# Mobile Soil and Rock Flows

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Synopsis: Mobile soil and rock flows arise in a variety of geological and geomorphological settings. Four case histories are presented by way of example. The first, from Hong Kong, resulted from the collapse of poorly compacted fill during infiltration of rainwater. The second, from Vancouver Island, is due to shallow instability and erosion in debris and is also associated with intense rain. The third, from Brazil, is caused by large scale instability in residual soil followed by channelling of the debris through a steep, narrow gorge. The fourth example refers to the general problem of rock debris avalanches where large volumes of dry rock can achieve remarkable mobility. A historical example from British Columbia is cited. Examples are given which illustrate that protective structures are often best designed utilizing basic principles of fluid mechanics rather than more traditional views of soil behavior. The object of the paper is to draw attention to the class of problems associated with mobile soil and rock flows and the research needed to understand their mechanisms.

## INTRODUCTION

In routine considerations of slope stability, analyses are undertaken within the framework of limiting equilibrium. For purposes of design, the shear strength parameters required to equilibrate the potential sliding mass are compared with the available strength parameters in terms of the Factor of Safety. If the Factor of Safety is greater than unity, large movements will not occur. The selection of the appropriate Factor of Safety to limit deformations depends upon geological history, soil type, groundwater conditions, the operational requirements of the slope, and experience.

If the Factor of Safety is less than unity, there is an imbalance between driving forces and resisting forces and the soil mass will accelerate. The subsequent motion, if it is translational or rotational, is readily calculated within the framework of rigid body mechanics provided that the shearing resistance along the base of the moving mass is known throughout the motion. For rotational motions or translational trajectories that flatten, the moving mass soon enters a regime where resisting forces are greater than disturbing forces and it rapidly decelerates to a stop. If during the motion the mass were to encounter some obstacle, considerable resistance, up to the passive resistance of either the obstacle or the sliding mass, would be developed. These forces are calculable from traditional concepts of earth pressure.

There is a large body of experience, not only with natural phenomena but also with engineering works, that indicates that the considerations discussed above grossly underestimate the mobility of many moving soil and rock masses. Moreover there is a class of problems that arises in practice that is not concerned with the evaluation of whether a slope will move or not; but instead is obliged to assume movement and design against the consequences. Where it is not practical to eliminate movements, the following questions arise:

(1) How much material will move?
(2) What will be the time history of the movements in terms of velocities and accelerations?
(3) How are protective structures designed against moving masses ?

This paper draws attention to the behavior of soil and rock masses that are more mobile than indicated by rigid body mechanics and traditional considerations of soil mechanics. Related phenomena are snow avalanches and high density turbidity currents. Four case histories are presented to illustrate mobile soil and rock masses. Even though their fundamental behavior is not well understood, it is important to gain an appreciation of how to address these problems in practice.

## MOBILE SOIL MASSES

### *Hong Kong Mud Avalanche*

On August 25, 1976 at about 10:00 a.m., a fill slope immediately behind Block 9 of the Sau Mau Ping Estate, Hong Kong, failed. The resulting mud avalanche buried the ground floor of the block killing eighteen people (Figure 1). Catastrophic landslides have not been rare events in Hong Kong during recent years. In June of 1972 another failure of an embankment, also at Sau Mau Ping, resulted in the death of 71 persons. At that time a Commission of Inquiry conducted an investigation into the Sau Mau Ping and other slides that had occurred. Among their various findings, they concluded that no fault was found with the manner in which the design and construction of the embankment was carried out.

Since high fill slopes are common in Hong Kong, the consternation that arose after the 1976 failure is readily understood. In these circumstances the Government of Hong Kong sought independent opinion on the cause of the slope failure at Sau Mau Ping and they formed an Independent Review Panel on Fill Slopes. The material in this section is drawn from the report of the Independent Review Panel*. In their inquiries, the Panel was assisted by Binnie and Partners (Hong Kong), Consulting Engineers.

Figure 1. View of catastrophic mud avalanche of 25th August, 1976 at San Mau Ping. Photo courtesy of South China Morning Post Ltd., Hong Kong

A cross-section of the slope that failed is shown in Figure 2. The fill slope was constructed at 1-1/2:1 to a height of about 30 m. Failure occurred following intense rain. However, the return period of the storm was about 4 to 5 years and cannot be considered abnormally severe for the conditions that prevail in Hong Kong. The debris was essentially a mud and it was extremely mobile. Its nature may be seen in Figure 3.

