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A STUDY ON THE CAUSES OF SOME ROCK SLOPE FAILURES ALONG HIGHWAYS IN TAIWAN

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SUMMARY Geographically, the terrain of the island of Taiwan is mountainous, many highways cut across hillsides of soft rock formation. In the last few years, numerous rock slope failures occurred along highways causing serious obstruction to traffic and sometimes loss of life. The causes of failures can be grouped as:

(1) types of geological formations

(2) heavy rainstorms inducing high pore water pressures in the soil or rock formations

(3) reduction of strength and swelling of soft rocks due to adsorption of water

(4) weathering of unprotected or inadequately protected slope surface

(5) improper design and/or construction

Generally, a slope failure may be attributed to one or a combination of several of the above mentioned causes. This paper presents a detailed discussion on these causes of failure with particular emphasis on the sandstone-shale formation commonly occurred in Taiwan. Several case studies of rock slope sliding are reported.

INTRODUCTION

Geographically, the terrain of the island of Taiwan can be classified as mountainous. Only about one-third of the total area are plains with elevations below 100 m above the mean sea level. The rest two-thirds of the island are occupied by hills and mountains with elevations varying from 100 m to 4,000 m. Generally speaking, the topograph is rather steep and rugged. With the rapid economic growth and population increase on the island in recent years, activities and developments are inevitably extended to slopelands and hilly areas. Landslides and slope stability become one of the most important problems in many civil engineering work.

Since the completion and opening to traffic of the National North-South Freeway in 1978, several slope failures have occurred in cut areas along the northern part of the Freeway, particularly in the section between Keelung and Neihu, which have caused serious obstructions to the traffic. Moh and Associates has been engaged by the Freeway Bureau to investigate several of the major slope failures and to evaluate the stability and remedial measures for some of the potentially dangerous slopes. The paper discusses the main causes of these slope failures, illustrated with several case studies.

GENERAL GEOLOGICAL CONDITIONS OF TAIWAN

The topograph of Taiwan is greatly affected by its geological condition. The island can be broadly divided into three major geological provinces, i.e. the Central Mountain range, the Western Foothills and the Coastal regions as shown in Fig. 1. The Western Foothills is the most important

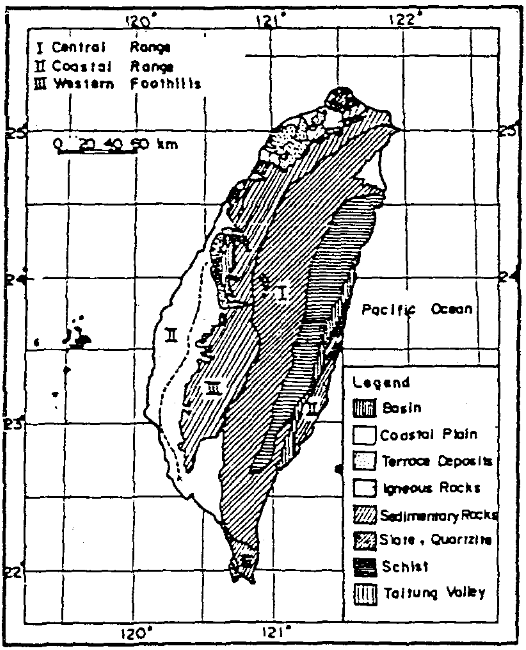

Fig. 1 Geological Provinces of Taiwan

geological province where majority of the development and population of the island are concentrated. This Province has flat coastal plains on the very Western Coast and the elevations of the foothills vary between 100 m to 600 m. The geological formation in the foothill area are composed of Neogene clastic rocks with interbedded sandstone and shale as the primary rock formation. There are occasional thin layers of clays and siltstones. The thickness of the Tertiary deposit increases in a north to south direction and composition of the formation also changes in a southernly direction with more shale and fine particle content. In the southern part of the island, the major rock formation becomes mudstone. Within this Province there were numerous folds and faults due to past tectonic movements. Since the interbedded sandstone-shale formation in this Province occupies about one-fourth of the total land arèa of the island and since major part of the North-South Freeway passes through this region, this paper concentrates its discussion on stability problems of the sandstone-shale slopes.

FACTORS AFFECTING THE STABILITY OF SANDSTONE-SHALE SLOPES

Geological Characteristics of Sandstone-Shale Formation

The sandstone-shale formation in the northern part of Taiwan is of the Miocene age. According to Lin and Chow(1974), the thickness of the Miocene stratum in Taiwan increases gradually from about 3,000 m along the northwest coast as it is towards southeastern and southern parts of the island. The proportion of the shale in the formation increases in that direction whilst the particle size of the sandstone tends to become finer. In the northern part, the sandstones are composed of medium to fine sand particles cemented together by calcium deposits and/or clays. These sandstones generally have rather low strength and can be considered as easily disintegrated soft rock. The shales are of the soil-like type. They derive their cohesion from consolidation under the overburden pressure and generally do not contain strong cementitious materials. The shales contain large amount of silt-size particles, often exceeding 50%, and are of low plasticity. The major clay minerals in the shale are illites and chlorities.

