Slope failure along Lomsak-Chumpae Highway, Thailand

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Abstract

Field examination and limit equilibrium analyses have been performed to identify the failure characteristics of four rock slopes along Lomsak-Chumpae highway. The failure mechanisms are intricate due to the heterogeneity of the materials, the irregularity of the slope profile, and the fluctuation of groundwater in the rock mass and the overlying soil. Various combinations of the modes of failure have been found, e.g., plane and wedge slides, block toppling, and circular failure. Rapid weathering has been the cause for the initial and minor failures. These progressive failures normally associated with heavy rainfall. Such factors had not been taken into consideration in the original slope design and in the later stabilization schemes. As a result, inappropriate stabilization methods had been implemented, which subsequently contributed to the recent massive failures. A computerized expert system has been used to determine a new stabilization scheme. It recommends that further failure may be prevented by using fully grouted rock bolts, small opening wire mesh, and long drained pipes. Shotcrete should be avoided.

Keywords: slope failure, rock support, circular failure, erosion, water pressure, weathering

1. Introduction

Lomsak-Chumpae highway is an important strategic passage connecting the north and the northeast regions of Thailand. Constructed in the early 1970's, it is 105 km long, cutting through the steep terrain formed by limestone, siltstone and shale. Few years after construction, minor rock fall has been observed on at least ten locations along the highway. This is normally associated with heavy rainfall. Such instability has not been much of the problem in term of the safety [1]. The trenches and gabions have been effective in trapping the rock debris. In 1987 an attempt has been made by the Thai Department of Highway (DOH) to improve the stability of these slopes. Combinations of shotcrete, wire mesh and short drained pipes have been applied on the entire face of these slopes. For the past two years, massive failure has repeatedly occurred on four locations of the stabilized slopes. The failures brought down both the earthen and the supported materials. The remaining rock mass on the slopes also appears to be very unstable. The debris obstructed the traffic for several hours. The failure has posed severe economic impacts on the commercial and tourism of the regions. This paper presents an attempt at identifying the true mechanism governing the instability, offering the reasons for the ineffectiveness of the previous stabilization methods, and presenting new remedial methods to prevent further failure.

2. Slope History and Geology

Lomsak-Chumpae highway cuts through the steep terrain formed by heavily jointed limestone, clayey siltstone, slaty-shale, and weathered shale. The rock formations are part of the Paleozoic and Mesozoic strata of Khorat group [2]. The Paleozoic strata are exposed along the road as tight folding and faulting structures. The regional trend is in the north-south direction, with the regional bedding plane inclines to the east. The Mesozoic strata are subject to less tectonic activity than the Paleozoic strata. They have a relatively gentle dip angle to the east.

The highway was constructed in the early 1970's. The excavation was made primarily by the drill-and-blast method. The design was relied on the benching concept, which resulted in relatively steep and high slopes. The working slopes vary from 50 to 70 degrees, the bench height from 10 to 20 meters, and the total slope height from 10 to 70 meters. The slope toe is normally less than 1.5 meters from the edge of the pavement. This narrow gap later posed some difficulties in terms of the rock stabilization. A trench was excavated at the slope toe to catch the rock fragments and to drain the surface runoff. After excavation, the slope top was generally overlain by clayey soil. Thickness of the soil layers was about 2-5 meters. Few years after in services, minor rock fall had been observed particularly during and after heavy rainfall.

In 1984, Changsuwan [3] used the NGI tunneling quality index [4] to classify the rock mass forming the slopes along the highway. The objective of this geo-engineering classification is to assist in the stability evaluation, and the design of the rock support. It was reported that from km 17 to km 79, there were at least ten unstable rock slopes. The rock mass forming these slopes was classified as poor to very poor based on the quality index. The modes of failure included plane sliding and circular failure. The failure resulted in minor debris flow, which usually associated with the heavy rainfall.

