ABSTRACT

The Laprak landslide is a complex consisting of translational and rotational landslides, debris flows, and rockslides triggered by exceptionally heavy rainfall in July 1999, with continuing movement during subsequent monsoonal wet seasons. The active portion of the complex is a structurally constrained wedge of colluvium with a triangular or trapezoidal cross section, the geometry of which is controlled by two or three sets of foliation and joint planes, derived from phyllitic and quartzose metamorphic bedrock of the High Himalayan Crystalline Sequence. Stability analyses, constrained by field observations, limited geotechnical testing, and finite element simulation of steady-state seepage, show that the landslide most likely moves as three sections, with the middle section being more stable than the upper and lower sections. Non-slope-parallel seepage simulated by the finite element model appears to locally decrease instability by allowing drainage into the discontinuous bedrock. The stability analyses also suggest that the landslide will continue to move during the wet season when pore-water pressures are moderately high (although complete saturation of the slope is not required). Moderate earthquakes would exacerbate wet season instability but do not appear sufficient to trigger movement during the dry season, when pore-water pressures are low. The remote location of Laprak, which is a two- to three-day walk from the nearest road-head, and lack of money limit remedial options to those that can be undertaken using manual labor and native materials, for example, replacement of dry stone masonry buildings with lightweight wooden structures and surface drainage improvement.

INTRODUCTION

The village of Laprak, which is home to about 3,500 people, is located at 28°13’4.8"N, 84°48’12.7"E, at about 2,150 m elevation along the Raizo Khola valley in the Gorkha District of western Nepal (Figure 1). There is no road access, and a two- to three-day trek, depending on weather conditions, through mountainous terrain is required to reach the village. On July 3, 1999, heavy rain in excess of 340 mm in 24 hours, much of it apparently occurring during about 3 hours, triggered a series of rockslides and debris flows that led to the reactivation of part of an existing landslide of unknown age and upon which the village is built. One woman was killed, ten homes were destroyed, and about 12 hectares of cultivated land were taken out of production along the banks of the adjacent valley during the initial episode of movement. The landslide has since moved during each summer monsoon season, with more damaging movement in 2002, 2006, and 2007, destroying an additional 14 homes and taking an additional 11 hectares of cultivated land out of production (Figure 2). Because most buildings in Laprak are constructed with unreinforced dry stone, and many have already been damaged by landslide movement, seismic shaking with or without additional landslide movement poses an added threat to the village. The remote location of the village, lack of road access,
reliance on manual labor, and severe budget limitations all place significant constraints on mitigation options.

**Purpose and Scope**

This paper summarizes the engineering geology and stability of the portions of the Laprak landslide that began moving in 1999 and that continue to move episodically during yearly monsoons. Our analysis—undertaken to both understand the initiation of the 1999 landslide and elucidate conditions that may trigger future movement—is based upon a series of reconnaissance visits to Laprak, topographic and engineering mapping of features related to the landslide, digital terrain modeling to visualize the geomorphology of the landslide and bedrock controls on colluvium accumulation, a limited program of laboratory soil testing, and slope stability analysis of several static and seismic scenarios with and without contributions from bedrock groundwater flow.

**Initial Movement**

The Laprak landslide began on the morning of July 3, 1999. While walking at about 6:30 am, Mr. Dambar Gurung, the village peon or local administrator, noticed newly deposited boulders below quartzite cliffs at the head of Chhelong Gully, which forms the northwestern lateral boundary of the current landslide. He also witnessed small rockfalls or rockslides at that time and estimated that the main debris mass had been deposited around 4:00 to 5:00 am. After a period of increasing rainfall intensity, debris began to flow down Chhelong Gully at about 7:30 am, sweeping away homes built along the gully edge and existing check dams within the gully. Because no pre-flow topographic maps exist, it is impossible to determine the amount of debris-flow scour and/or deposition along the gully. Movement of the main landslide mass is likely to have been contemporaneous with or subsequent to the large debris flow. As will be discussed, small-scale quarry-
ing of the quartzite near the gully head may have also contributed to the initial movement.