In the Western Foothill region where most of the slopes are composed of sandstone-shale formation, many faults are also concentrated. Due to activities of these faults in the past, numerous joints and fractures were formed in the sandstone-shale strata. Tectonic movement of the earth crust has further caused the strata to be inclined at an average slope between 15° and 30° The slopes of these interbedded formation are either dip slopes or escarpment.

Engineering Characteristics of Sandstone-Shale Formation

Table 1. list the strength and permeability values of rock cores obtained from eight side slopes along the Freeway. The strength properties of sandstone formation are greatly affected by the amount and type of cementitious materials contained in the rock. The cementing agent present in the sandstone strata in the northern part of Taiwan is usually calcium carbonate and its amount is relatively low. Consequently, the uniaxial compressive

strengths of intact specimens of the sandstone are quite low. These sandstone formations are classified as "Soft" rock according to the system proposed by HOEK and BRAY(1977). Furthermore, the strength of the sandstone decreases with increase in the degree of saturation due to softening of the cementitious material. For sandstones with weak cementing bond, the rock mass would even disintegrate upon contact with water. For those sandstone blocks broken off along joints or bedding planes, the strength values are even lower.

The shear strength values of the shale formation depend upon the clay content and the degree of cementation. As shown in Table 1, the uniaxial compressive strength of water soaked shale specimen is only about 2-65 kg/cm^2 which indicates that the shale belongs to the category of "very soft" rock. Many of the shale specimens completely disintegrated during the soaking process.

Table 1. Engineering Properties of Sandstone, Shale and Siltstone Formation

Location	Sandstone		
	Friction angle of joint surface ϕ, deg.	Uniaxial compression strength kg/cm^2	Coef. of permeability k, cm/sec
128K	26.0 - 31.5	0.1 - 13.0	8.6×10^{-4} - 3.2×10^{-6}
16KN	-	42.3 - 129.6	2.7×10^{-6} - 3.4×10^{-8}
19KN	26.5 - 29.0	35.3 - 225.2	1.1×10^{-5} - 8.1×10^{-9}
20K+700N	28.0	112.4 - 169.6	4.6×10^{-9} - 9.9×10^{-9}
22KN	28.0 - 31.0	2.1 - 443.0	2.8×10^{-8} - 2.7×10^{-9}
Patu	28.0 - 31.0	35.4 - 206.3	1.1×10^{-6} - 4×10^{-9}
Shale			
128K	25.5	0.2 - 12.0	4.3×10^{-6} - 8.8×10^{-7}
14KN	19.0	2.0 - 30.1	1.5×10^{-8} - 4.3×10^{-9}
20K+700N	28.0 - 32.0	3.0	-
22K+200N	28.5	65.7	-
Siltstone			
19KN	25.0 - 28.0	30.8 - 141.3	-
19K+800N	28.5 - 32.0	26.4 - 242.2	-
20K+700N	28.0 - 30.0	88.7 - 230.9	1.3×10^{-8} - 2×10^{-9}

For either sandstone or shale, the bedding planes or joint surfaces are usually the weak planes where most of the slope failures and slides occur. It is rather difficult to accurately determine the strength characteristics along these planes. The values of the angle of shearing resistance listed in Table 1. were determined by direct shear tests along artificially cut surfaces of intact rock specimens to similate bedding planes.

Within the joints and between bedding planes of the sandstone-shale formation, joint-fill materials are often found. These materials are usually composed of very fine particles which will become extremely soft upon contact with percolating or seepage water. Laboratory tests performed on one of the joint fill material obtained from a side slope near Taipei show the following properties: Liquid Limit 133%, Plasticity Index 35%, natural water content 118%, amount passing No. 200 sieve 96% and total angle of shear resistance $\phi = 10^{o}$. It is obvious that if there are joint-fill materials existing in an interbedded rock formation, the strength values of this formation will greatly decrease and consequently the stability of slopes composed of these types of formation will be reduced.

The permeability characteristics of sandstones and shale are quite different. Results of laboratory tests performed on intact rock specimens indicate that the coefficient of permeability of the sandstone varies from 8.4×10^{-4} to 4×10^{-9} cm/sec and that for the shale varies from 4.3×10^{-6} to 4.3×10^{-9} cm/sec. The shale formation belongs to low permeability material whilst the permeability characteristics of the sandstone varies considerably. Comparing with sandstone from other regions (HOEK and BRAY, 1977), the sandstones in Taiwan belong to "low permeability" to "permeable". However, the actual permeability of the sandstone-shale formation in the field is expected to be even higher than the laboratory determined values due to presence of numerous joints and bedding planes in the rock formation.