In 1985, Wannakao et al. [1] investigated the rock slopes along the highway from km 18 to km 24 where the failures were frequent. It was reported that large-scale plane failure occurred at km 19. The bedding planes of the slaty-shale were nearly parallel to the slope faces, and were daylighted in some parts of the slope. This resulted in a progressive failure on the slope face. Circular failures were also found on the slopes between km 19 and km 21.5. It was concluded that small spacing and low shear strength of the rock joints caused by the weathering and erosion processes contributed to the observed circular failure. These investigators also concluded that even though most of the failures were progressive, the failure processes appear to be only minor rock fall and relatively small-scale debris flow. The trenches and gabion had been effective in trapping the rock debris. The rock fragments rolling onto the pavement were quickly removed by the on-site workers.

In 1987, an attempt was made by the Department of Highway (DOH) to improve the stability of several slopes along the highway. Shotcrete (5-7 cm thick) and wire mesh were vastly applied onto the entire slope face for at least ten locations along the highway. Short drained pipes, made of 50 cm long perforated PVC, were installed on the slope face to drain water behind the shotcrete. The drained pipes had a square pattern with an average spacing of 1.5 meters.

Since the year 2000, massive failure had repeatedly occurred on some slopes where the shotcrete has been installed. Four of these slopes failed within weeks apart. The failures brought down both the earthen and the supported materials. The remaining rock mass on the slopes also appears to be very unstable. Until then, through the service life of the highway, such massive failure had never occurred. Figures 1 and 2 show examples of the remaining slope faces after the massive failure occurred.

3. Rock Slope Characteristics

In this study, field investigation has been conducted on four unstable rock slopes along the highway to understand their recent massive failure. The collected data include slope geometry, joint conditions and orientation, intact rock strength, performance of the existing rock supports (if any), modes of failure, and groundwater conditions. The study criteria follow as much as practical the methods suggested by the International Society of Rock Mechanics (ISRM) [5]. Even though the rock mass characteristics of these locations are relatively different, their cause for the instability is similar. The failure process is relatively unique, and deserves a close attention. The knowledge obtained here can also be used as a case example for the slope stabilization practices in the similar engineering and geological environments. Figure 3 shows the locations (indicated as A, B, C and D) of these four slopes along the highway.

Figure 1 Massive failure occurred after the installation of shotcrete. Failure surface cuts from the upper slope through the mid-height of the slope face. Part of shotcrete remains at the lower slope face.

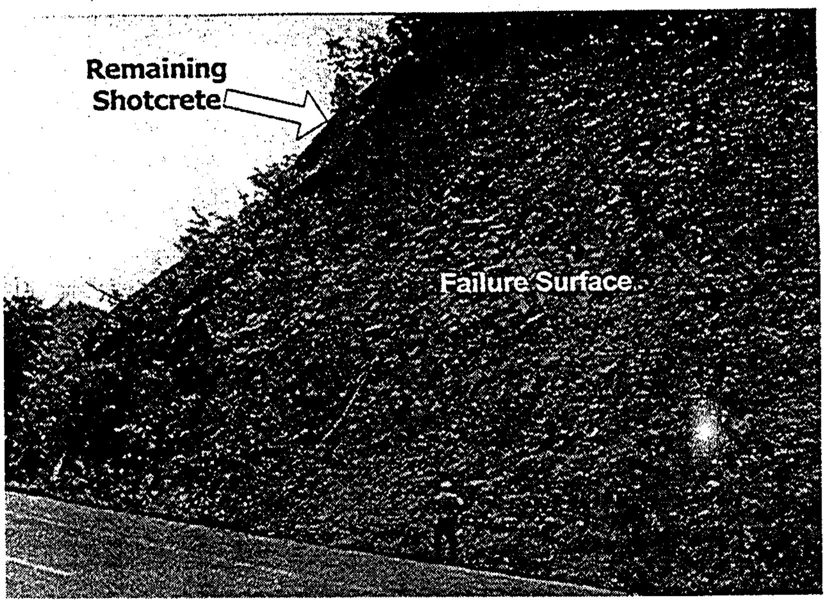

Figure 2 Failure surface appeared after massive slope failure. The failure brought down both earthen and installed materials. The remaining shotcrete appears on the left side of the slope face.

