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REPEATED COLLAPSE OF CUT SLOPES DESPITE REMEDIAL WORKS

S. G. Lee
Department of Civil Engineering
The University of Seoul, Seoul, Korea

S. R. Hencher
Halcrow China Ltd.
Department of Earth Sciences, University of Leeds, UK

Abstract: *The collapse of slopes and the subsequent costs of remedial works are often the result of insufficient geological investigation and inadequate interpretation of ground conditions prior to design. This is compounded by inadequate safety inspections and systematic problems associated with poorly defined responsibilities for the stability of cut slopes. This paper reviews such problems in detail with reference to large slope failures in two areas in Korea.*

INTRODUCTION

Expenditure on ground investigation is typically low in Korea, of the order of 0.05-0.2% of the total construction costs; ground investigation costs for cut slope work are generally even lower. Low cost, low quality investigation means that geological structure is rarely recognized as an important factor for the stability of cut slopes at the design stage. Related statistics are that almost 30% of cut-slopes collapse during their formation and, following an in-depth review of ground investigation and design-related items for 102 designs for cut slopes, only 10% were found to have been carried out using appropriate methods (Lee & Hencher 2007; Lee et al. 2007). It is estimated that there are about 1 million cut slopes in Korea which is of the order of 20 times the number in Hong Kong (Lee et al. 2008).

Standards for geological investigation and design are not generally well-specified in Korea and this contributes to the difficulties in identifying the responsible party for any particular collapse despite apparent poor design. Furthermore as investigations following failures are also often inadequate, causes of collapse are not properly identified and failures are therefore attributed incorrectly to some unforeseen natural cause. In addition, where the responsible party remains unidentified, it is common that remedial works have to be paid for by the Client/Owner regardless of the cause of collapse and the cost of the remedial works. Given that failures are generally attributed to natural causes, losses and injuries from cut slope collapse are often not indemnified despite legal action being taken in many cases.

One of the ironies in the attribution of slope failures to natural causes (generally heavy rainfall) is that cut slope design standards of Korea generally require that groundwater is assumed at the ground surface which should account for the most severe condition - it should not be possible to put the blame on excess water pressures as the result of abnormally severe rainfall. Conversely therefore, failures might generally be attributable perhaps to inadequate geological investigation and incorrect ground model, wrong geotechnical parameters or errors in slope stability analysis. These difficulties in identifying contributing factors in cut slope design and subsequent failure has become a matter of public concern in Korea. KBS TV, Korea's public service broadcaster, has presented two documentaries entitled “Money is earned only after collapses” (May 2001) and “Collapses attributable to the sky?” (September 2002). Two examples of large cut-slope collapses, which illustrate the problems concerning cut slopes in Korea, are discussed below (Figure 1).

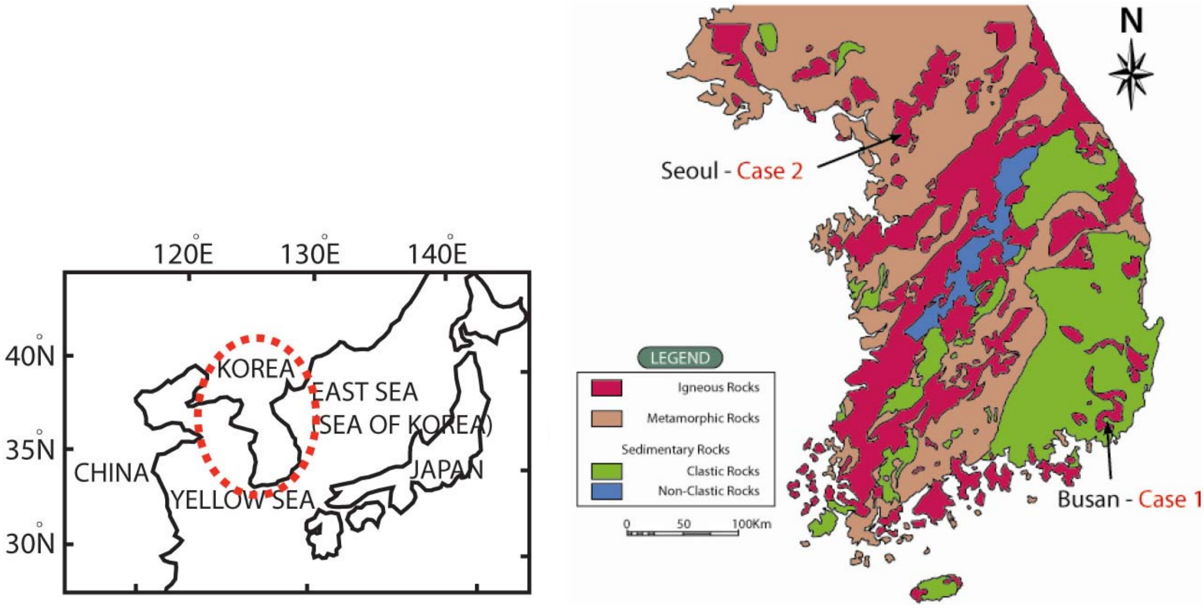


Figure 1: The locations of two examples of large cut slope collapses in Korea

CASE 1: ROAD CUT NEAR TUNNEL PORTAL AREA OF THE WHANGRYEONG MOUNTAIN IN BUSAN

Overview

Busan is the second biggest city in Korea with a population of 4 million in an area of 760km^2. Forty three percent of the area is mountainous and is classified as a “cut-slope disaster risk area” (Lee 1999). In September 1999, tension cracks of 20-30m deep and 15m wide were found 80m behind a large scale cut slope (50m high and 130m wide) located at the entrance of Busan Tunnel where 80,000 vehicles are passing through per day. The related failure of the cut slope involved 140,000m^3 of material that moved 15m; several vehicles were buried and 130m of bridge piers were destroyed. The local geology comprises inter-bedded layers of strong sandstone and weak tuffaceous shale with bedding dipping at 15-20° (Figure 2).