Rainfall Conditions

Taiwan is under the influence of southwest seasonal wind and tropical low pressure during the summer, and is often attacked, directly or indirectly, by typhoons. Therefore there are plenty of rainfalls on the island. From Fig. 2 which shows the iso-rainfall lines (WU and CHEN, 1977), it is obvious that in major parts of the island, the annual rainfall exceeds 2,000 mm. On the average, there are nine months having rainfall exceeding 100 mm. The average rainfall in the month of June reaches as much as 300 mm. On the basis of evaluation of rainfall data accumulated during the past twenty years, WU and CHEN (1977) concluded that the heavy rainfalls in the northern part of Taiwan are caused mainly by typhoons, stationary front, tropical depressions, southeast monsoons and low pressure depressions. Generally, the amount of rainfall in the mountainous areas is much larger than that in the plains, due to the effect of terrain and causes of rainfalls.

There are a number of problems related to slope stability which can develop due to rainfall, particularly heavy and continuous rains. They include:

(1) increases of the in situ pore water pressure, as illustrated in Fig. 3,

(2) fill-in of the joints and cracks with water which in turn increases the active force, and

(3) swelling, weathering and erosion of the slope surfaces due to adsorption of water.

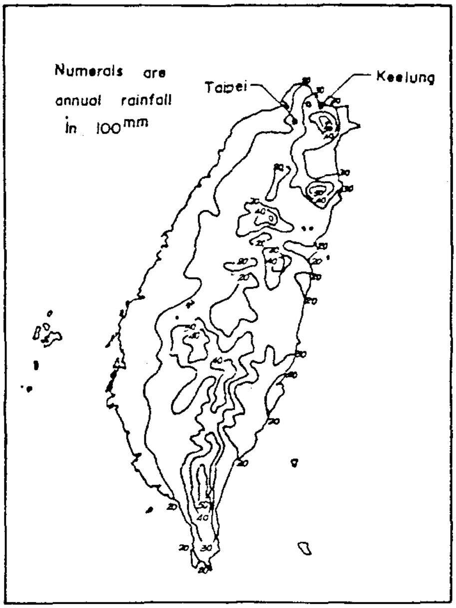

Fig. 2 Iso-rainfall Lines in Taiwan

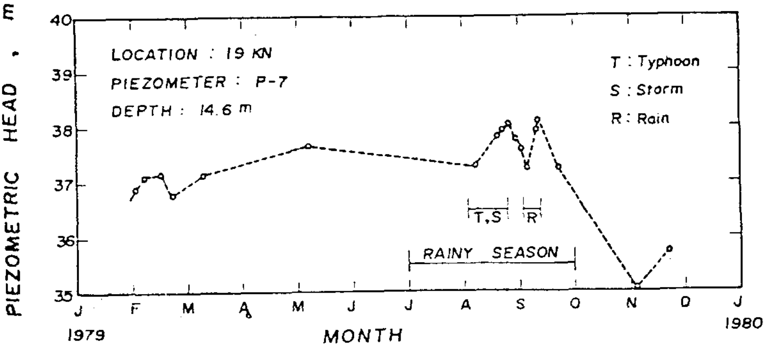

Fig. 3 Variation of Piezometric Head with Time

Other Factors

Other factors which often contribute to slope failure include improper or inadequate design and/or construction. Due to either insufficient predesign information or reasons of false economy, drainage facilities, in particular subsurface drainage, are usually inadequate. In some cases even the surface drainage system only involves sparsely spaced ditches without lining and appropriate connections.

For slope protection, concrete-grass grids, i.e. concrete grids with grass planted inside, are often used. This type of surface protection is also used along the Freeway. In many places, it gives successful protection but in many other locations, the grids became a source of instability. In areas where the rock formation is easily weathered or where the rock has many cracks and joints, the grids become traps for water which seeps through the joints and bedding planes or is adsorpted by the underlying water-sensitive shaly rock.

MODES OF FAILURE OF SANDSTONE-SHALE SLOPES

The modes of failure of the interbedded sandstone-shale formation in Taiwan can be classified into two major categories, i.e. plane failure and wedge failure, Figure 4(a), shows the conditions for a plane failure. For a dip slope of interbedded sandstone-shale, if the cut slope at the toe of the dip slope is steeper than the angle of inclination of the dip, the upper rock block has a tendency to slide downward. This tendency of slide must be resisted by the shearing strength along the bedding planes of the rock. If the cut slope is steeper than the angle of shearing resistance and there is no cohesion between the beddings, slide would occur. On the other hand, even if the inclination angle of the cut slope is smaller than the angle of shearing resistance, but there is excess pore water pressure in the joints, slide failure may also occur under the influence of the additional external forces.

When there exists two sets of discontinuous planes, such as joints or beddings, etc., in the sandstone-shale formation, the block above these discontinuous planes may slide along the intersecting planes, as illustrated in Fig. 4(b). This type of failure is the so-called wedge failure. On a cut slope, the necessary condition for a wedge type of slope failure is that the angle of the cut slope must be larger than the angle of intersection of the two discontinuous planes. The weight of the wedge shaped rock block is resisted by the shearing resistance developed along the discontinuous planes. In the sandstone-shale formation, the most commonly seen wedge failures involve the sliding of sandstone blocks along the beddings of shale or joints of the sandstone.