The nearest rain gauge is in the village of Barpak, a half-day walk over a 3,000-m-high ridge from Laprak. The 1999 movement occurred in response to extreme short-term rainfall of 342 mm in 24 hours and 10-day antecedent rainfall of 235 mm at Barpak. According to residents of Laprak, most of the rainfall that triggered the 1999 landslide occurred during the three hours before movement began. Based on anecdotal accounts of the rainfall duration, the 1999 storm appears to plot near the landslide-triggering intensity-duration threshold developed for the Nepal Himalaya by Dahal and Hasegawa (2008) and well above the sediment-producing threshold developed for the Annapurna region of Nepal by Gabet et al. (2004). Although damaging movement of the landslide in both 1999 and later in 2007 was associated with extreme rainfall events, other damaging episodes in 2002 and 2006 were associated with average to below average rainfall. Less significant movement appears to occur almost every year during the summer monsoon season.

Geologic Setting

Laprak lies above the Main Central Thrust (MCT), which is one of three major thrust faults running the length of the Himalayas. The MCT defines the High Himalayan Front and brings predominantly Precambrian metamorphic rocks of the High Himalayan Crystalline Sequence to the surface (Figure 3). Bedrock in the vicinity of the village, which is exposed locally along riverbanks and ridges, consists of garnet mica schist, calcite schist, and quartzite. Instrumental records (Pandey et al., 1999) show a band of seismicity coincident with the High Himalayan Front defined by the MCT, and several segments are inferred to be capable of producing earthquakes similar in size to the 1934 M = 8.1 Nepal-Bihar earthquake. Lavé et al. (2005) described paleo-seismic evidence for an even larger Mw = 8.8 earthquake in Nepal sometime around 1100 A.D.

The Laprak landslide is located on a steep colluvium-mantled slope with strong bedrock structural control (Figure 4). Foliation dips steeply toward the northeast and is generally oblique to the average...
topographic slope in and around Laprak. In some places near the head of the landslide, however, discontinuities in quartzite are nearly parallel to the local topographic slope. The quartzite at the ridge near the headscarp of the landslide contains joints and fissures that make rockslide and rockfall potential an issue, in addition to landsliding further down the slope. There is a 5-m- to 13-m-thick layer of colluvium, as estimated from stream-bank exposures, above the bedrock in most places along the slope.

Topographic slope in the vicinity of Laprak averages about 30° and in some places exceeds 45°; there is a ridgeline above its head and the Raizo Khola (river) at its toe. More than 70 percent of the area is steeper than 25°, and more than 30 percent is steeper than 30°. Elevation ranges from about 2,500 m at the ridgeline to about 1,750 m at the toe along the Raizo Khola. The landslide is about 1,550 m long and 200 m to 650 m wide, and there are 25- to 60-cm-wide and 4- to 12-m-long tension cracks in several places. Springs also appear during the monsoon season. In particular, soil along the banks of the adjacent Chhelong Gully is chronically unstable, and the houses along the edge of the gully are more vulnerable than those on other parts of the landslide. New cracks continue to develop in and around the village with each episode of landslide movement.

The landslide is a complex consisting of several translational landslides, rotational landslides, debris flows, and rockslides (Varnes and Cruden, 1998); the translational components are predominant. Field observations and interpretation of mountain-scale geomorphology using shaded relief images and contour maps produced from a 30 m ASTER (Advanced Spaceborne Thermal Emission and Reflection) satellite DEM (digital elevation model) obtained for this project suggest that the current Laprak landslide may be part of a much larger ancient landslide complex of unknown age and depth, which may or may not involve bedrock (Figure 5).

FIELD, LABORATORY, AND OFFICE INVESTIGATIONS

Fieldwork

The descriptions in this paper are based upon fieldwork performed during four visits to Laprak. Preliminary observations were made in November 1999, three months after the landslide. Additional

A detailed 1:2,000 topographic map of the landslide was prepared by contouring points surveyed using a total station and global positioning system (GPS) unit. The locations of footpaths, surface drainages, and representative buildings in the village were also surveyed; however, the density of construction and steep slopes within the village made it impossible to survey all of the building locations during the time available for fieldwork. Colluvium thickness, as exposed in gullies adjacent to the landslide, surface drainage paths, and seepage locations, were noted during mapping.

Because the colluvium contains a considerable proportion of sand and rock fragments, it was not feasible to obtain undisturbed samples for testing. Disturbed samples, however, were obtained from depths of about 1 m at four different locations shown in Figure 4.