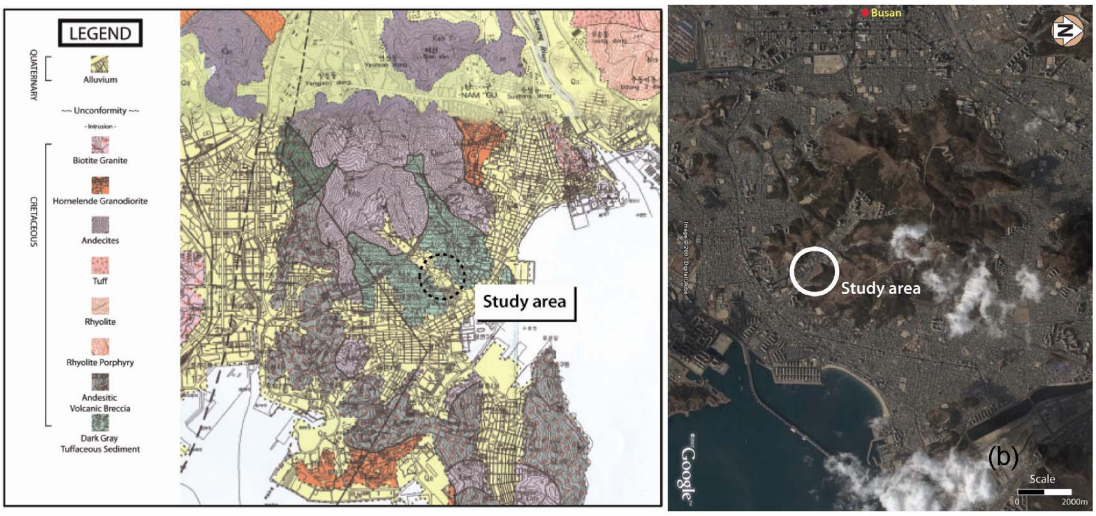


Figure 2: (a) Geological map (KIGAM 1978) and (b) Satellite photograph (Google Earth 2007) of Busan area

Process of Development

Original Design (July 1991)

Boreholes were not put down specifically at the cut slope for the original design (Yoo 1991); a borehole investigation (terminated 1m into moderately weathered (MW) rock) was conducted for bridge foundations 30m away from the cut slope. Upper soils were only 1.2-1.7m thick. Underlying the soil, MW rocks were collected as rock fragments; TCR $\leq$ 10%, RQD = 0 (Figure 3). The 40m high slope was then designed with a 1:0.5 (63°) standard gradient, based on the Korean construction standards (KEC 2001; MOCT 2004) for the design of cut slopes as follows: moderately weathered to fresh rock 1:0.5 (63°), highly weathered rock 1:1 (45°), and soil 1:1.2 (40°) to 1:1.5 (34°) depending on rock material strength.

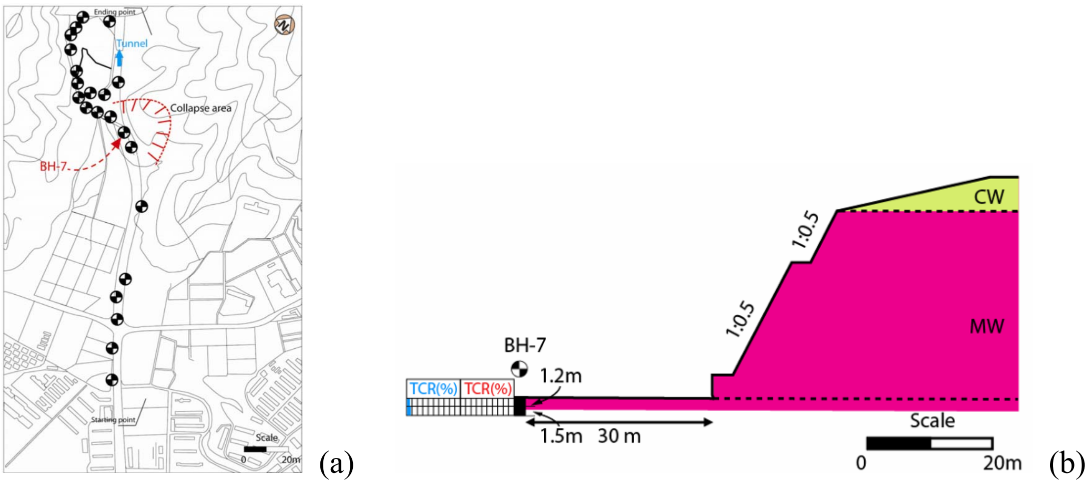


Figure 3: (a) Location of boreholes for design of the structures relative to the failed slope and (b) assumed cross section for the slope

1st Safety Inspection (January 1993)

Field mapping was carried out after a small scale 1st collapse before completion of cutting work and the geological conditions are described in some detail in Kim (1993). Vertical joints were commonly developed within bedding units; thickness of sandstone layers was typically 300-400mm with thin (10-20mm) highly weathered (HW) intercalated shale layers. Stereographic projection analysis indicated possible toppling failure; the upper part of slope was therefore redesigned with a gentler gradient of 1:0.6 (59°) (Figure 4).