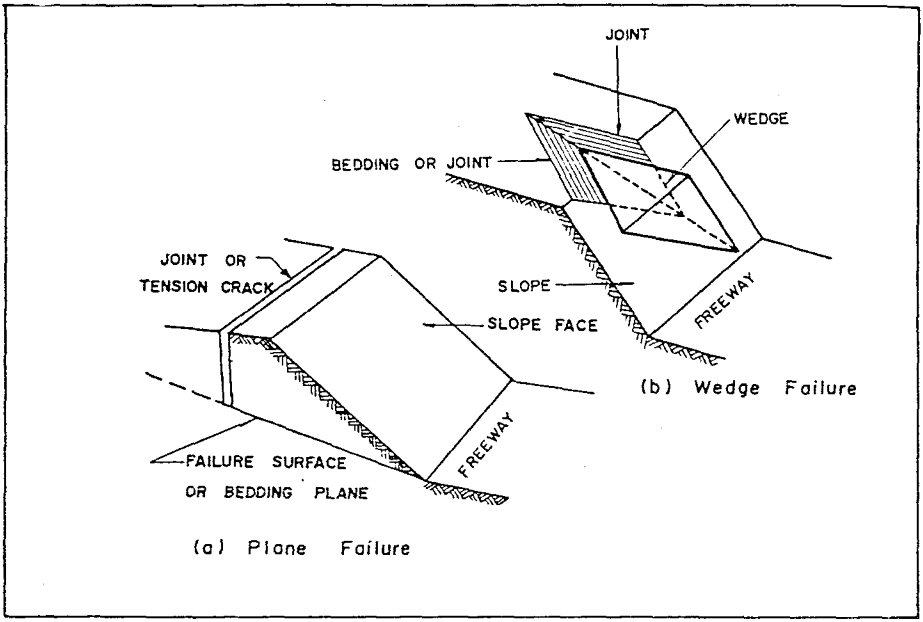

Fig. 4 Modes of Failure in Sandstone-shale Formation

CASE STUDY OF SLOPE FAILURES

Among the many slope failures in the sandstone-shale formation in northern Taiwan, the failures or slope movements along the N-S Freeway appear to be most typical. The following sections report three of the case studies.

Case One

On 23 September 1977, a major slide occurred at the northside of the cutslope at Station 16K + 600N of the N-S Freeway. The slide occurred after a period of continuous heavy rains. The slide covers an area approximately 285 m long and 107 m wide. The total amount of earth moved was about 130,000 cu. m. which have blocked one lane of the Freeway.

The rock formation in this area is composed of interstratified gray fine-grained sandstone and clayey shale. The direction of the rock formation is at strike N58°E with dip angle of 25°E. This means that the dip of the bedding plane is towards the Freeway. Sliding of the rock mass occurred along these bedding planes. The original slope in this area was about 25° which is parallel to the bedding planes. Due to construction of the Freeway, the slope was cut back to 45°. The slope failed during the first rainy season after the construction. As shown in Fig. 5, there were two sets of