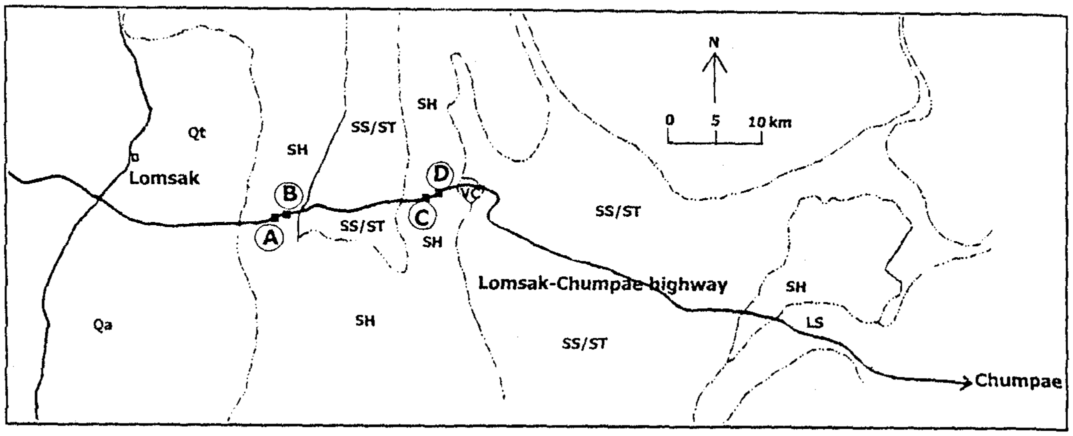

Figure 3 Locations of the studied slopes (A, B, C and D) along Lomsak –Chumpae highway. SH = Shale, LS = Limestone, SS/ST = Sandstone-Siltstone Interbed.

Slope A locates at km 20.5. The orientation of the slope face is (strike/dip angle) 225/60. The height is 30 m and length is 80 m. The slope is formed by heavily fractured shale. There are three joint sets with the following orientations (strike/dip angle); 075/65, 203/48 and 350/42. The average joint spacing is 15 cm. Persistence of the rock discontinuities is 100%. The joint roughness coefficient (JRC) is 1 (smooth planar). The width of the apertures is 0.1-1 cm (moderately wide to wide). They are filled with soft clay. The uniaxial compressive strength of the intact (field estimation) is 25-50 MPa.

Slope B locates at km 21. The orientation of the slope face is 55/54. The height is 16 m and length is 40 m. The rock mass is weathering shale. There are three joint sets with the following orientations; 220/43, 092/69 and 318/81. The average joint spacing is 8 cm. Persistence of the rock discontinuities is 80-100% with JRC = 1. The joint apertures are 0.1-1.5 cm (moderately wide to extremely wide) and are filled with clay. The uniaxial compressive strength of the shale is approximately 25 MPa.

Slope C locates at km 36. The rock is highly weathered shale. The slope profile is very irregular with the average orientation of 321/50. It is 20 meters high, and 50 meters in length. There are three joint sets: 293/55, 023/70 and 318/80. The average joint spacing is 15 cm. Most joints are tight with JRC = 1-3. Persistence of rock discontinuities is 100%. The estimate compressive strength is 25 MPa. Slope D locates at km 37. It is 15 meters high and 25 meters long. The slope orientation is 303/55. The rock mass characteristics are virtually identical to those of the slope C.

4. Initial Instability

Due to the irregularity of the slope profile, closed form solutions, such as those given by Hoek and Bray [6], can not be strictly applied in the stability evaluation for these slopes. The use of computer modeling to quantitatively determine the failure behavior may not be appropriate because of the heterogeneity of the rock mass and the unknown magnitude and distribution of the groundwater pressures. From the results of the field examination, limit equilibrium analyses have been performed for the preliminary evaluation of the slope stability. Figure 4 shows the stereographic projections of the discontinuities as compared to the slope orientation for these slopes. The projection of each joint orientation represents the average of nearly 40 measurements.

The orientations of the representative plane of the discontinuities (Figure 3) indicate that these slopes are not stable. Prior to the installation of any rock support, plane and wedge slides likely occur for all slopes. For Slope D, block toppling is also possible. This interpretation is supported by the results from the field observations that piles of rock fragments with an average size of 8x10x10 cm have been found at the slope toes. This minor failure is likely initiated by the surface runoff and groundwater pressure. This is evidenced by the fact that during the dry seasons all slopes remain stable; no rock fall has been observed. The weathering and erosion have significantly reduced the intact rock strength and the joint shear strength. The rock joints with very small spacing and low shear strength also increase the possibility of circular failure for all slopes, particularly during and after heavy rainfall. Since the rock mass has relatively high permeability (heavily fractured), the groundwater can freely seep out from the slope face. Subsequently, the water pressure within the slopes had never reached a critical point that a massive circular failure could occur.