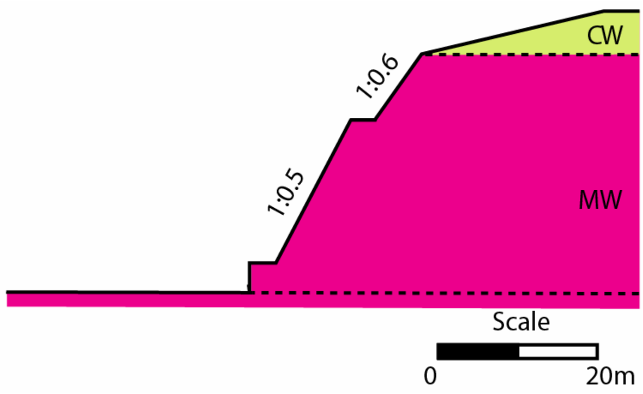


Figure 4: Result of 1st cut slope safety inspection (reduced gradient of the upper part to 1:0.6).

2nd Safety Inspection (December 1995)

Further collapses of the cut-slope occurred infrequently after completion in May 1995 and, according to the field geological survey conducted for the safety inspection in December 1995 (Chae 1995), some small-scale wedge mechanisms were identified but the risk of bedding plane translational failure was not analyzed. The possibility of circular failure was also considered high (Fs = 0.99), so for stabilization, it was suggested to reduce the gradient of the lower part of the cut-slope gentle to 1:0.6 (59°) (Fs = 1.14). In addition, the surface of cut slope was to be covered with wire-mesh and vegetated shotcrete (Figure 5).

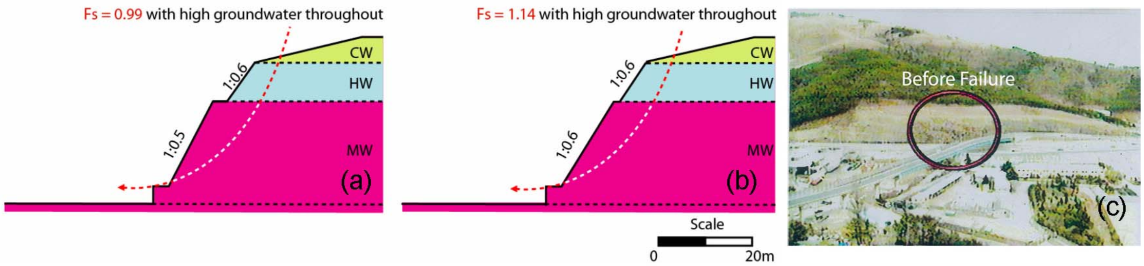


Figure 5: Result of cut slope stability review in 2nd safety inspection. (a) MW slope by 1st safety inspection (1:0.5), (b) MW slope by 2nd safety inspection (1:0.6) and (c) status of reinforced cut-slope after 2nd safety inspection (before 3rd collapse)

3rd Safety Inspection (November 1999)

Heavy rain occurred in September 1999. The accumulated rainfall for 2 days before failure was 100mm; maximum rainfall per hour on the day of the failure was 39mm (KMA 1999). Such rainfall intensity is considered dangerous according to landslide triggering data of Korea (KFS 1993). A 3rd large scale collapse developed along a bedding plane inclined at 15-20° and filled with clay. This failure resulted in the burial of 10 vehicles and destruction of a bridge (Figures 6 & 7). As this important road was to be re-opened urgently, a temporary protective wall was installed using H-piles and landslide debris removed (toe-cutting); collapses occurred continuously in October 2000 and January 2001 during the works.

At the 3rd safety inspection by a professional institute (Han 1999), boreholes were put down at 8 locations to 22-35m depth and direct shear tests carried out on clay samples which gave reported strengths of ϕ = 21.4°, c = 11kPa at natural moisture content and ϕ = 16.8°, c = 11kPa where saturated.

The failure mechanism was identified as bedding plane slip involving clay infill. A more gentle gradient (1:2.0 (27°)) was proposed together with reinforcement H-piles to provide Fs = 1.4 which is higher than Fs = 1.2-1.3 normally adopted because of the importance of the slope (Figures 8 to 10).

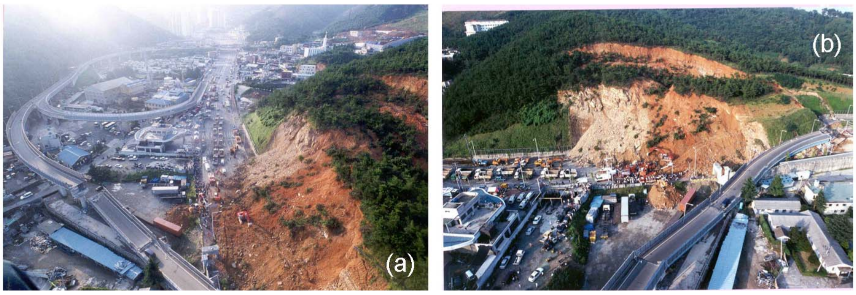


Figure 6: Scene soon after major landslides. (a) Side view and (b) front view

Figure 7: Geological characteristics of failure planes. (a) Failure plane from side, (b) bedding plane from the bottom, (c) tension cracks at the top, (d) bedding planes exposed in tension cracks.