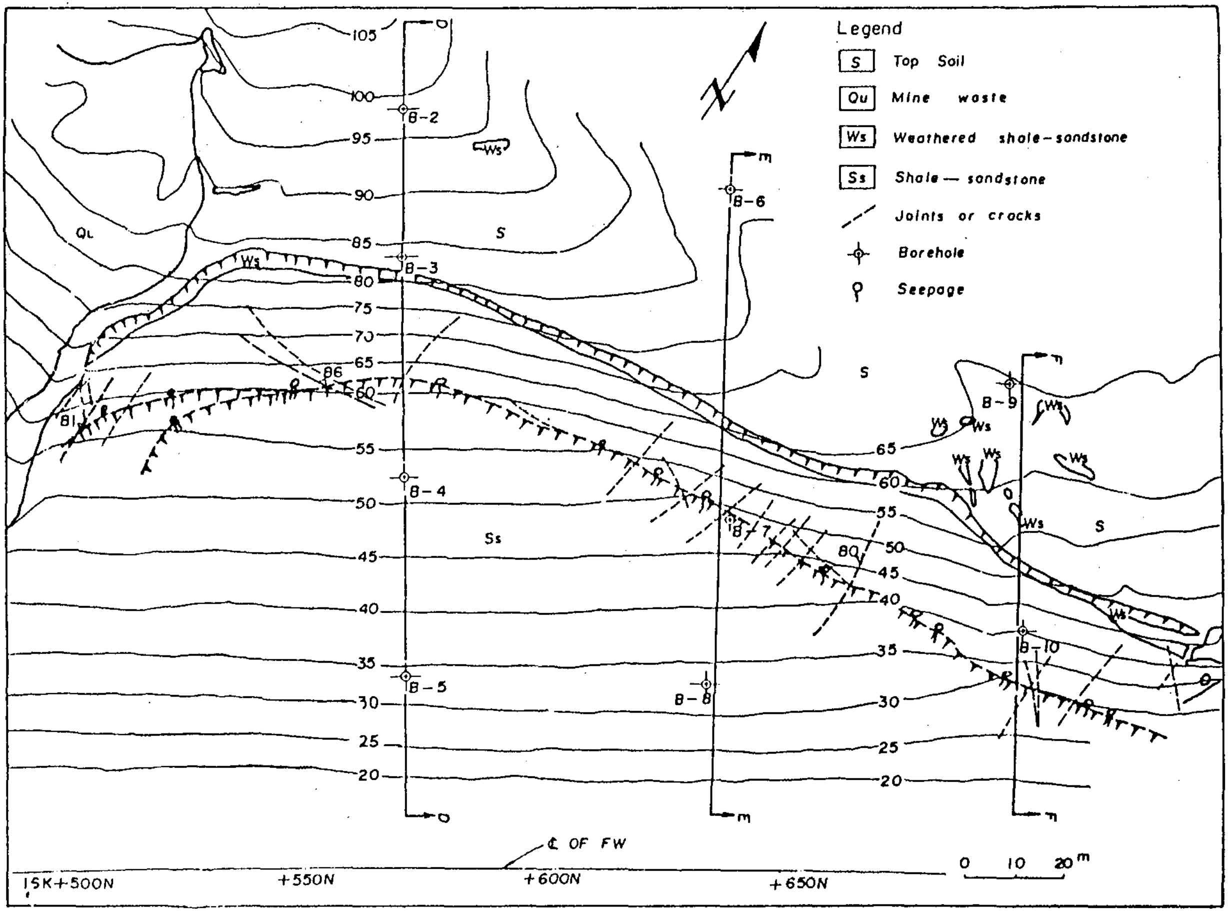

Fig. 5 Geological Map of 16 KN

joints intersecting at right angles with each other present near the failure surface. These joints have dip angle between 80^{o} to 85^{o} nearly perpendicular to the horizontal surface. It was also found that groundwater continuously seeped out along the bedding planes in the zone between the remaining slope and the failure surface. In consideration of the overall safety of the entire slope, a thorough investigation was undertaken to delineate the causes of the slide and to determine the necessary remedial actions. A total of 10 vertical boreholes was drilled to depths varying from 10 m to 40 m. Triple-tube core barrels were used to recover the rock samples. After completion of drilling, hydraulic type piezometers were installed at different depth to monitor the underground pore water pressures. In addition, an engineering geological map was prepared which gave detailed description of the surface geology of the entire slope including rock formation types, patterns, joints, fractures and etc. Two types of laboratory tests were carried out on the rock cores. They were shear strength test along the bedding planes of the rock and water affinity tests. The latter included slaking tests, water adsorption and permeability tests. Results of field exploration and laboratory tests show that in this area seepage water flows much more easier along the bedding planes of the rock formation. The sandstone-shale is sensitive to water. It adsorbs water easily and develops surface cracks which tend to disintegrate. Swelling of this type of rock can amount to 5 - 15% of the total volume. From these information, it can be easily concluded that the rock formation in this area cannot be allowed to be exposed. They must be properly protected from weathering.

Figure 6 shows a cross-section of the failure surface. Assuming that the pore water pressure acting on the failure surface at time of failure is equivalent to 2/3 of the height of the slope, it can be back figured that the strength parameters of the rock formations at time of failure were $c = 0.1\ kg/cm^2$ and $\phi_r = 32^{o}$. By using the in-situ measured pore water pressure values, it was found that the factor of safety of the remaining slope was less than 1.5 in most of the sections. It was thus concluded that the remedial stabilizing measure should include two aspects. They were: reducing the pore water pressure in the rock by installing underground drains, and the use of rock anchors to stabilize the rock formation.

Underground drains were essentially perforated pipes installed deep into the rock formation for the purpose of intercepting seepage water trapped between bedding planes. Prestressed cables were used as rock anchors. The anchors were fixed on 60 cm x 60 cm reinforced concrete cross beams placed on the rock slope. As discussed in the previous paragraphs, the rock formation in this area is sensitive to weathering and erosion, the exposed slope must therefore be properly protected. After cleaning to the fresh rock surface, shotcrete, 5 cm in thickness, was sprayed onto the exposed surface. In order to reduce tension cracks, steel wire mesh was placed in the shotcrete. Figure 7 shows the stabilized slope two years after the construction.

