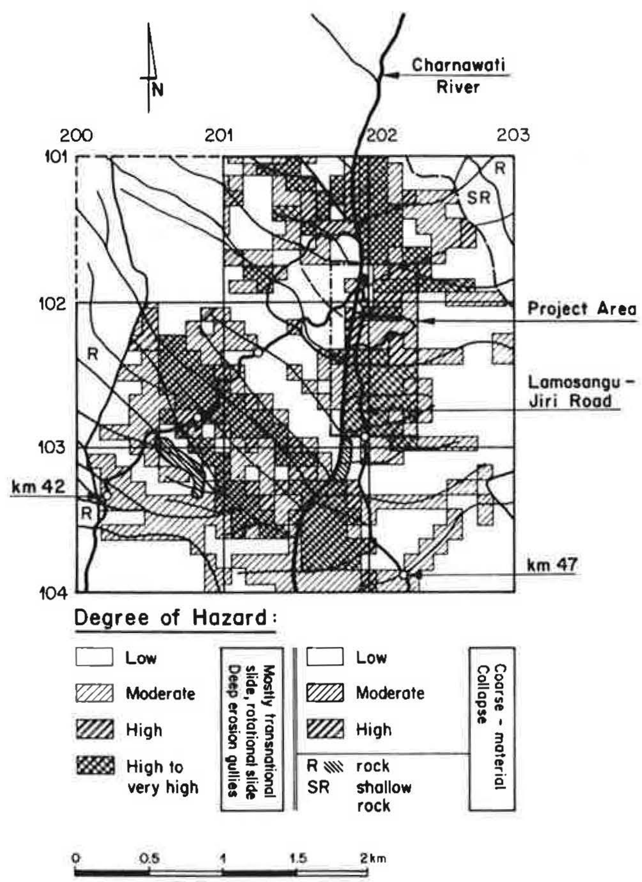

FIGURE 3 Charnawati Valley soil hazard map.

The latter investigations and stability computations (method of Janbu and Bishop), confirmed later by extensometers (see the next section), revealed that the slip surfaces controlling the slides are at a depth of between 3 and 7 m.

REHABILITATION WORK

Construction work during the project can readily be divided into four different groups:

1. River control work to stabilize the toe of the slides,
2. Drainage in and above the slides,
3. Slide stabilization work, and
4. Road rehabilitation.

Figure 4 shows a plan of the project area indicating the extent of the slides and the main structures.

River Control Work

The Charnawati River bed was stabilized and partially lifted for a length of 450 m by means of nine check dams, constructed using gabions with all exposed parts (crest, etc.) covered with concrete to avoid wire abrasion problems. The geometry and gradient of the riverbed called for a dam width of 13 to 23 m and a height of 4 to 6 m. The horizontal spacing of dams was between 30 and 50 m. Since all excavation work in the riverbed was undertaken manually, the foundation depth was limited to 1.0 to 1.5 m. To avoid scouring problems downstream of the dams, the aprons were reinforced by concrete slabs with a cover of stone slabs. Generally, the river banks between the dams were protected by gabion guide walls, thus buttressing the toe of the endangered slopes. Figure 5 shows the layout of the check dams in the vicinity of the river crossing.

Stabilizing the river bed with check dams represents a classical technique, although the size of the dams constructed can be considered substantial for Nepalese conditions. Since the series of check dams is retaining most of the bed load of the river, scouring will be pronounced at the last downstream dam. Construction of a stilling basin with controlled dissipation of energy, in which excavations to a depth of more than 5 m would have been necessary, was found not to be feasible because of the high gradient and the labor-intensive construction methods involved. Two alternatives were discussed to solve the problem: (a) continuation of the series of check dams downstream until a flatter area is reached, necessitating the construction of up to 40 more check dams of similar shape, and (b) reinforcement of the riverbed by concrete blocks (so-called flexible river protection). The latter alternative was found to be more attractive and less costly because protection work could be confined to the critical area downstream of the check dams.

The principle of flexible river protection is to make use of the existing boulders in the riverbed and to selectively reinforce them with concrete blocks. The concrete elements are shaped in a three-dimensional H-form similar to the elements used for breakwater protection. This allows better interlocking of the elements and still permits the low water to flow between them.