The strike and dip of foliation in the schist, joints, and the ground surface slope were measured near the headscarp, along the western edge, and along the southern edge of the landslide to obtain information about rock fractures and constrain kinematic slope stability analyses. A modified Q-system chart (Barton, 2002) was used for rock mass classification in the field. Spacing of discontinuities, roughness of discontinuities, groundwater flow, discontinuity length, aperture, or separation, infilling materials, and weathering grade were also recorded.

Digital Terrain Modeling and Landslide Morphology

The topographic contours were interpolated onto a 2 m grid to create a digital elevation model (DEM) using the geographic information system (GIS) software GRASS 6.3. This was accomplished by importing the contours, rasterizing them, and then interpolating values between the rasterized contours. The resulting DEM and geomorphic derivative maps

Figure 5. Areas of suspected landsliding along the Raizo Khola drainage in the vicinity of Laprak, as inferred from field observations and interpretation of a 30 m ASTER DEM. Simulated illumination is from an inclination and azimuth of 30°/315°.
created from it were used to visualize the surface morphology and lateral extent of the landslide, as well as to ascertain the control of bedrock structures on colluvium accumulation, using a suite of shaded relief, contour, slope angle, and slope aspect maps produced using standard GRASS functions to supplement the topographic contours (Figure 6A–C).

The head of the active portion of the landslide consists of an arcuate crack and attendant topographic break at the southwestern end of the slide. The northwestern (left) lateral boundary coincides with the Chhelong Gully. The southeastern (right) lateral boundary is indistinct but likely skirts a knob of shallow to exposed bedrock upon which the village school, which has not been damaged by landslide movement, is built and may follow the path of several small surface drainages in that area. Near the toe, the landslide widens downslope toward the Raizo Khola.

The derivative maps created from the DEM suggest that the Laprak landslide consists of four segments,
three of which constitute the active portion of the landslide. The inactive portion of the landslide, which extends approximately 200 m upslope from the currently active headscarp, consists of a quartzite boulder field derived from quartzite cliffs near the ridgeline. No cracks or other signs of ongoing movement are visible in this portion of the landslide, which is covered largely by cultivated fields. The three active portions of the landslide consist of a steep and thick upper portion, a less steep and possibly shallower central portion, upon which much of the village is located, and a steep, incised, and bulging toe that may consist of several subsidiary components. Both the upper and lower active portions are largely covered by terraced agricultural fields, and breaks in the slope angle map (Figure 6B) suggest that the toe may consist of at least two large landslides in addition to several smaller superficial landslides and erosional scars. The slope aspect map (Figure 6C), in particular, appears to illustrate bedrock structural controls on colluvium accumulation. The southeast- and northwest-facing slopes (green and yellow-orange, respectively, in Figure 6C) share orientations similar to metamorphic foliation and joints observed in the field. This bedrock signature is faint in the upper inactive and upper active portions of the landslide above most of the village, but it is much stronger in the middle portion that underlies most of the village, where the colored bands are narrow and well defined. Evidence of bedrock control extends downslope from the village, but the width of the bands on the aspect map increases and definition decreases, which may indicate that the bedrock is deeper near the toe than beneath the village and that thick colluvium precludes the topographic expression of all but the largest bedrock irregularities.

The relationship between surface topography and underlying bedrock structures can be further examined by exploiting the fact that if there is no soil, the slope aspect will coincide with the dip direction of the bedrock structures that dominate the local topography (e.g., Jaboyedoff et al., 2007). In situations where bedrock structures are covered with colluvium or other surficial deposits, however, the dip angle of the bedrock may not be accurately reflected by the ground surface slope. Therefore, only the slope aspect or dip direction was considered during this project. Figure 6D is a recoded version of the aspect map in which the red patches represent areas in which the ground surface dip direction agrees closely with the metamorphic foliation dip direction (D1: 330° to 360°) and the green patches represent areas in which the ground surface dip direction agrees with joints (D3: 120° to 150°). The slope aspect map also contains distinct zones that occur only in close association with the foliation-parallel slopes (D2: 295° to 325°). These zones are shown in purple in Figure 6D and, because of their spatial and geometric association with observed foliation attitudes, may represent intrafolial shears not originally observed in the field. The yellow areas in Figure 6D represent areas in which the dip direction is similar to the discontinuities along which planar rock slope failures occur near the head of Chhelong Gully (D4: 025° to 055°). Although the D4 discontinuities were observed only in quartzite near the gully head, the presence of large yellow patches in Figure 6D suggests that the D4 discontinuities may influence the general topographic slope and geometry of the colluvium mass comprising the main landslide in areas where they are not visible at the surface. Figure 7 is an upslope longitudinal view of the landslide created using the digital elevation model and discontinuity locations taken from Figure 6D, illustrating the role of bedrock structural controls on planar rocksliding at the head of Chhelong Gully and the accumulation of a colluvial wedge dipping downslope toward the Raizo Khola.