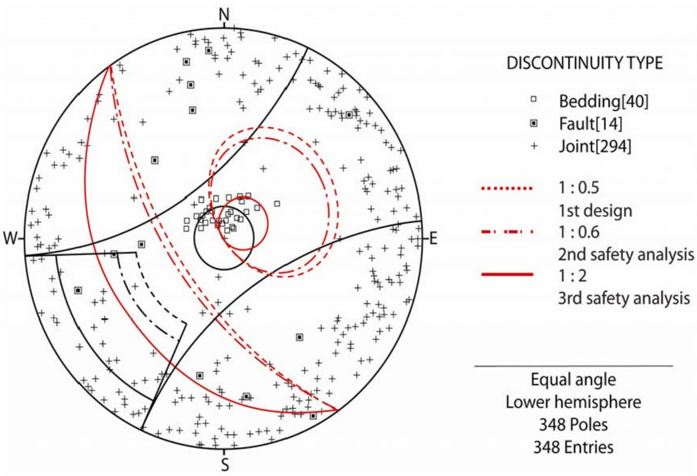


Figure 8: Stereographic projection for fracture data - mostly bedding and orthogonal joints (Choi & Paik 2002).

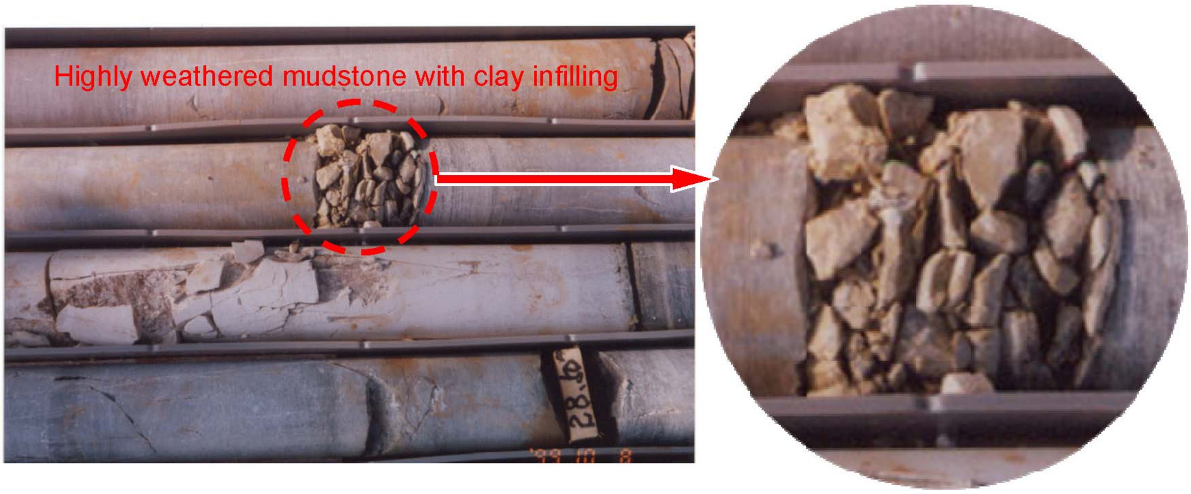


Figure 9: Photograph of core of HW shale with clay filling developed at 26 m deep in MW-SW sandstone (Han 1999)

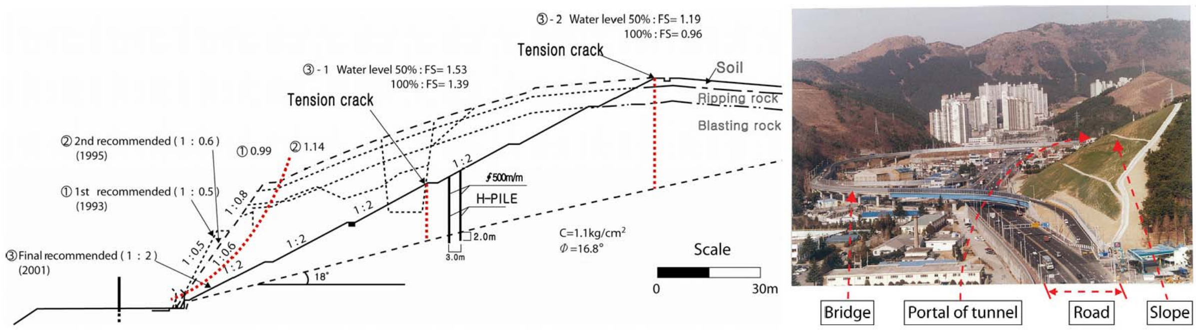


Figure 10: (a) Result of 2nd safety inspection and (b) scene of completed remedial works

Discussion

(1) The fundamental problem was that a proper ground investigation was not performed for this large, important cut slope and that boreholes were taken to an insufficient depth at 30 m. Data from the investigations for nearby structures were totally inadequate for design of this slope.

(2) Once the nature of the geology had been broadly established (it was not difficult), the application of kinematic analysis was poor, totally overlooking the possibility of translational failure on low angle but weak bedding surfaces and poor understanding of the limitations of the stereographic projection method (Hencher 1987; Hencher & Knipe 2007).

(3) Rock cut slope collapses are commonly the result of infiltration of groundwater into tension cracks developed to the rear of cut slopes; analysis of aerial photographs shows that tension cracks were already developed at the back of cut slope by the time of 2nd safety inspection in 1995 (Figures 11 & 12). The failure type of the cut slope was misjudged as a circular failure of soil rather than translational bedding plane slip in the 2nd safety inspection despite the Fs for such a mechanism to be low as 0.84 when fully saturated. A contributing factor to the history of collapse was that remedial works were carried out in a hurry with no identification of the basic cause.