The initial layout of the concrete elements is of great importance and was extensively tested in hydraulic models. Most of the elements were placed at the riverside and on the banks (to act as stabilizers if the bank is undercut). In addition, the

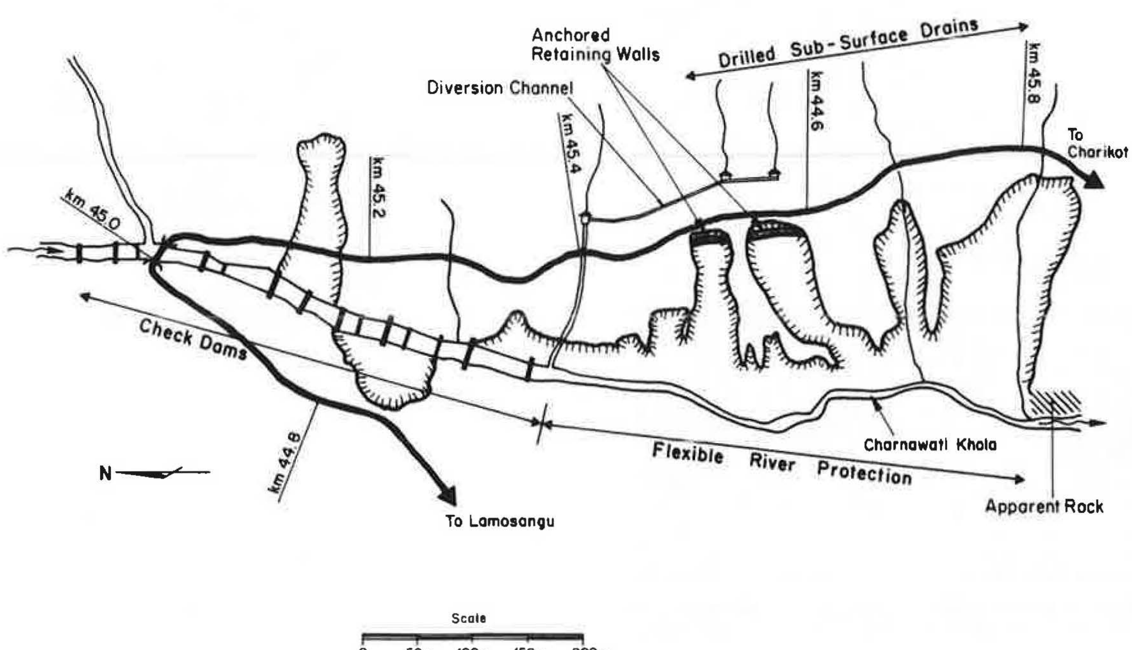

FIGURE 4 Charnawati Project: overall plan view. Only the main substantial structures are presented; drainage and biotechnical measures in the slides are not shown.

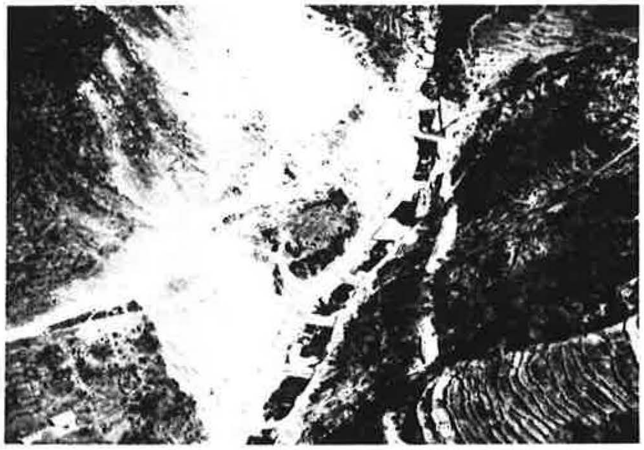

FIGURE 5 Aerial photograph of check dams and road crossing. Note the slide on the right-hand bank where the alignment had to be shifted to the mountainside. The cleared area to the left of the bridge was used for quarrying.

accumulation of concrete elements forming a sort of spur or sill proved to be efficient. On the average, the Charnawati River bed has a 90 percent fraction of boulders of 1 m in diameter, although some rare big blocks of up to 10 m in diameter can be observed. In view of the above parameters, the height and width of the concrete elements was chosen as 2.5 m, resulting in a weight of 10.5 tons per element. Upon completion of the project, 700 elements will have been placed in the riverbed over a length of 500 m.

Considerable effort was invested in construction planning. The concrete elements were cast in situ (in the riverbed) using steel formwork composed of manually portable pieces. Concrete from the batching plant was transported by cable crane to the river site. This technique allowed an overall construction progress of three elements per day.

Drainage

As is often the case, water (below and at the surface) was the key factor for most slides at the Charnawati banks. It was

FIGURE 6 Typical causeway for smaller rivulets.