Following clean-up and the immediate installation of surface protection to minimize erosion in subsequent rain, extensive investigations were conducted. These included borings and trial pits to obtain samples and stratigraphy, piezometric installations to determine ground water characteristics, laboratory tests to obtain strength and deformation characteristics, and theoretical investigations, particularly with regard to the relation between the advance of a wetting front due to infiltration and the onset of instability. The details of these investigations are beyond the scope of this paper but the main results can be summarized as follows:

(1) The soil type on the fill slope was decomposed granite soil, a coarse grained sand with appreciable silt and clay content, of a texture well known in Hong Kong and having no special peculiarities.

(2) The soil on the slope was in an extremely loose state to a depth of at least 2 m below the slope

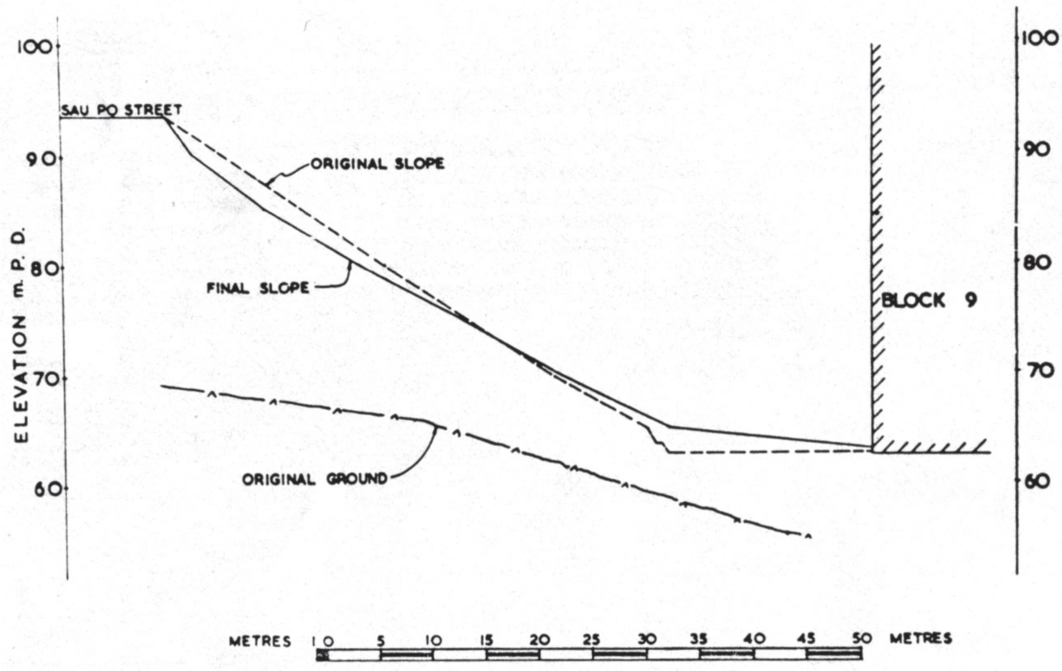


Figure 2. Typical section

*The Independent Review Panel was composed of J.L. Knill, Professor of Engineering, Geology, Imperial College, University of London; P. Lumb, Professor of Civil Engineering, University of Hong Kong; S. Mackey, Professor of Civil Engineering, University of Hong Kong; V.F.B. de Mello, Professor of Civil Engineering, University of Sao Paulo; N.R. Morgenstern, Professor of Civil Engineering, University of Alberta; B.G. Richards, CSIRO, Australia.

surface, the dry densities being an average 1.35 $t/m^3$, corresponding to about 75 % of standard compaction.

(3) The soil on the slope was definitely layered parallel to the slope surface, with layers between about 100 and 300 mm thick, see Figure 4. This was the result of end-tipping with no compaction after filling.

(4) Beyond the crest of the slope, the dry densities were low but variable to a depth of 7 m, dropping from about 1.65 $t/m^3$ to about 1.2 $t/m^3$ (90 to 70% of standard compaction) and showing a gradient of densities with depth consistent with the soil having been placed in horizontal layers 1 to 3 m thick. At greater depths, the dry densities were around 1.5 $t/m^3$ to a depth of 20 m below the surface.