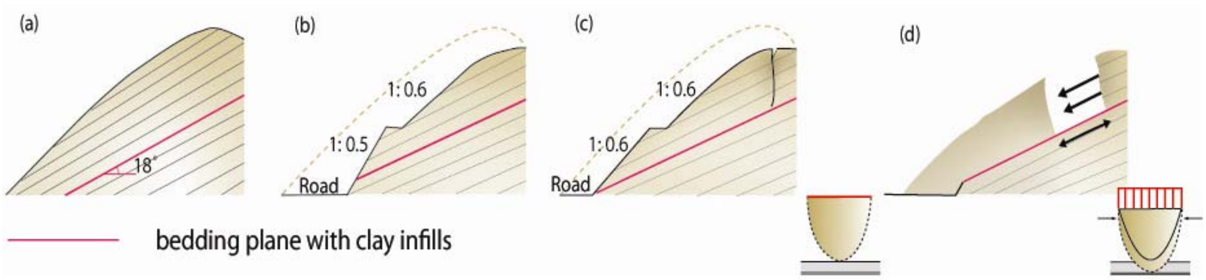


Figure 11: Process of collapse from original design to final catastrophic failure. (a) Original ground before cutting work, (b) after cutting work (1995), (c) initial development of tension cracks at the back of rock masses (1995-1999) and (d) creation of large landslide (1999).

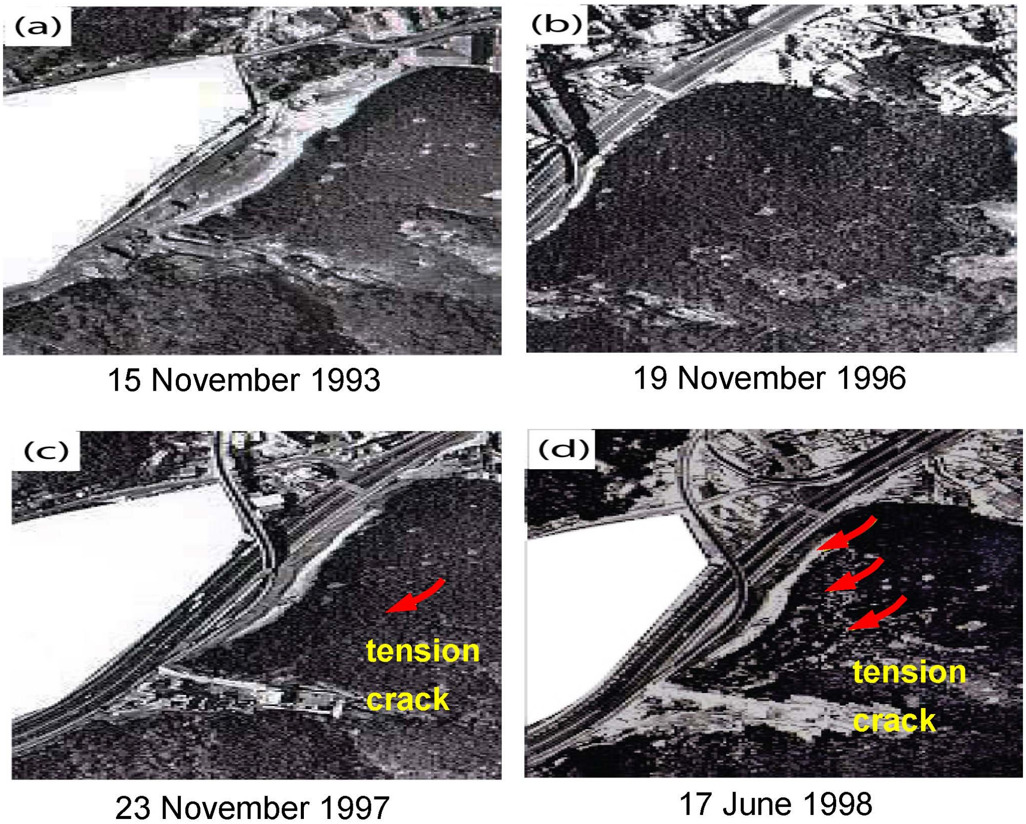


Figure 12: Aerial photographs of the landslide area before the major landslides in September 1999; (a) 15/11/93 (b) 19/11/96 (c) 23/11/97 (d) 17/06/98.

CASE 2: CUT SLOPE AT THE DAEMO PRIMARY SCHOOL IN SEOUL

Overview

Seoul, Korea's capital has a population of 10 million people in an area of 605 km^2, 20% of which is mountainous. A primary school for 1,700 students was constructed on a cut platform in the gneissic area in Seoul. The 1st safety analysis and landslide preventive works were performed following a collapse soon after construction of the slopes; a 2nd safety assessment was carried out because of continuing concerns regarding the stability of these cut-slopes adjacent to a school (Figure 13).