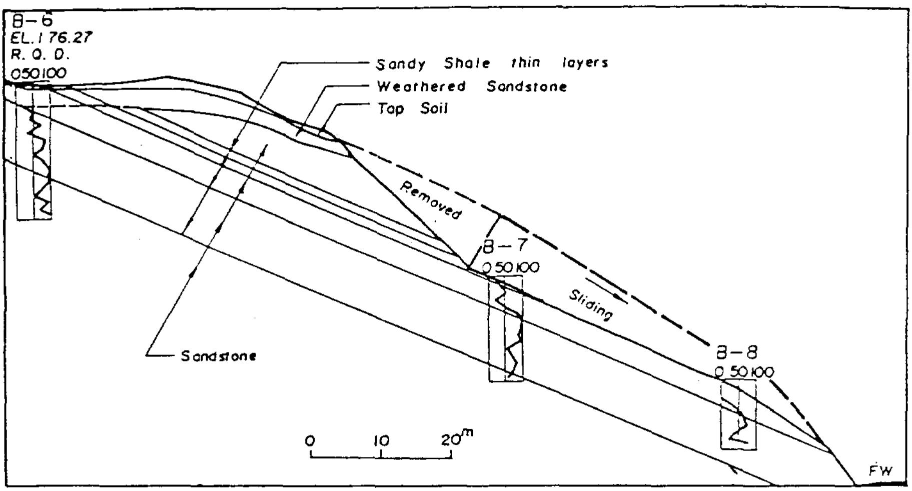

Fig. 6 A Cross-Section of Failure Slope at 16 KN

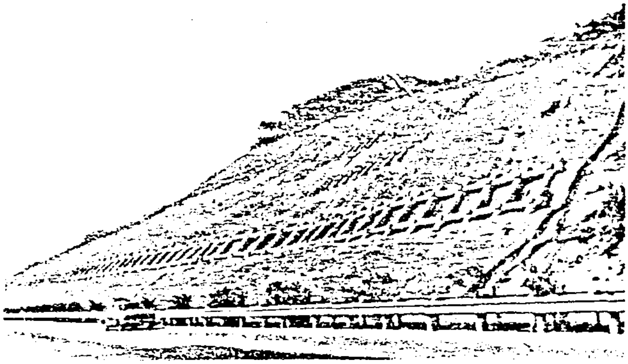

Fig. 7 Stabilized Slope at 16 KN

Case Two

At Station 128K of the N-S Freeway, there is a long cut slope approximately 450 m in length and 85 m high. The slope surface was not protected. Due to exposure to rain water and weathering action, a large slide occurred in 1978. The original slope was at an angle of 20° to the horizontal. After the cut, the 85 m high slope is divided into 10 steps with berms at each step. The height of each step is 8 m with a side slope of 1 on 1.8 except that the top two steps have side slope of 1 on 2. At each berm, horizontal drains and side intercepting drains were constructed. The total amount of material removed from the cut was about 520,000 cu. m. The slide occurred in the middle of the cut area started from the edge of the 9th berm towards the Freeway. The sliding mass was bounded by bedding planes and joint surfaces and was in a wedged shape as shown in Fig. 8. The total amount of sliding mass was about 25,000 cu. m.

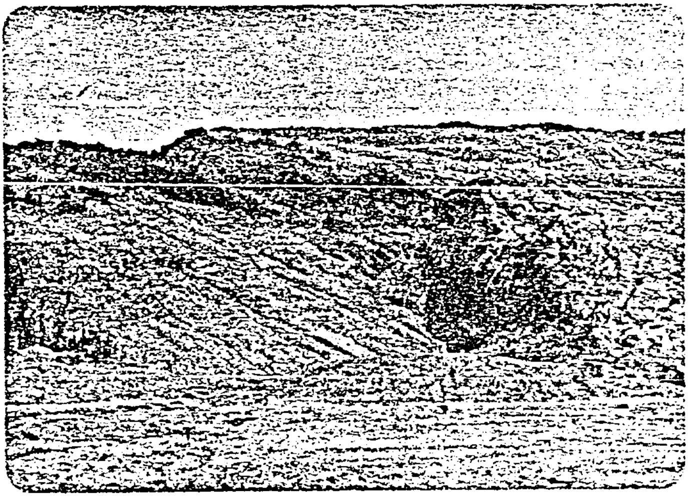

Fig. 8 Slide at Station 128 K

The rock formation in this area belongs to the upper strata of the Cholan Formation of the neogene pliocene era. They are mostly alternative layers of sandstone and siltstone-shale. The sandstones are generally loosely cemented and soft which are easily eroded by water. The shale formation is sensitive to water, becomes soft and swellable upon contact with water. Swelling may reach as high as 23%. Upon weathering, the shale will gradually disintegrate in an onion-like shape or form cracks and fractures. The direction of the beddings is generally between N5°W to N25°W with dip angle of 20°SW. This forms a monocline type of geological structure.

By combining the bedding planes, joint surfaces of the rock formation and the cut slope according to space geometry, a wedged shape failure was obtained. During the investigation, several sets of joints were discovered on the slope outside the sliding area. These joints have strike at N83^{o}E with dip of 53^{o}NW, and the joints extended for 5 to 10 m in length. The directions of these joints are very similar so those found in the sliding area. In areas near the slide, joints with openings of 2 to 3 cm were also found. All these observations indicate that the safety of the entire slope was in question. Figure 9 shows a geological map of the sliding area.