Because of inadequate compaction, the soil strengths were very much less than would be obtained with well compacted fill. Rainwater falling on the surface of a loose soil can percolate readily into the soil mass, wetting the soil to an appreciable depth and reducing the strength even further. If the soil is almost saturated, during shearing the soil may collapse, lose its strength, liquify and cause a mud avalanche. Laboratory tests indicated that the Sau Mau Ping soil contracts at a dry density lower than 1.5 $t/m^3$ (about 85% of standard compaction) but will expand at higher densities. Consequently, the upper few metres of fill material in the slope can be stated to have been in a potentially collapsible form.

A major finding of the Independent Review Panel was that failure resulted from the development of seepage conditions within the wetted zone as water penetrated the face of the slope. The loss of strength of the fill resulted in downhill movement and an almost instantaneous conversion of the slope into a mud avalanche with considerable destructive energy. As indicated in Figure 2, the volume of the slide was only a few thousand $m^3$. The hazard arises from its extreme mobility.

Loose fill embankments were found to be widespread in Hong Kong and the Panel recommended reconstruction of existing embankments by excavation, recompaction and provision of suitable surface protection and drainage works. There is considerable pressure in Hong Kong to maintain high, steep slopes and excessive flattening would result in a substantial economic burden to the community. While it is not possible to avoid all instability at slopes 1-1/2:1, it is possible to eliminate the hazard to public safety by adequate compaction and drainage. The hazard arises from the mobility of the mud avalanche. Instability of well compacted material would not develop fluid-like characteristics and if any instability arises it could be treated on a maintenance basis.

Figure 3. Characteristics of debris to the rear of Block 9, looking away from Block 15, at about 11.00 am on 25th August, 1976. Photo courtesy of Housing Authority

Figure 4. View showing the shallow depth of the slide and layers parallel to the slope

** This study was undertaken by Mr. H. Naismith, Thurber Consultants Ltd., Victoria, British Columbia, who has kindly provided the information given here.

Figure 5. Gullies above Port Alice

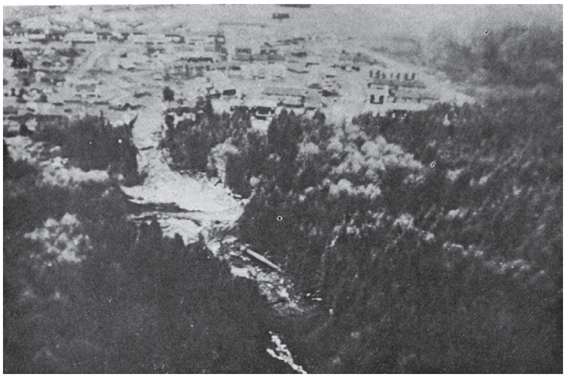

Figure 6. 1973 debris flow

Figure 7. Mobility of 1973 flow

Figure 8. The 1975 debris flow

*Port Alice Debris Flows***

The town of Port Alice is located on the north end of Vancouver Island, British Columbia, on the west coast of Canada. It is an area of heavy precipitation with as much as 500 cm of rain annually. The town was established in 1965 to serve logging and paper industry interests. It was built on a fan composed of slide debris reworked by alluvial processes. The elevation at the apex of the fan is about 100 metres above sea level and at the crest of the ridge, behind the town, the elevation is about 870 metres above sea level. As can be seen in Figure 5, there are three gullies incised into the ridge which provide the source for the debris flows that attack the town periodically.

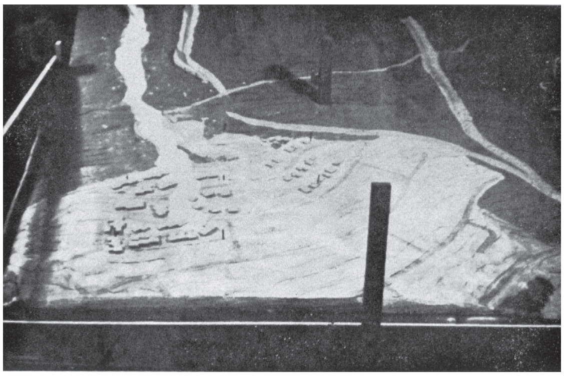

Figure 9. Physical model of debris flow

Figure 6 shows a view of the lower portion of the path of the 1973 flow. The volume of debris was about 26,000 $m^3$ and 30 cm diameter trees were broken down before the flow entered the town. In Figure 7, looking toward the mouth of the gully, one sees the 1973 slide. The house near the middle of the picture was moved off its foundation, but for the most part the debris flowed around buildings and was diverted by minor topographic irregularities. In town, the slide moved at a fast walking speed, i.e. 5 to 8 km per hour.