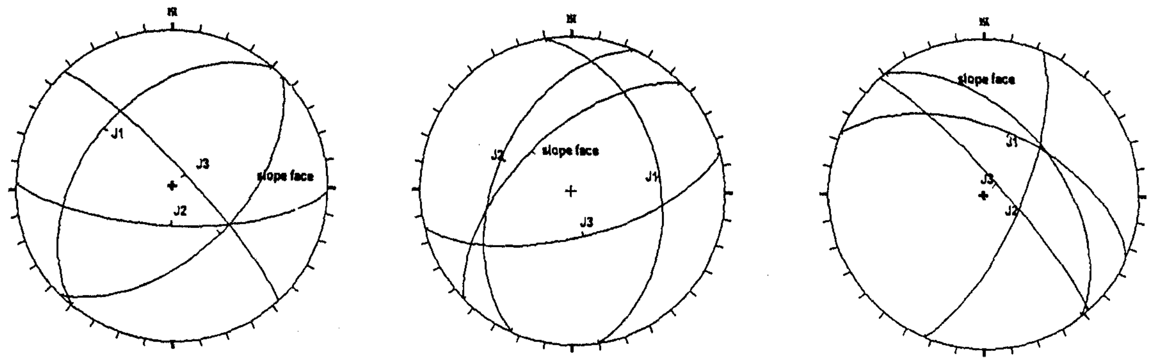

Figure 4 Stereographic projections of slope face and discontinuities (J1, J2, J3) for slope A (left), slope B (Middle) and slopes C & D (right).

5. Causes of Massive Failure

Figures 5 through 8 show the cross-sections of the massive failure for Slopes A, B, C & D, respectively. The common cause and the sequence of the failure for these slopes are that the full-face shotcrete with inappropriate drained pipes prevented the infiltrated water from seeping out of the slope. The shotcrete was not effective in preventing the rock fragments from dislodging from the slope face. It is observed that slabs of shotcrete in many areas were detached from the surface of the weathered rocks. The drained pipes were poorly designed. They were too short and were not effective in reducing the pore pressure. No water flowed from any of those pipes during the rainfall. No water stain was observed below the pipes during the dry seasons. The permeability increase allowed groundwater to infiltrate even deeper into the rock mass behind the slope face, and eventually increased the pore pressure. Prior to the massive failure, the groundwater table nearly reached the upper slope face as indicated by the water stain near the cracks in the shotcrete. The failures were sudden. The failure surface on these slopes was a combination of plane and wedge sliding and circular failure. The sliding occurred on the top and the middle, and the circular occurs at the bottom of the failed surface. It is interesting to note that the failures of these four locations had occurred within weeks apart.

6. Recommendations on Stabilization Methods

A computerized expert system, ROSES program [7] is used to design the stabilization methods for the present four rock slopes. The slope geometry, rock mass characteristics, existing rock supports, previous failure, and groundwater conditions (described above) are used as data input. The program also considers the level of safety required by the structures. The recommendations can be summarized as follows. A square pattern of fully grouted rock bolts with 2 meters spacing is required on the slope face. They should be installed in horizontal and normal to the slope face. The minimum bolt length is 4 meters. Resin should be used as grouting material in the holes. The bolts should be used with 5 cm opening wire mesh or chain link. The mesh should be galvanized. A series of drained pipes should be installed horizontally with 2 meters spacing. The drained pipes should have a minimum length of 15 meters, and are fully perforated. Shotcrete should be avoided. The overlying soil should be removed to at least 10 meters from the slope face.