FIGURE 7 Stabilized slide at Charnawati left bank, Km 45.1. Two main drains crossing the road are supported by the gabion guidewall at the river.

observed that the silty soil flowed like mud once it was completely saturated and the protective vegetation cover was destroyed. Access to these areas was impossible during monsoon time.

In principle, an attempt was made to control all surface water in the sliding area. A typical causeway with gabion protection, as used for cross drainage before the slides at Km 45.4 to 45.6, is shown in Figure 6. Surface water of two rivulets was contained above the road and diverted to a rivulet in a more stable area. All slide areas were drained to a depth of 2 m by gabion drains located in topographical depressions and aligned in the direction of the slope gradient, thus evacuating water by the most direct route. Tributary drains at intervals of 15 m were designed to collect water and guide it to the gabion drains. A successful application of this system can be seen in Figure 7, showing a stabilized slide with two main gabion drains crossing the road.

Stability computations (see previous section) showed that at the steep slopes between Km 45.4 and 46.0, drainage to a depth of 2 to 3 m is not sufficient to achieve reasonable safety factors, even in combination with retaining structures. It was therefore decided to implement horizontally drilled subsurface drains, which allow drainage to much greater depths. Although application of this technique, which is new in Nepal, necessitated a considerable investment in drilling equipment

FIGURE 8 General view of slides, Km 45.4 to 46.0. Drainage and biotechnical measures can be seen in the upper parts of the slides.

and operator training, the system proved to be the most effective in raising the safety factor to an acceptable level. A factor of 1.2 to 1.3 was considered reasonable under local conditions, because the achievement of higher safety factors would have involved prohibitive costs. The drilled subsurface drains are 25 to 35 m long to reach a depth of 5 to 6 m. They are arranged in fans so that up to 10 holes can be drilled from one spot, thus keeping the water table below 4 m even during monsoon time. The drains consist of perforated pipes (63-mm diameter) made of polyethylene and sheathed with geotextile.

Stabilization Work

Figure 8 is a general view showing the status of the upper parts of the four slides from Km 45.4 to 46.0 in February 1990. It shows that parts of the slides could be stabilized but that further work is necessary. At the upper edge of the two main slides in the foreground, anchored retaining walls were constructed just below the road to prevent further upward development of the slides and to retain the road. The layout of the walls and drainage efforts are shown in Figure 9.

Vertical series of four earth anchors were arranged at intervals of 4.3 m. The anchors consist of steel rods St 500/600 with a diameter of 32 mm and a length of 25 m (upper two anchors) and 20 m (lower two anchors). Anchor forces are built up by friction and dilatation effects between the grouted anchor hole and the soil. In the stability computations the permissible anchor forces were 250 kN per anchor. Pullout tests showed that values of 400 kN could be reached if boulders were hit during drilling, thus expanding the anchor root.

Since such anchoring techniques have not been applied in Nepal before, care was taken to monitor the behavior of the structures by means of extensometers, measuring deformations in the soil. The anchors are not prestressed; consequently the full anchor forces are built up only after deformations of approximately 1.5 cm. In order to have a structure compatible with these deformation requirements and to save some (expensive) concrete work, the wall was executed as a gabion wall with intermittent concrete blocks anchored back.

To complement the structures, all bare surfaces of the slides were treated with biotechnical measures to avoid water infiltration at the surface and to stabilize the slide surface. The techniques applied were