Another debris flow occurred in 1975 bringing about 7,500 $m^3$ of material into the town from a different gully. Figure 8 illustrates the 1975 debris flow where it entered into the upper part of the town. While none of the buildings were displaced from their foundations, considerable damage occurred due to flooding after the debris flow.

It is possible to provide protection against debris flows in various ways. Systematic clearing at the source may be undertaken and in some instances retention dams have been constructed. In this instance, the Consulting Engineers chose to protect the town by constructing a dike. A model of the configuration of the town and adjacent gullies was constructed to a scale of 1 : 250. While the rheology of debris composed of soil, vegetation, trees and water is obviously not understood, it proved possible to find by trial and error a mixture of bentonite, barite and water that produced a reasonable duplication of the volume of debris, depth, area of deposition and velocity of the front of the debris flow recorded in prior events. Modelling of the 1973 flow is shown in Figure 9. Hence protection works against a proposed design flow could be modelled on a rational basis.

The concern at Port Alice was with mobility of soil debris and design proceeded more with considerations of fluid mechanics than with traditional concepts of soil mechanics. The use of prior natural occurrences as a calibration in order to find the appropriate modelling medium is especially noteworthy.

*Grota Funda Flow*

The third case is located in Brazil, in the State of Sao Paulo, near the town of Paranapiacaba. The actual site is close to the Santos Sao Paulo highway as it traverses the Serra do Mar. Grota Funda means deep gorge and it is the flow within this gorge that has been of concern. Some of the details presented here have been obtained by Geotecnica, S.A., Sao Paulo.

The geological conditions are similar to those encountered elsewhere in the Serra do Mar. A deep weathering profile has formed in the predominantly granite-gneisses. The upper soil-like zones are thin to absent within the steep confines of the gorge and instability has developed in the saprolite, a mixture of residual soil and boulders of decomposed rock. Locally the climate is subtropical and humid. The average yearly rainfall is more than 2000 mm and local intensities can be very high, reaching more than 100 mm/hour. Even the drier months have significant rainfalls.

At the head of the gorge, a steep slope rises approximately 200 m to the upland. Following intense

Figure 10. Source of Grota Funda flow

Figure 11. The Grota Funda flow

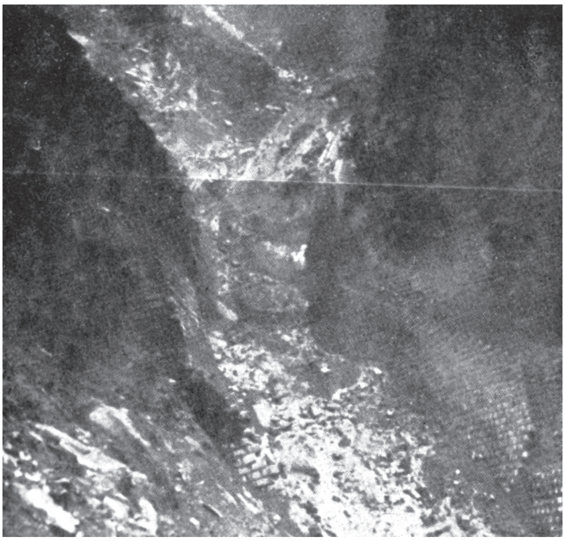

Figure 12. Channelization of flow in gorge. Upstream front view of gorge showing total channel obstruction

rains at the end of 1975, and early in 1976, major instability has occurred in this slope resulting in an area of about 300,000 $m^2$ becoming unstable. It has not been possible to estimate the volume of the potential slide mass with any accuracy, but it is certainly a few million cubic metres. This mass accumulates within the slope below the upland and is then channelled at high velocity through the gorge below.

Figure 10 illustrates the source area of the debris. The characteristic "sugar loaf" weathering forms are discernible in the right hand side of the photograph. Figure 11 shows the source material moving down the steep slope and gathering on a bench above the deep gorge. Two railway bridges cross the gorge. The short upstream bridge connects two tunnel sections of an old funicular railway. The downstream bridge is more recent and is longer. The debris flows within the gorge impinge on one of its piers and this is the major concern associated with these flows. Both railroads are important arteries connecting Sao Paulo with the port of Santos. The channelization of the flows may be seen on Figure 12. The geometry of the channel influences both the discharge velocity and the height to which the debris will rise. Discharges of debris occur episodically and traverse the gorge carrying enormous core stones. These are pulsating, high velocity flows that have been observed to travel at 30 km/hour. Flow debris in the gorge and some of the core stones left by a flow are shown on Figure 13 and Figure 14. Figure 15 indicates the level to which debris rises when a flow is passing through the gorge. The bridge pier that is to be protected from the flows is evident in Figures 15 and 16, as is the destructive force of the surges.