Soil and Rock Geotechnical Properties

Soil samples collected from four locations (L1, L2, L3, and L4 in Figure 4) were returned to the Indian Institute of Technology in Delhi for classification, index property measurement, and shear strength testing using standard methods. The specific gravity, density, liquid limit, plastic limit, and grain-size distribution for each sample were determined, and direct shear tests were conducted using remolded material. Pertinent properties are summarized in Table 1. A large 30 cm by 30 cm Shear Trac II direct shear test box was used to estimate the cohesion and strength of the soils. The soil samples were collected from an average depth of 1 m, and the applied normal loads for the direct shear tests ranged from 100 kN/m² to 300 kN/m². The shear strength testing was conducted on dry samples.

The grain-size distribution curves of the soils are shown in Figure 8 and summarized in Table 2. Soils from all four sites have more than 70 percent coarse materials, and their grain-size distribution curves are similar except soil at sample 2 (location L2), which contained a considerable amount of fines (about 17 percent silt). Three of the soil samples fall into the Unified Soil Classification System (USCS) GP (poorly graded gravel) category, and one falls into the USCS GP-GM (poorly graded gravel to silty gravel) category.

Field-scale rock mass strength parameters were estimated using a modified Q-system approach. The
The cohesive strength of the fractured bedrock was estimated using (Barton, 2002)

$$c = \frac{RQD}{J_n} \frac{UCS}{SRF} 100$$  \hspace{1cm} (1)

where $c$ is cohesion in MPa, $RQD$ is the rock-quality designation, $UCS$ is the uni-axial compressive strength, $J_n$ is the joint set number, and $SRF$ is a stress reduction factor. The friction angle was estimating using (Barton, 2002)

$$\phi = \arctan \left( \frac{J_r}{J_a} \right)$$  \hspace{1cm} (2)

where $\phi$ is the friction angle in degrees, $J_r$ is a joint roughness factor, $J_w$ is a joint water reduction factor, and $J_a$ is a joint alteration factor. Using field-based estimates for all of the required input, Eq. 1 and Eq. 2 yield estimates of $c = 480$ kN/m² and $\phi = 37^\circ$.

The saturated hydraulic conductivity of soil sands with $0.1 \text{ mm} \leq D_{10} \leq 0.3 \text{ mm}$ can be estimated as (Hazen, 1892; Bear, 1972)

$$k_{soil} = c f(n_s) D_{10}^2$$  \hspace{1cm} (3)

where $k_{soil}$ is in m/s, $D_{10}$ is the effective grain size in mm, $n_s$ is the porosity of the soil, $c$ is an empirical

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (m)</th>
<th>Cohesion (kN/m²)</th>
<th>Dry Unit Weight (kN/m³)</th>
<th>Saturated Unit Weight (kN/m³)</th>
<th>Friction Angle (°)</th>
<th>Porosity</th>
<th>Saturated Hydraulic Conductivity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>10–20</td>
<td>0</td>
<td>22</td>
<td>24</td>
<td>34</td>
<td>0.35</td>
<td>$5.4 \times 10^{-4}$</td>
</tr>
<tr>
<td>Rock</td>
<td>NA</td>
<td>480</td>
<td>24</td>
<td>25</td>
<td>37</td>
<td>NA</td>
<td>$6.1 \times 10^{-6}$</td>
</tr>
</tbody>
</table>

Figure 7. Upslope visualization of the Laprak landslide based on the digital elevation model created for this project, illustrating the wedge-like nature of the structurally controlled colluvium accumulation and the discontinuities responsible for rockslides at the head of Chhelong Gully. Representative discontinuity orientations, locations, and colors were taken from Figure 5D and are shown as large disks. Coordinates are in meters (UTM zone 44).
constant, and the porosity function is

\[ f(n) = 1 + 10(n - 0.26) \quad (4) \]

For the soil at Laprak, laboratory test results suggest values of \( c = 0.01, n_s = 0.35, \) and \( D_{10} = 0.2 \text{ mm} \). Those values yield a saturated hydraulic conductivity of \( k_{\text{soil}} = 5.4 \times 10^{-4} \text{ m/s} \), which is a reasonable value for the fine sandy soil comprising the colluvium matrix. Therefore, more complicated formulae were not evaluated.