7. Conclusions

The objectives of this paper are to identify the true mechanism governing the instability of four rock slopes along Lomsak-Chumpae highway, to offer the reasons for the ineffectiveness of the previous stabilization methods, and to propose new remedial methods to prevent further failure. Detailed field examination and limit equilibrium analyses have been performed on four unstable slopes along the highway. The results indicate that the failure mechanisms are intricate due to the heterogeneity of the materials, the irregularity of the slope profile, and the fluctuation of groundwater in the rock mass and the overlying soil. Various combinations of several modes of failure have been found, e.g., plane and wedge slides, block toppling, and circular failure. Soon after the construction, rapid weathering and surface runoff have been the main factors for the initial and minor failures. The occurrence of these progressive failures usually associates with heavy rainfall. The weathering, erosion and groundwater pressure have not been taken into consideration in the original slope design and in the later

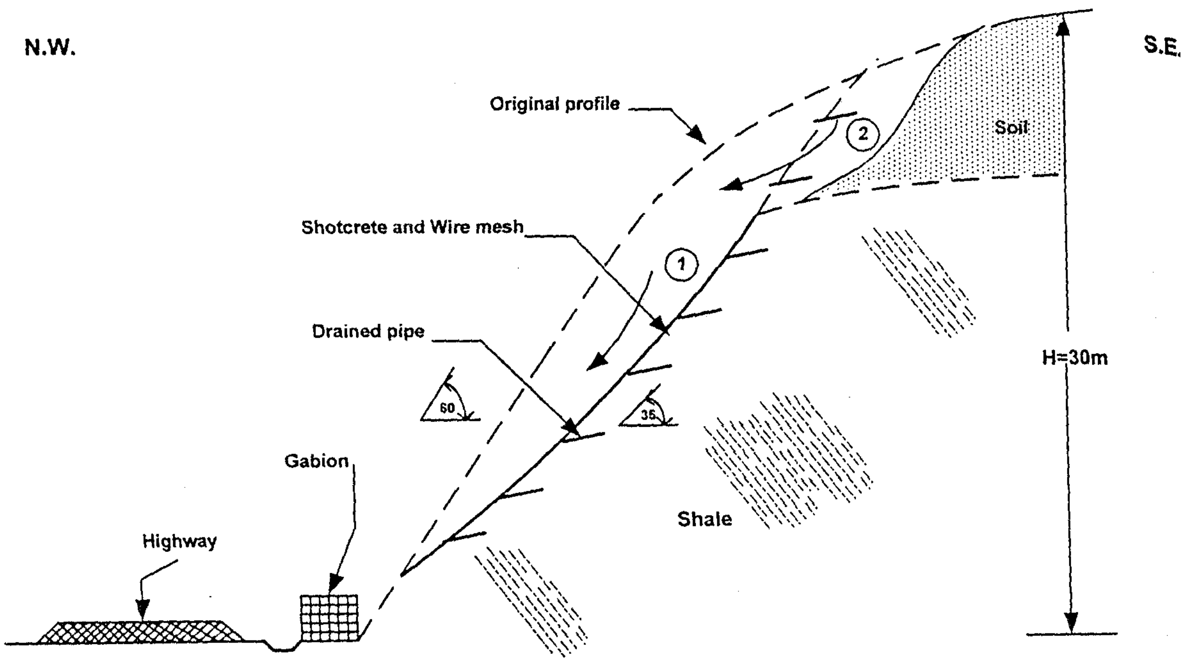

Figure 5 Cross-section of the failing rock slope at location A.