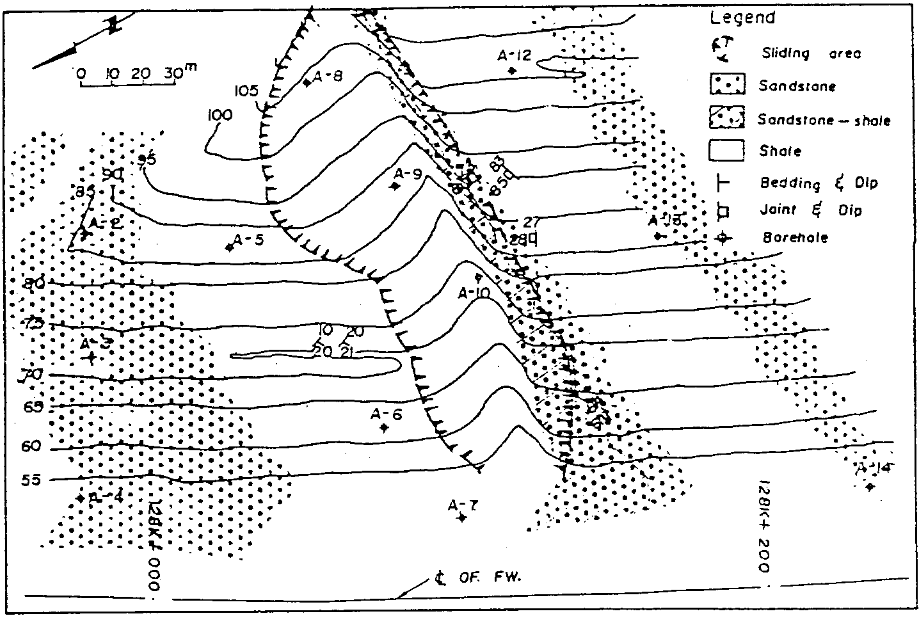

Fig. 9 Geological Map of Station 128 K

From results of geological study and site exploration, cross-sections were prepared. These sections show that the rock formations in this region crisscross with each other in a paralled pattern which is a clear characteristics of monocline structure. Piezometric data indicate that in certain parts of this region, the underground piezometric pressure is very high, particularly during rainy season. For example, the piezometric pressure head of the piezometer installed in the sandy shale formation formation at 7 m below the ground surface in Borehole A-14 was 1.2 m higher than the ground surface. Overall speaking, this phenomenon was confined to localized areas. Since the entire surface of the slope concerned was exposed, the rain water would either flow along the surface to drainage ditches which would cause erosion, or permeate through cracks, joints and bedding planes into the rock formation. These water would stop at the upper boundary of the shale stratum which has rather low permeability and thus form a temporary perched water zone. These perched water zone is a major cause of instability of the slope.

Results of stability analysis according to method proposed by HOEK and BRAY (1977) indicate that the entire slope would have a safety factor about 1.5 under the condition of no pore water pressure. This factor of safety is adequate to maintain stability of the slope. The remedial measure therefore emphasize the installation of adequate underdrainage and surface drainage. furthermore, cement mortar was applied to protect the slope surface from erosion and infiltration.

Case Three

At Station 20k + 700N, there is a 50 m high, 250 m long, slope on the north side of the Freeway. The slope faces the highway in a fan-like shape as shown in Fig. 10. Prior to the construction of the Freeway, the original terrain in this area is a cut slope with a dip angle of 20°. The slope was steepened to 26° with localized areas up to 28° for the Freeway. During the construction of the Freeway, some landslips have occurred along the slope. The slope surface was then covered with concrete-grassgrids. In addition, there were three horizontal surface drains connected to a transversal intercepting drain.