The hydraulic conductivity of the bedrock was estimated as (Goodman, 1989)

\[ k_{\text{rock}} = \frac{\gamma_w}{6\mu} \left( \frac{a^3}{s} \right) \quad (5) \]

where \( k_{\text{rock}} \) is hydraulic conductivity of the rock in m/s, \( \gamma_w \) is the unit weight of water in N/m³, \( \mu \) is viscosity of water (1 \( \times 10^{-3} \text{ N-s/m²} \) at 20°C), \( a \) is joint aperture in mm, and \( s \) is joint spacing in mm. Although Goodman (1989) described Eq. 5 as being useful for back-calculation of aperture, \( a \), from \textit{in situ} hydraulic conductivity test results, in this project it was used to estimate \( k \) from field-based estimates of \( a \) and \( s \). Field observations of discontinuities in the quartzite bedrock exposed at the head of Chhelong Gully suggest typical values of 1 mm (which we infer to decrease by an order of magnitude to \( a = 0.1 \text{ mm at depth} \)) and \( s = 300 \text{ mm} \), so that Eq. 5 yields a fractured bedrock saturated hydraulic conductivity estimate of \( 6.1 \times 10^{-6} \text{ m/s} \) and a soil:bedrock hydraulic conductivity ratio of 88:1.

**Rock Slope Stability Analysis**

Because the Laprak landslide is a complex made up of components with different mechanisms that interact in ways difficult to ascertain after the fact, it is impossible to conduct a single overall slope stability analysis. Therefore, three kinds of analyses were performed to better understand different aspects of the landslide. First, kinematic analysis was used to establish the nature of rockslides in quartzite near the head of the landslide, which may have played a key role in triggering the larger soil landslide in 1999. Second, the potential for rockslides along planar surfaces in the quartzite (as suggested by the kinematic analyses) was evaluated for dry static, saturated static, and seismic conditions using limit equilibrium analysis. Third, the nature of and potential for movement of the main colluvium landslide mass with composite slip surfaces were evaluated using limit equilibrium analyses for dry, saturated, and seismic conditions as described in a subsequent section of this paper. Pore-water pressures in the main colluvium landslide were estimated using two approaches: subjective specification of a phreatic surface (water table), taking into account the locations of known monsoon-season seeps, and finite element simulation of steady-state flow through the colluvium and underlying fractured bedrock.

Field measurements show that a prominent discontinuity set in quartzite outcrops at the head of Chhelong Gully is oriented 48°/039° (dip/dip direction), whereas the ground surface is oriented 52°/044°, or nearly parallel to the predominant discontinuities in that area. Discontinuity width is less than 1 mm without filling materials but increases to more than 2 mm in areas where filling is present; it may be less at depth. Discontinuity spacing in the quartzite is generally less than 0.7 m, and joint surfaces are slightly smooth. Kinematic analysis of the discontinuous quartzite using Markland’s (1972) method and the estimated rock friction angle suggests that the slope is susceptible to rocksliding along the slope-parallel discontinuities, as opposed to wedge or topping instabilities (Figure 9). Subsequent limit equilibrium rock-slope stability analyses using RocPlane 2.0 software yield factors of safety against sliding of 1.25 for dry static conditions, 0.92 for fully saturated static conditions, and 0.98 for dry seismic conditions.
conditions with a pseudo-static horizontal acceleration coefficient of 0.12 based upon local codes. Figure 10 illustrates the geometry and results for the fully saturated static analysis; the dry static and seismic analyses used identical geometry but different loading conditions.