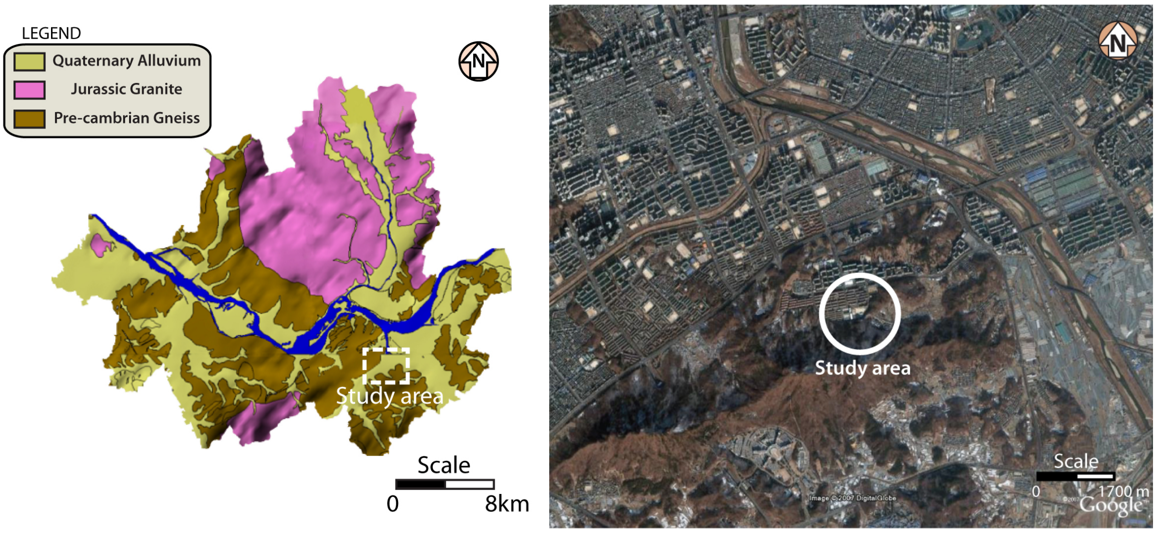


Figure 13: (a) Geological map (KIGAM 1982) and (b) Satellite photograph (Google Earth 2007) of the study area

Process of Development

Original Design (May 1994)

For the original design in 1994, no boreholes were carried out despite the large scale of cutting (20~30m high slopes, 250m wide). Instead, borehole data 300m away from the site was assumed to be relevant; the area was considered to comprise completely weathered (CW), HW, and MW rock. The lower part of the cut-slope was designed to have a 1:0.5 (63°) standard gradient with vegetated surfaces (Cho 1994) (Figure 14).

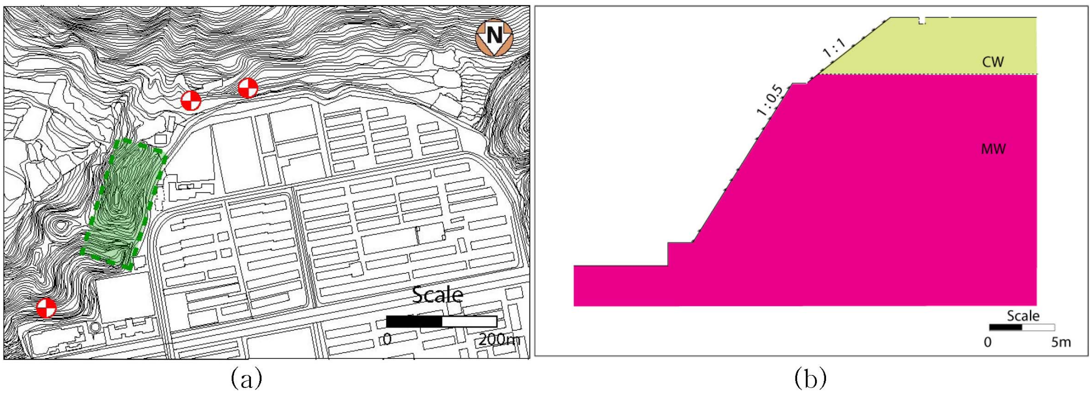
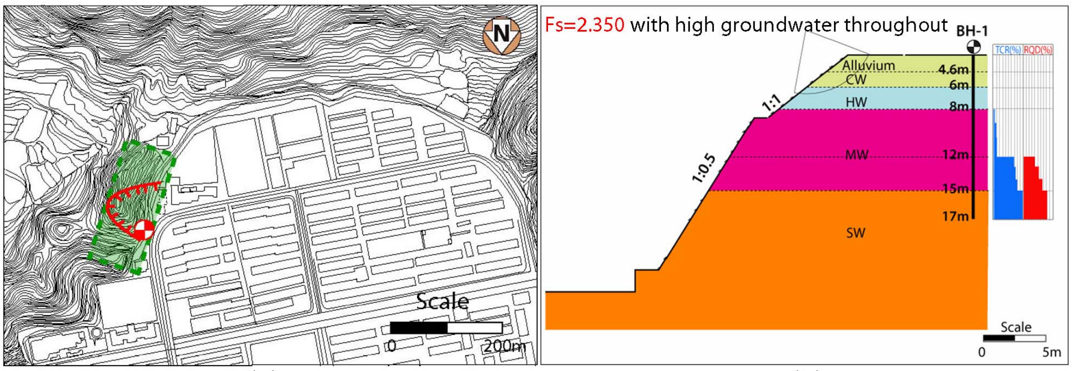


Figure 14: (a) Location of boreholes used for design; (b) cross section (assumed geology)

1st Safety Inspection (November 1998)

Part of the cut slope collapsed during heavy rainfall in July 1998. The failure apparently occurred along the boundary between soil and MW rock (Figure 15). One borehole was proposed at the location of the collapse to terminate up to 2m into slightly weathered rocks. Rock bolts were proposed in some sections considered prone to planar failure. Otherwise, the lower part of the cut slope was considered safe with a 1:0.5 (63°) gradient. The upper part of the cut slope was proposed to be cut at a gradient of 1:1 (45°) (Park 1998) (Figure 16).