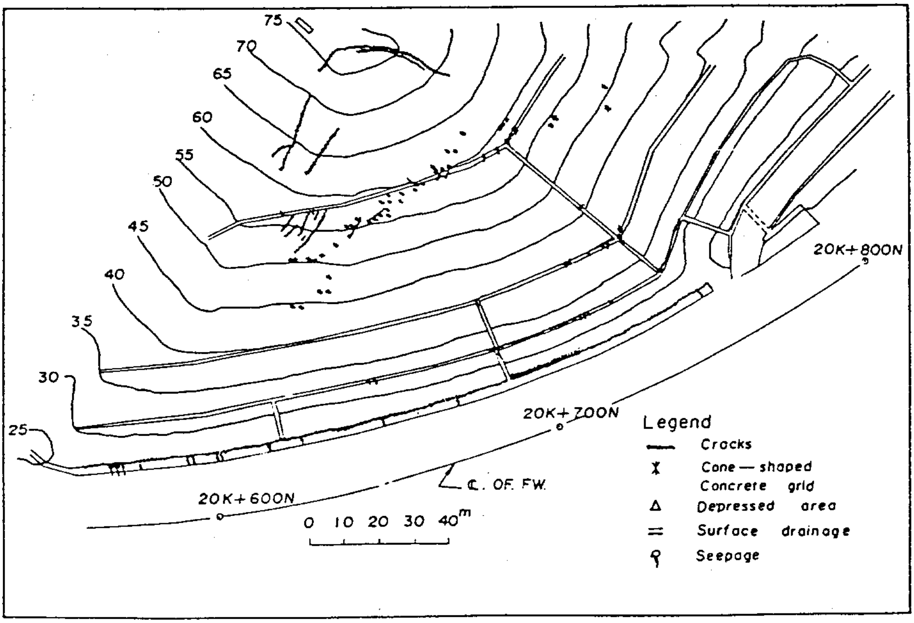

Fig. 10 Geological Map of Station 20 KN

Prior to 1980, some minor movement of the concrete-grass grids near the toe of the slope was noticed. By July-August 1980, the movement became serious and caused a 25 m long, 35 cm wide crack at the top of the slope. At one location, the concrete beams moved about 21 cm and total length of cracks extended to over 400 m. The longitudinal concrete ditches were broken off at many places. Due to uneven movement of the substrata, the concrete grids at over 30 locations were pushed up and form cone shaped configurations as illustrated in Fig. 11. Continuous movement of the slope was observed upto September but the slope survived the Norris Typhoon on 27th August without serious failure, as the cracks were filled and sealed with cement mortar before the typhoon.

Fig. 11 Photo Showing Cone shaped Concrete Grids

A detailed investigation was carreid to delineate the causes of the slope instability. It was found that from the surface to a depth of about 21 m, the slope is composed of interstratified layers of sandstone-shale and silt-stone. There were three strata of fractured materials interbedded in the formation. The bedding planes of the surface layer dip towards the highway. The piezometric pressure of the groundwater in this area does not appear to be exceptionaly high which was even not affected by the Norris Typhoon.

Results of stability analysis indicate that the factors of safety of all potential sliding surfaces of the slope are less than the designed requirement for stability. Based on the investigation results, the causes of this slope can be attributed to the following:

(a) Poor Geological Condition

The dip angle of the bedding planes of the rock formation is towards the highway, and the bedding planes between the sandstone-shale and siltstone form weak planes. The rock formations are sensitive to water and easily weathered. It was also found that the dip angle is about equal to the residual angle of shearing resistance of the rock.

(b) Creep

The sandstone-shale and siltstone formations are composed of fine particles which have high creep potential. Laboratory test results, as illustrated in Fig. 11, indicate that the rock slope in this area would develop significant movement due to creep under constant load. Furthermore, additional creep could develop due to seasonal variations in water content and differential strength characteristics between the various rock strata at the contact surfaces.

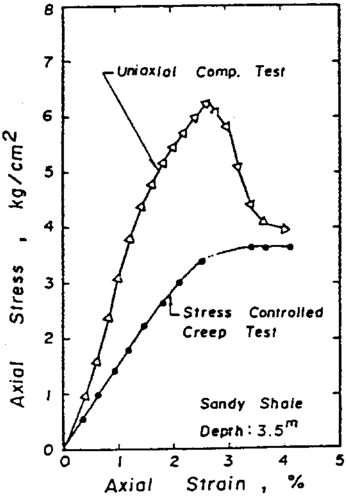

Fig. 12 Creep Test Result of Sandstone-Shale at 20 KN

(c) Swelling of Shale Formation

In the first half of 1980, there was an overall dry weather in Taiwan, the surface layer of the slope concerned became desiccated. In July, part of the grass plant on the slope was destroyed by a wild fire and thus the surface of the slope was partially exposed. Upon rain, the dried rock surface quickly adsorbed water and swelled which increased the rate of movement of the sliding mass. Test results indicated that the shale formation expands up to 8% upon saturation with water.

(d) Inadequate Drainage

Some of the concrete grid beams was not properly buried under the slope surface. In several places, due to erosion of the soil inside the grids, the beams were exposed above the ground surface and became traps for water which greatly increased the amount of water seeping into the underlying rock. Furthermore, the spacing between the longitudinal drains was too large and thus was not efficient enough to intercept heavy rain water.

It was found that it is impractical to stabilize a creeping slope with rock anchors or retaining structure. The most efficient way is to reduce the mass of creeping body and to flatten the slope. The main remedial measures adopted for this slope involve the removal of a part of the rock mass in the upper central portion, cutting the upper slope into two steps with side slope of 20^{o}, installing additional surface drains, and protecting the entire slope with grass plants without concrete grids. At the time of preparation of this paper, the design of the remedial measures was completed but construction has not started. In order to ensure long term safety of the slope, a monitoring system including installation of slope indicators, piezometers, and surface measurements was incorporated in the design.

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