It was not practical to work on the unstable slope at the head of the gorge to eliminate the instability and the most effective protection for the pier appeared to be afforded by the construction of a tied-back retaining wall. The object of this design would be to deflect flows away from the pier and to protect it from the impact of the large, fast moving core stones. Traditional considerations of earth pressure are not appropriate for this design and, as in the Port Alice Flows, an approach based on fluid mechanics is preferable. The force to be resisted by the anchors can be calculated by evaluating the change in momentum of the design debris flow. In addition, reinforcing within the wall is needed to provide resistance against the local impact of the core stones.

## ROCK DEBRIS AVALANCHES

The three previous cases have been presented in order of increasing size. The mud avalanche in Hong Kong contained only a few thousand $m^3$ while the Grota Funda flow contained over a million $m^3$. The availability of water is a common factor in each case. Water is undoubtedly a necessary factor in the case of the Hong Kong and Port Alice flows. It is also an

Figure 13. Core stones in gorge

Figure 14. Debris in Grota Funda

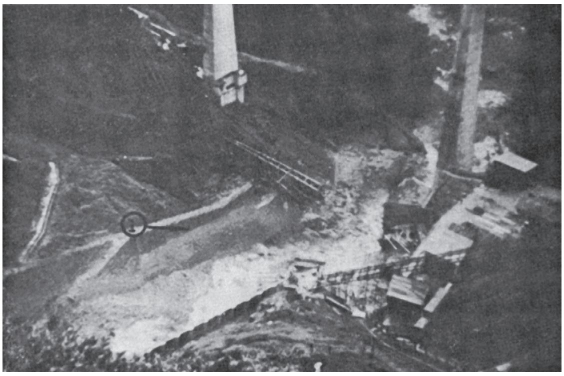

Figure 15. Level of flow passing piers

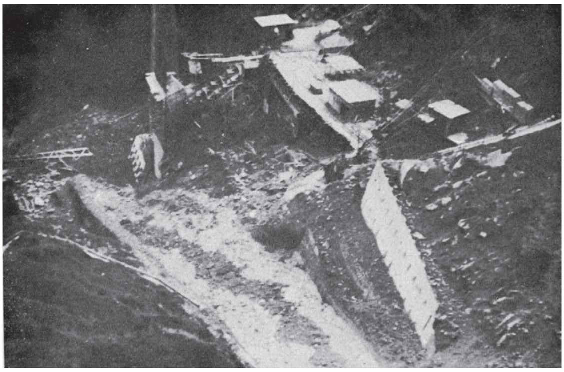

Figure 16. Flow impinging on bridge pier

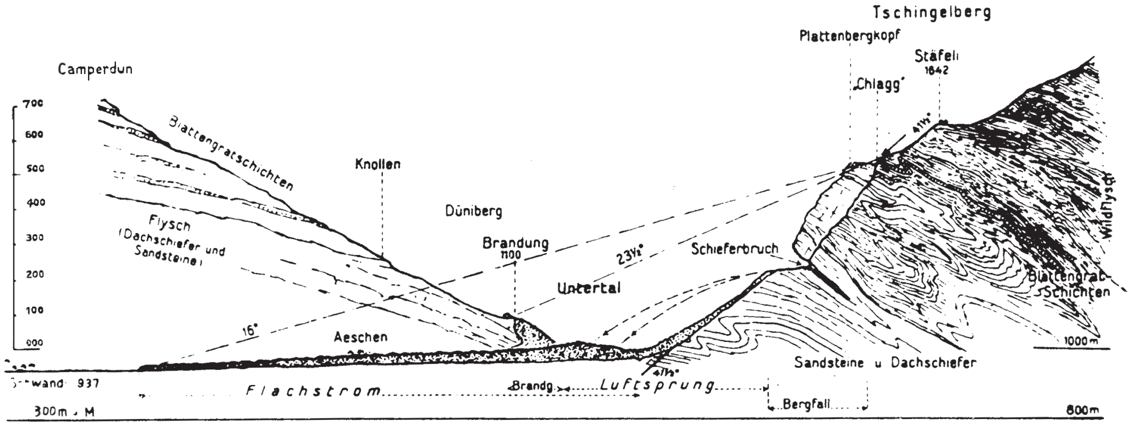


Figure 17. The Elm, Slide (Heim, 1932)