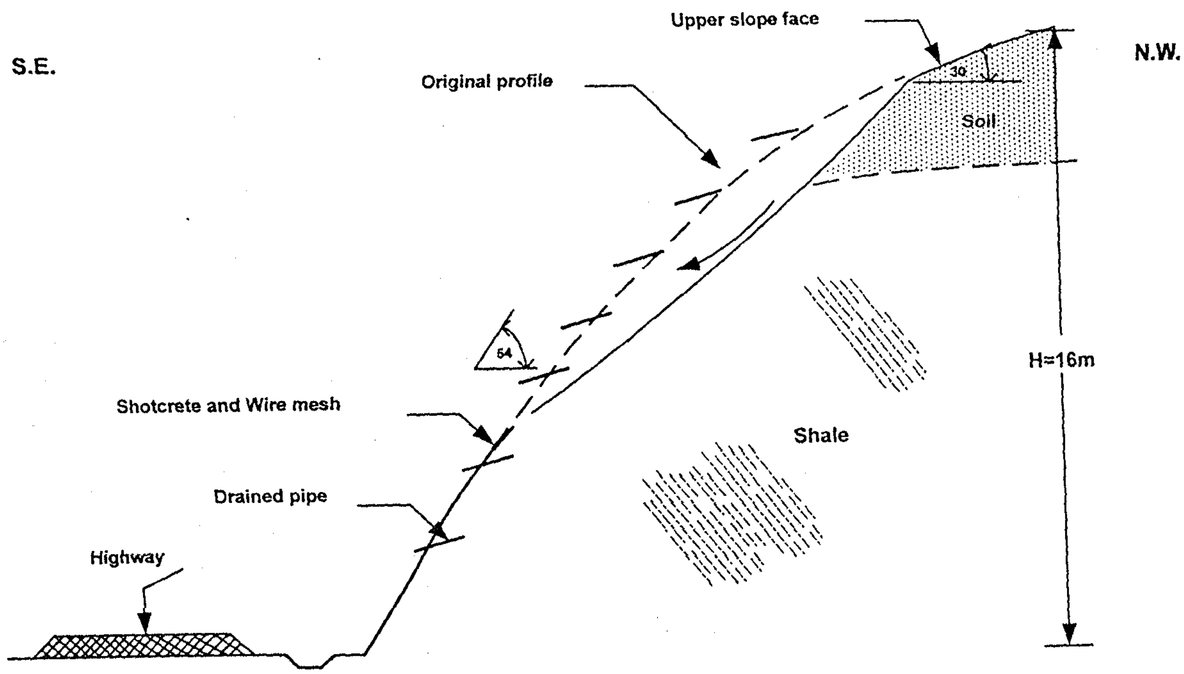

Figure 6 Cross-section of the failing rock slope at location B.

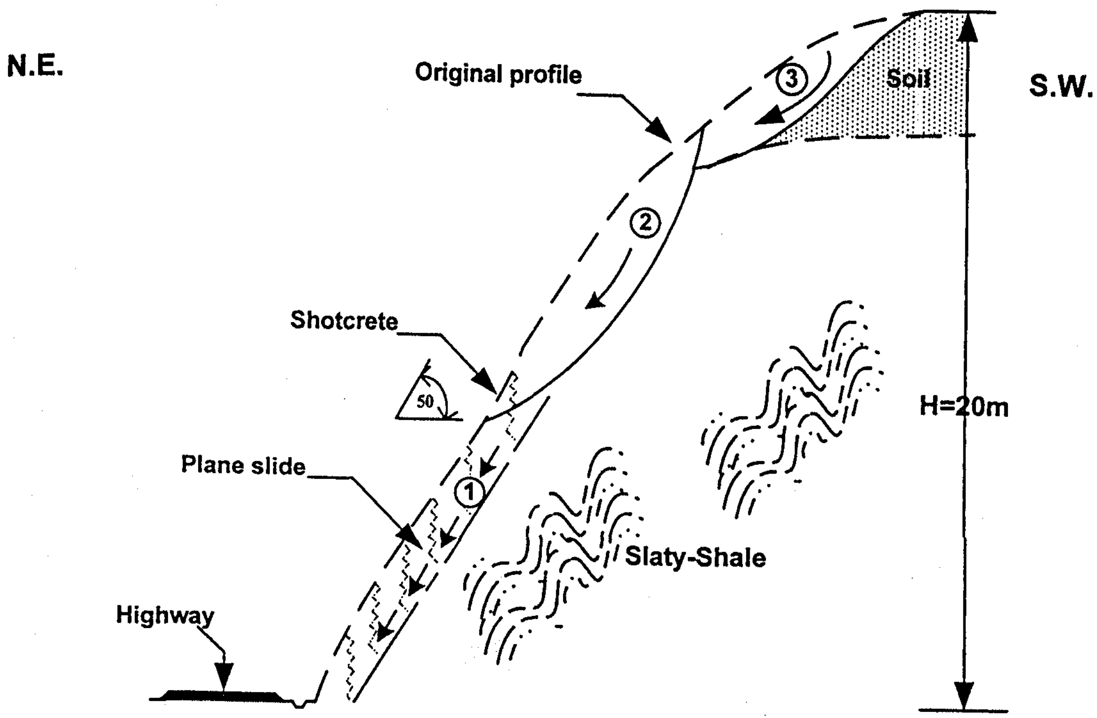

Figure 7 Cross-section of the failing rock slope at location C.