Figure 15: Cut slope collapses in August 1998. Front view after landslide (Lee 2001)



(a) (b)

Figure 16: Location of boreholes and cross section following 1st safety inspection. (a) Location of borehole investigation and (b) cross section.

2nd Safety Inspection (November 2000)

Works were completed following the recommendations from the 1st safety review of the collapsed area but residents near to the site made civil appeals to Seoul Metropolitan City Council and a 2nd review was instigated for the entire cut slope (Lee 2000).

Despite much of the cut slope being covered with vegetation, exposed areas of slope were mapped geologically (Figure 17). In addition, 13 boreholes were put down to the height of the cut slopes and logged using TV cameras (Figure 18). In addition, 2 lines of field seismic refraction survey were performed at the ground surface of the upper part of the cut slope. A 3D stratum model (Figure 19) was created through integrated analysis of the surveyed data including the videos from the boreholes. The investigation allowed the joint fracture network to be characterised and several faults were located. It was also found that the boundary between soil and rock in the whole cut-slope area was inclined towards the school. Shear strength tests were performed on the boundary between soil and rock together with a variety of other laboratory tests at natural and saturated moisture conditions (Lee 2001).

Using numerical analysis by DEM (distinct element method), failure mechanisms involving discontinuities were found to be relatively stable. Displacement analysis by FDM (finite difference method) showed that displacement was greatest at the boundary plane between CW and MW rock (Figure 20). Considering both translational and circular failure mechanisms along the boundary of CW and MW rock by limit equilibrium, it was demonstrated that there was a concern for stability under high groundwater conditions (-3m from ground surface which equates to a 100-year frequency 3-day consecutive rainfall analysis). It was therefore proposed to reduce the gradient (1:1) and apply anchoring/nailing to satisfy both circular failure and planar failure conditions (Figures 21 to 23).

Figure 17: Panoramic photograph of Daemo primary school slope (Lee 2000)

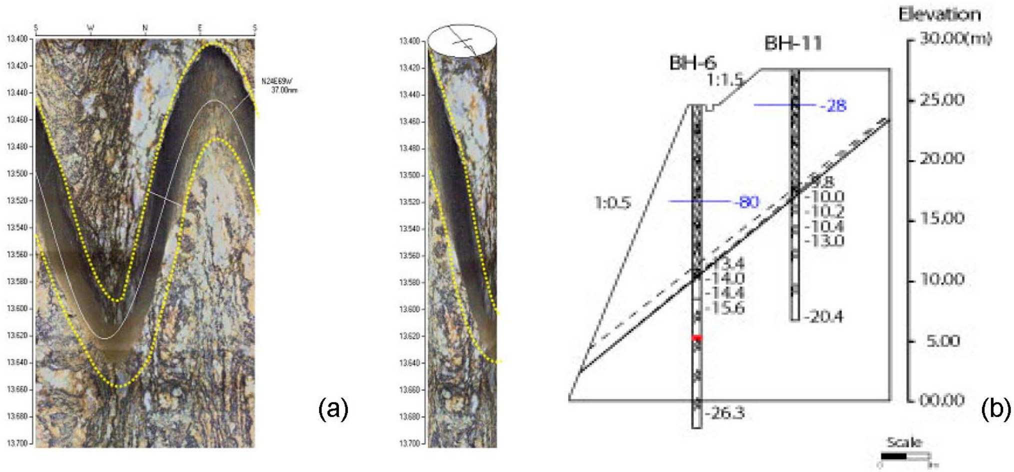


Figure 18: Results of borehole investigation. (a) Fault zone observed by borehole camera (BH-9) and (b) vertical section view interpreted from two boreholes (Lee & Geum 2002).

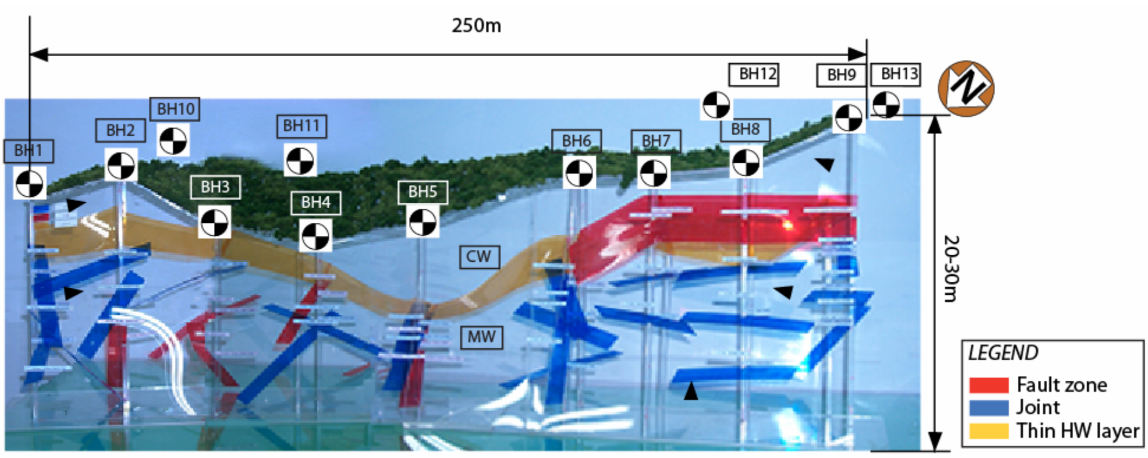


Figure 19: 3D geological models based on borehole camera interpretation (Lee & Geum 2002)