Figure 18. Debris from the Elm Slide (Heim, 1932)

important contributing factor in the Grota Funda flow. However, some pulses were observed to descend the gorge in a remarkably dry state, unassociated with any immediately prior rainfall. The rock debris avalanches referred to in this section are much larger phenomena, generally over 10 x $10^6$ $m^3$, of remarkable mobility, and apparently do not require water as a necessary part of the mechanism.

Heim (1932) was perhaps the first to observe systematically the characteristics of rock debris avalanches and to suggest that they flowed like a fluid. It was the Elm slide of 1881 that initiated his interest in this behavior. The characteristics of the Elm slide are illustrated in Figure 17 which also draws attention to the mobility of the debris. The nature of the debris is shown in Figure 18. Vivid eyewitness accounts have recently been reviewed by Hsü (1975) which indicate that while the mass was essentially dry, it was fluidized. Many other examples of comparable mobility have been listed by Hsü who established a broad correlation between the Excessive Travel Distance (km) and the volume of the avalanche. The Excessive Travel Distance is the horizontal distance travelled by the tip of the avalanche beyond what would be expected on the basis of frictional sliding without pore pressure effects. While there are undoubtedly a variety of factors such as height of fall and valley geometry that also affect mobility, it appears that volumes of about 5 x $10^6$ $m^3$ are needed to develop fluidization in rock. For volumes of about 10 x $10^6$ $m^3$, the Excessive Travel Distance is 1 to 2 km and for volumes of about 50 x $10^6$ $m^3$ it rises to 2 to 4 km. Not all rock slides transform into flows.

Figure 19 illustrates the source area of a historical rock debris avalanche in Western Canada that has received considerable attention recently (Moore & Mathews, 1978; Garibaldi Advisory Panel, 1978). The cliff is known as the Barrier and it is located at the head of Rubble Creek Valley, about 30 km north of the town of Squamish, near the west coast of British Columbia. In 1855 a rock debris avalanche occurred involving approximately 30 x $10^6$ $m^3$. The avalanche flowed through the narrow valley, illustrated in Figure 20, and ultimately spread out into a fan where the creek valley entered an adjacent river valley. This movement involved an Excessive Travel Distance of

Figure 19. The Barrier

Figure 20. The Barrier and Rubble Creek Valley

a few km and other geomorphological data attest to its mobility. Details are given in the references noted above.

Large rock debris avalanches are catastrophic events that would destroy almost anything in the path of the main flow. It is seldom, if ever, practical to consider stabilizing such large rock masses to prevent instability where a rock debris avalanche is feared. Therefore it is of importance to be able to estimate their motion if some form of protection is sought either by zoning restrictions or the construction of protective structures. However, the mechanism whereby such large rock masses become mobile is not well understood. The extent of their travel can in some cases be estimated by invoking high pore pressures associated either with the presence of air or water. High pore pressure along the base of the flow does not adequately account for the internal fluidity of the rock debris. Fluidization due to an upward flow of entrapped air has been postulated, but a better explanation appears to be the reduction in internal shearing resistance due to momentum transfer between particles while the flow is in a somewhat dispersed state. A more detailed discussion of various mechanisms is given by Hsü (1975) and the Garibaldi Advisory Panel (1978). It is not the intention of this paper to enter into an evaluation of competing hypotheses to explain rock debris avalanches, but merely to draw attention to the research needed if their mobility is to be understood.

## CONCLUSION

Mobile soil and rock flows are encountered in variety of geological and geomorphological settings. In some instances mobility arises from the collapsing nature of the soil and mobility can be greatly reduced by compaction. In other instances an abundance of rainfall results in mobility of surficial deposits. Large volumes of soil or rock can also become fluidized by virtue of energy transfer mechanisms following instability, but the mechanics remain obscure. It appears that from an engineering point of view the motion of mobile flows and the design of protective structures should proceed using principles of fluid mechanics rather than the more common considerations of shearing resistance in soil and rocks. However, a considerable effort is needed to systematically classify soil and rock flows and to understand the processes whereby they become fluidized.

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