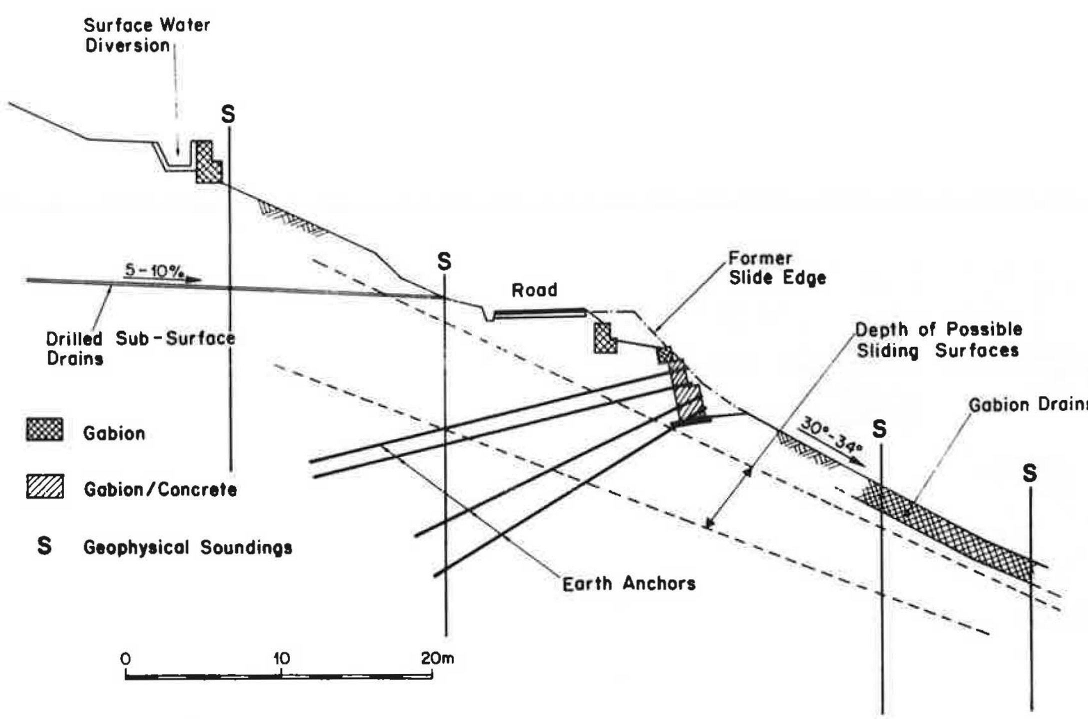

FIGURE 9 Cross section of slide stabilization measures at Km 45.4.

- Terracing of all slopes,
- Mulching of surface with branches of bushes that sprout during rainy season fixed with wire nets at steep slopes,
- Reinforcement of the top soil layer with stone arches (dry masonry layers arranged in arches to a depth of approximately 30 cm) and hedge brush layers,
- Sowing of grass, and
- Planting of locally growing deep-rooting trees

For these measures a nursery was maintained throughout the construction period. Successful applications of these techniques which are also aesthetically satisfying, are shown in Figure 7 and 8 (upper parts of these views).

Road Rehabilitation

Approximately 600 m of the Lamosangu-Jiri Road had to be reconstructed and the centerline needed to be shifted to the mountainside above the slides. On the basis of the traffic assumptions outlined earlier, the pavement was designed with 25 cm of untreated crushed aggregates as the subbase and 15 cm of water-bound macadam as the base course. An additional blacktop (penetration macadam) will be applied after completion of construction work.

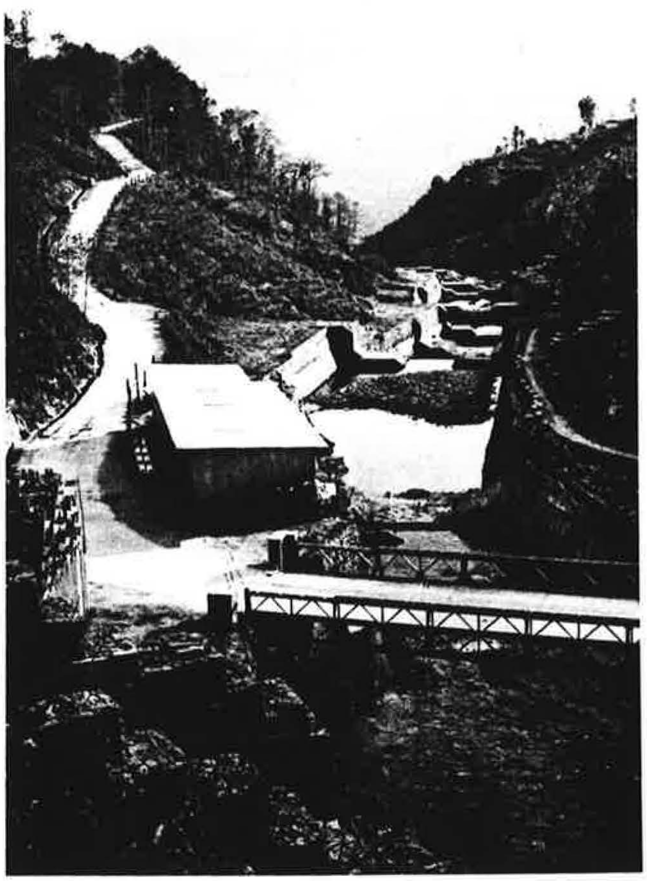

FIGURE 10 New Charnawati bridge crossing. The store in the foreground is used for current construction work.

Soon after the disaster the 10-m-span concrete bridge that was washed away was replaced by a Bailey bridge of 21.3-m span, which can be erected in short time (Figure 10). This bridge was initially meant as a semipermanent solution, but is likely to become permanent.