Soil Slope Stability Analysis

The limit equilibrium slope stability analyses and supporting finite element simulations of groundwater flow for the colluvium comprising the main portion of the Laprak landslide were performed using the commercial slope stability software Slide 5.0. For each case evaluated, factors of safety were calculated using the Morgenstern-Price, Janbu Corrected, and Spencer methods. Although Slide 5.0 can calculate pore-water pressures for variably saturated steady-state or transient flow as described by the Richards (1931; also see Fredlund and Rahardjo, 1993)
equation, this analysis used a high steady recharge rate equal to the saturated hydraulic conductivity of the soil ($k_{soil} = 5.4 \times 10^{-4} \text{m/s}$) to ensure complete saturation under steady-state conditions as a limiting situation. This was done to avoid the complications of extreme uncertainty in the initial conditions, soil moisture characteristic curves, and precipitation input necessary to constrain the highly non-linear problem of transient flow in variably saturated soil and rock. Instead, the steady-state simulations were supplemented with a simpler order of magnitude estimate of the hydrologic response time of the Laprak landslide. Analysis of debris-flow mobilization and movement, which is an additional hazard at Laprak, is beyond the scope of this paper.

Stability analysis of the main colluvium slope at Laprak was performed using the parameters in Table 1 and a vertical surface load of 2 kN/m$^2$ in the area occupied by the village in order to simulate the weight of the stone buildings. For the dry season simulations, the phreatic surface was specified along the soil-bedrock contact except at the locations of springs above the village and at the toe of the slope along the Raizo Khola, where it was allowed to rise to the ground surface. For the wet season simulation using a specified phreatic surface, the phreatic surface coincided with the ground surface along the entire length of the slope, but the bedrock was assumed to be impermeable, eliminating the possibility of flow into or out of the bedrock. The topography used in the analyses was a representative profile taken down the centerline of the landslide. Results for the static and pseudo-static seismic analyses with a horizontal seismic coefficient of 0.12 and vertical seismic coefficient of 0.08 are shown respectively in Tables 3 and 4. Differences among the Morgenstern-Price, Janbu Corrected, and Spencer methods were negligible for each case. Figures 11 through 14 illustrate the model geometry and Morgenstern-Price factor of safety distributions for multiple slip surfaces as generated by the Slide 5.0 software.

As would be expected from its history of wet season movement, the main colluvium slope is calculated to be stable under both dry static and dry seismic conditions. Both the specified phreatic surface and finite element seepage results show the upper portion to be the least stable and the middle (village) portion to be the most stable parts of the main colluvium slope, for both static and seismic wet season conditions. The finite element seepage results, however, yield consistently higher factors of safety for all portions of the slope than does the specified phreatic surface model (see Tables 3 and 4). For wet season static conditions, the finite element seepage results suggest that the middle (village) portion may be nearly or perhaps even marginally stable, with a factor of safety ranging from 0.96 (Spencer method) to 0.98 (Morgenstern-Price method).

The consistent differences in calculated factor of safety values obtained using the specified phreatic surface and finite element seepage models suggest that, even though the soil:bedrock hydraulic conductivity ratio of 88:1 implies that flow into and out of the bedrock might be minimal, it is significant enough to produce noticeable changes in slope stability. The fractured bedrock functions as a drain that allows downward seepage of water out of the colluvium and a concomitant increase in local slope stability (see Iverson and Major (1986) for a discussion of seepage vector orientation). Factor of safety differences between the phreatic and finite element models ranged from modest (about 4 percent for the village portion of the slope under wet static conditions) to potentially significant (about 27 percent for the lower

### Table 3. Composite surface factors of safety against sliding for static dry and wet season conditions.

<table>
<thead>
<tr>
<th>Method</th>
<th>Dry Season (Specified Phreatic)</th>
<th>Wet Season (Specified Phreatic)</th>
<th>Wet Season (Finite Element Method)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lower</td>
<td>Upper</td>
<td>Lower</td>
</tr>
<tr>
<td>Morgenstern-Price</td>
<td>1.63</td>
<td>1.55</td>
<td>0.75</td>
</tr>
<tr>
<td>Janbu Corrected</td>
<td>1.67</td>
<td>1.59</td>
<td>0.74</td>
</tr>
<tr>
<td>Spencer</td>
<td>1.63</td>
<td>1.51</td>
<td>0.74</td>
</tr>
</tbody>
</table>