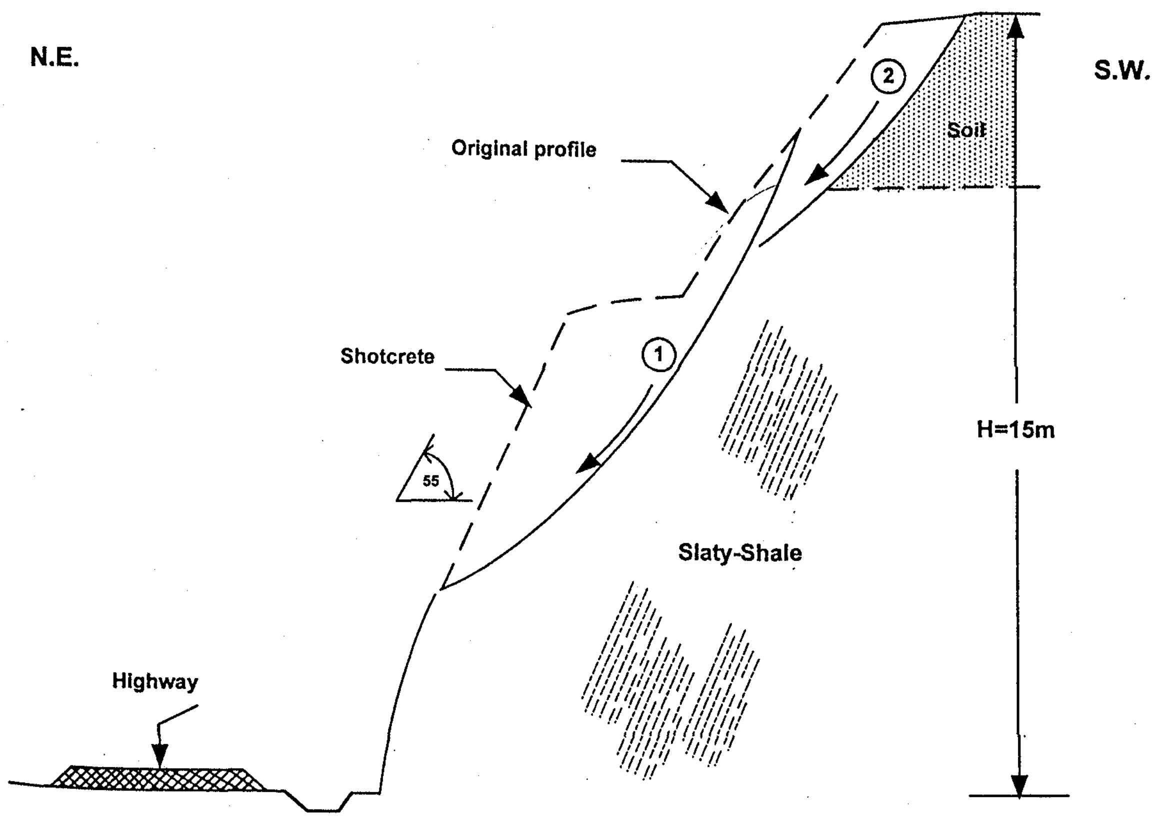

Figure 8 Cross-section of the failing rock slope at location D.

stabilization schemes. The application of the full-face shotcrete with poor drainage system likely contributes to the subsequent massive failure of these slopes. The shotcrete causes the water pressure to build up behind the slope face and subsequently induces the slides of rock mass and the upper soil. It seems that application of the shotcrete in this case is not only ineffective, but also inappropriate. A computerized expert system has been used in this study to determine a new stabilization scheme. It recommends that further failure may be prevented by using fully grouted rock bolts, small opening wire mesh, and long drained pipes. Shotcrete should be avoided.

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