Summary of Rehabilitation Work and Cost

In general, structures for stabilization of the area were restricted to the minimum necessary to save the road. Prime importance was given to drainage, which resulted in the highest stability improvement at lowest costs. However, retaining walls to complement the drainage system and the stabilization of the riverbed were necessary to achieve long-term stability. Despite the rather high investment (see below), the safety factors achieved are not substantially above 1.0 and many parts of the slides, for example, the lower parts of the slides between Km 45.4 and 46.0, have been treated only minimally. Part of the stabilization work was left to nature in the sense that the sliding process was allowed to continue in areas less critical for the road until a new equilibrium was found. Only then were supporting measures, for example, reinforcing the toe of the slide, undertaken.

It must be emphasized that, despite the involvement of special equipment for the innovative construction techniques described above, the main structures were built with labor-intensive methods. For example, all earthwork was executed manually. Because of the time constraints implied by the emergency character of the project, the concentration of laborers was considerable, with a maximum of 1,000 people working in a very restricted area. Work was carried out by piecework contractors engaged directly by the consultant. Each of these had a labor force of approximately 50 local people. The system proved to be quite efficient in view of construction progress and cost. It is discussed further in the next section.

Although the final cost figures cannot be given at the present time, it is estimated that the overall construction cost (including design, supervision, and administrative costs) for the rehabilitation of the road stretch from Km 44.6 to Km 46.0 will amount to $5 million (U.S.). Initial construction costs of the road, updated to comparable values with average inflation rates of Nepal, were of the order of $0.55 million (U.S.) per kilometer. Without the investment of the Charnawati Rehabilitation Project, 65 km of road toward Jiri would have been degraded to a fair-weather road and the Jiri area would have been cut off for at least 5 months during the monsoon period.

DESIGN OF HILLY ROADS IN UNSTABLE AREAS

It is clear that because of the active erosion processes in the Himalayan hills and mountains, disasters like the one at Charnawati will happen again and can only be limited but not avoided by a prudent choice of road alignment. The problem is enhanced if a low-cost approach is used for the road design and construction, minimizing structures and initial investments at critical locations. Actually, the two principles of low-cost approach and minimum maintenance demand are in conflict: it has been confirmed that long-term preservation of

hilly roads can only be ensured by a substantial maintenance effort. Thus, the need to consider event-related rehabilitation methods from the beginning is imperative.

Initial Design

Extreme care must be taken when the alignment is designed for a low-cost road in terrain such as that described above. Use of risk-hazard mapping and engineering as described by Wagner et al. (*6,7*) and Fookes et al. (*8*) should be compulsory for alignment design. These techniques allow the recognition of possible trouble spots with respect to floods or slides to elaborate the proper strategy to cope with the problem and to correctly choose among the possibilities between locally high initial investment with lower maintenance or low initial investment with higher periodic and event-related maintenance.

The Lamosangu-Jiri Road was constructed with a low-cost approach, implicitly accepting the risk of damage by unusual events and leaving the task of rehabilitation to event-related maintenance. The Charnawati case showed that this event-related maintenance can be very expensive, since the rehabilitation costs for 1.4 km equaled approximately the initial construction cost of 10 km of road.

The decision on the initial approach to be taken depends not only on the funds and technology available, but also on the estimated performance of the maintenance organization. Therefore it could easily be the case that higher initial investment would be justified if the maintenance organization were not able to achieve the targets necessary to safeguard the road or if the funds available did not allow high maintenance costs. Although labor-intensive construction techniques can be very efficient and successful, it is evident that major problems of slides or river scouring are often beyond the possibilities of these techniques and call for modern methods with or without the involvement of substantial equipment.

Without giving a final recommendation (which is not possible), either the trouble spots should be treated with full commitment to the best possible solution at the initial stage or the initial investment should be kept at a low level and, most important, authorities should be made aware of the need of further event-related maintenance.

Emergency Maintenance

Emergency maintenance is defined as the maintenance operations necessary to keep the road open with acceptable serviceability after unusual events (slides, floods, accidents). As for the initial design and construction, options are open from low-cost solutions (e.g., clearing by bulldozer after slides) to higher investment as for the Charnawati Rehabilitation Project. Again, the choice of approach has a strong influence on the future maintenance demand.

Regarding the organization of the emergency work, it has been found that the approach of overall project management by a consultant with direct labor contracts is advantageous if construction time is short. A drawback of this approach is that it required the bypassing of the normal tendering procedure. Experience showed that the tendering system in Nepal is not efficient enough to cope with serious time constraints. It is thought that these considerations for the design of roads in unstable hilly areas are applicable not only to Nepal but to many other countries developing their road network under financial constraints in geologically unstable areas.

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