### Table 4. Composite surface factors of safety against sliding for seismic dry and wet season conditions.

<table>
<thead>
<tr>
<th>Method</th>
<th>Dry Season (Specified Phreatic)</th>
<th>Wet Season (Specified Phreatic)</th>
<th>Wet Season (Finite Element Method)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lower</td>
<td>Upper</td>
<td>Lower</td>
</tr>
<tr>
<td>Morgenstern-Price</td>
<td>1.24</td>
<td>1.21</td>
<td>0.59</td>
</tr>
<tr>
<td>Janbu Corrected</td>
<td>1.22</td>
<td>1.24</td>
<td>0.57</td>
</tr>
<tr>
<td>Spencer</td>
<td>1.22</td>
<td>1.23</td>
<td>0.59</td>
</tr>
</tbody>
</table>
The Laprak landslide is a complicated feature, and understanding its reactivation is made more difficult by the fact that it occurred in a remote region with no baseline data or detailed pre-slide maps for reference. Moreover, information about the events leading up to the reactivation is limited to the anecdotal accounts of residents. Nonetheless, it is possible to make some important inferences about the sequence of events leading up to reactivation of the landslide and likely future patterns of movement.

Reactivation of the landslide apparently began with a series of small planar rockslides at the head of Chhelong Gully, where discontinuities dip nearly parallel to the local topographic slope. Quarrying for building stone may have exacerbated the rockslide potential by locally over-steepening the slope or removing key blocks. Rock slope stability analyses suggest a factor of safety against sliding of 0.92 for completely saturated static conditions, which is a reasonable supposition given the antecedent rainfall and intense rainstorm on July 3, 1999. The rockslide(s) may or may not have been related to the mobilization of the debris flow. One possibility is that debris-flow mobilization was facilitated by rapid loading and pore-pressure increases as a consequence of the rockslide(s). Another possibility is that the heavy rainfall alone was sufficient to saturate the colluvium and generate small landslides near the head of Chhelong Gully that subsequently mobilized into a debris flow, without the contribution of rockfall loading. The available field evidence does not allow us to favor either hypothesis over the other.

Regardless of its trigger, the debris flow scoured Chhelong Gully and led to the development of rotational slumps along the lower reaches of the gully. The scouring and slumping may have reduced the factor of safety of the colluvium mass and triggered the main landslide; however, it does not

Figure 11. Results of limit equilibrium slope stability analysis of the Laprak landslide for dry static conditions obtained using the Morgenstern-Price method (Table 3).

portion of the slope under wet seismic conditions, although in no case did the finite element results predict stability under conditions for which the phreatic surface model did not. Had the finite element simulations not been performed, this situation would not have been intuitively obvious given the large soil:rock hydraulic conductivity contrast.

DISCUSSION

The Laprak landslide is a complicated feature, and understanding its reactivation is made more difficult by the fact that it occurred in a remote region with no baseline data or detailed pre-slide maps for reference. Moreover, information about the events leading up to the reactivation is limited to the anecdotal accounts of residents. Nonetheless, it is possible to make some important inferences about the sequence of events leading up to reactivation of the landslide and likely future patterns of movement.

Reactivation of the landslide apparently began with a series of small planar rockslides at the head of Chhelong Gully, where discontinuities dip nearly parallel to the local topographic slope. Quarrying for building stone may have exacerbated the rockslide potential by locally over-steepening the slope or removing key blocks. Rock slope stability analyses suggest a factor of safety against sliding of 0.92 for completely saturated static conditions, which is a reasonable supposition given the antecedent rainfall and intense rainstorm on July 3, 1999. The rockslide(s) may or may not have been related to the mobilization of the debris flow. One possibility is that debris-flow mobilization was facilitated by rapid loading and pore-pressure increases as a consequence of the rockslide(s). Another possibility is that the heavy rainfall alone was sufficient to saturate the colluvium and generate small landslides near the head of Chhelong Gully that subsequently mobilized into a debris flow, without the contribution of rockfall loading. The available field evidence does not allow us to favor either hypothesis over the other.

Regardless of its trigger, the debris flow scoured Chhelong Gully and led to the development of rotational slumps along the lower reaches of the gully. The scouring and slumping may have reduced the factor of safety of the colluvium mass and triggered the main landslide; however, it does not
appear to have been a necessary event because factors of safety are well below unity for both saturated groundwater options evaluated as part of this study even without considering scour-induced modification of the toe. According to the results of the stability analyses, the colluvium landslide would have moved even if the scouring and slumping had not occurred had pore pressures been sufficiently high. The wet season factors of safety $\ll 1$ in Table 3 suggest that movement does not require complete saturation, but instead can occur if the soil column is only partly saturated.

The relationship between the dominant discontinuity and local topographic slope orientations in the quartzite outcrops at the head of the slope suggest planar rocksliding occurred in that area (Figure 9). Visualization of discontinuity orientations and the general correspondence between those orientations and the colluvium surface, however, suggest that the main colluvium landslide mass is a structurally controlled soil—not rock—wedge with a triangular or trapezoidal cross section. In the case of a triangular cross section, the wedge shape would be controlled by the red (D1) and green (D3) bedrock discontinuities shown in Figures 6 and 7. In the case of a trapezoidal cross section, the wedge shape would be controlled by red (D1), green (D3), and yellow (D4) discontinuities shown in those two figures. Although the yellow (D4) discontinuities were observed only in quartzite outcrops at the head of the slope, large areas of yellow ground in Figure 6D suggest that they may persist beneath the colluvium and truncate the bottom of a triangular wedge bounded by the red and green discontinuities. The remote location of Laprak makes it impossible to confirm the inferred bedrock control on colluvium wedge geometry with oriented cores or borehole televiewer surveys, but our inferred geometry seems reasonable given the local structural setting.

Calculated factors of safety for the colluvium above the village are significantly less than those for the colluvium below the village, and both the upper and...
Figure 13. Results of limit equilibrium slope stability analysis of the Laprak landslide for wet static conditions obtained using the Morgenstern-Price method (Table 3). Pore-water pressure was specified using a finite element model of steady-state seepage through both soil and bedrock (soil:rock hydraulic conductivity ratio of 88:1).

Figure 14. Results of limit equilibrium slope stability analysis of the Laprak landslide for wet seismic (pseudo-static) conditions with horizontal and vertical accelerations of 0.12 g and 0.08 g, respectively, obtained using the Morgenstern-Price method (Table 4). Pore-water pressure was calculated using a finite element model of steady-state seepage through both soil and bedrock (soil:rock hydraulic conductivity ratio of 88:1).
lower portions both appear to be less stable than the village portion. The wet season static factor of safety near unity for the village portion suggests that it would be very nearly stable, or perhaps even marginally stable, on its own but is moving as a consequence of its location between two much less stable portions of the slope. Baum and Fleming (1991) demonstrated that the zones of maximum displacement in some large landslides occur in the relatively undeformed middle portions between downslope zones of shortening and upslope zones of stretching. Although there are no comparable displacement or strain measurements available for the Laprak landslide, the existence of a relatively undeformed (but highly displaced) middle section between more highly deformed upper and lower sections appears to be consistent with the observed and calculated conditions at Laprak.

Even though the modeled bedrock hydraulic conductivity is nearly two orders of magnitude less than the colluvium hydraulic conductivity, the difference between the specified phreatic surface and finite element seepage results suggests that the discontinuous bedrock is permeable enough to allow drainage from the colluvium into the bedrock and provide for local decreases in instability that range from modest to potentially significant. This suggests that common assumptions of perfectly impermeable bedrock and a phreatic surface coincident with the ground surface may in general be an oversimplified approach. Although consideration of bedrock seepage in this case proved to be less conservative than the more common phreatic surface assumption, this relationship may not be universally applicable, and it should be evaluated on a case-by-case basis when performing slope stability analyses.

The seismic factors of safety in Table 4 suggest that while a moderate (horizontal pseudo-static coefficient of 0.12) dry season earthquake should not be expected to trigger future movement, a wet season earthquake could exacerbate the effects of high pore-water pressures and produce factors of safety against sliding as low as 0.5. Although a dry season earthquake producing the modeled acceleration might not trigger landslide movement, it may still result in collapse of or damage to unreinforced stone buildings. Moderate seismic acceleration may have some effect on rock slope stability near the head of the landslide, but it should not be as significant as the effect on the main colluvium landslide.

Mitigation measures currently under consideration include surface-water control, minor drainage projects, and perhaps replacement of unreinforced stone buildings with lightweight and flexible wooden structures. The remote location and economic considerations preclude heavy construction projects or such remedial measures as horizontal drains or toe buttresses. To that end, relocation of the village is also a possibility. However, there are no nearby locations that offer both stable slopes and a reliable water supply. Wholesale relocation of the community to a remote region is likely to entail considerable social and cultural difficulties, and in that regard is not an attractive option.

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