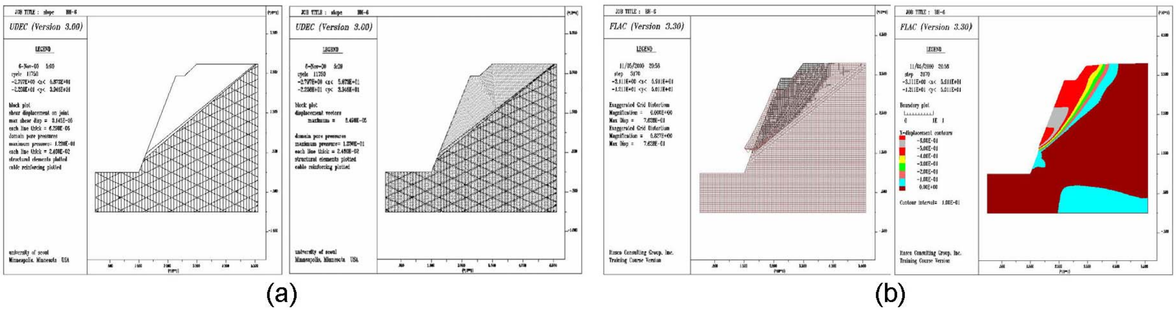


Figure 20: Result of numerical analysis of cut slope stability by (a) DEM and (b) FDM (Lee 2000).

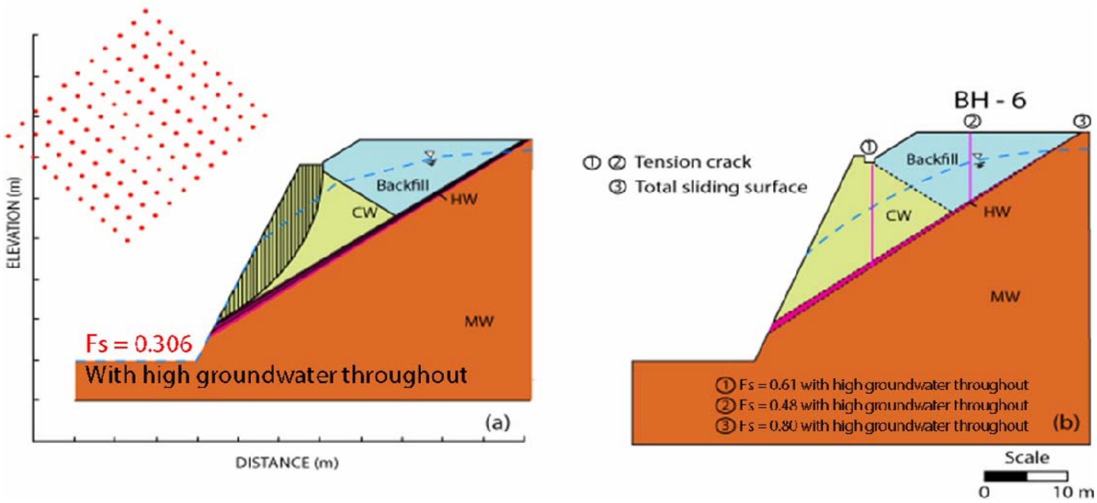


Figure 21: Original slope (at the highest water level). (a) Circular failure in soil and (b) plane failure along the boundary of soil and rock (Lee 2000)

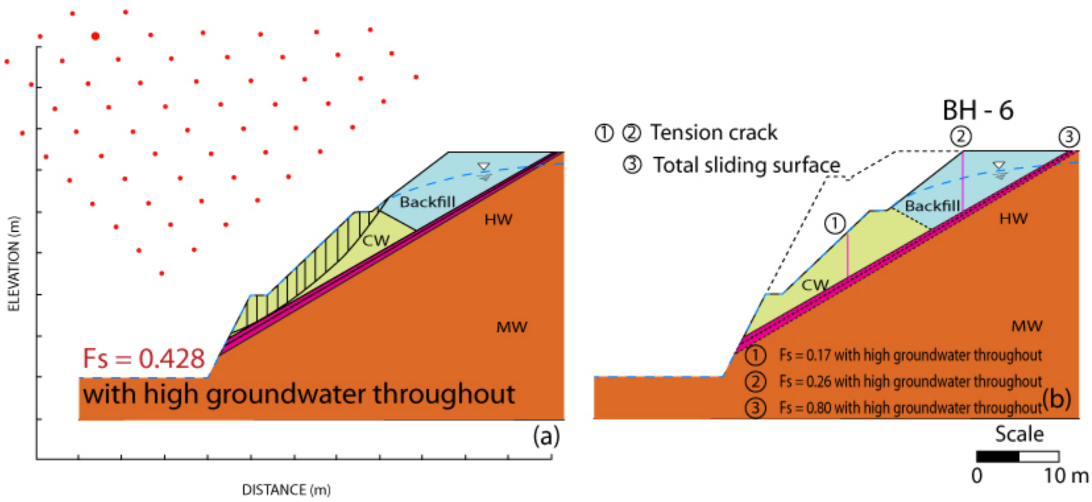


Figure 22: Cut slope (1:1, at the highest water level). (a) Circular failure in soil and (b) plane failure along the boundary of soil and rock (Lee 2000)

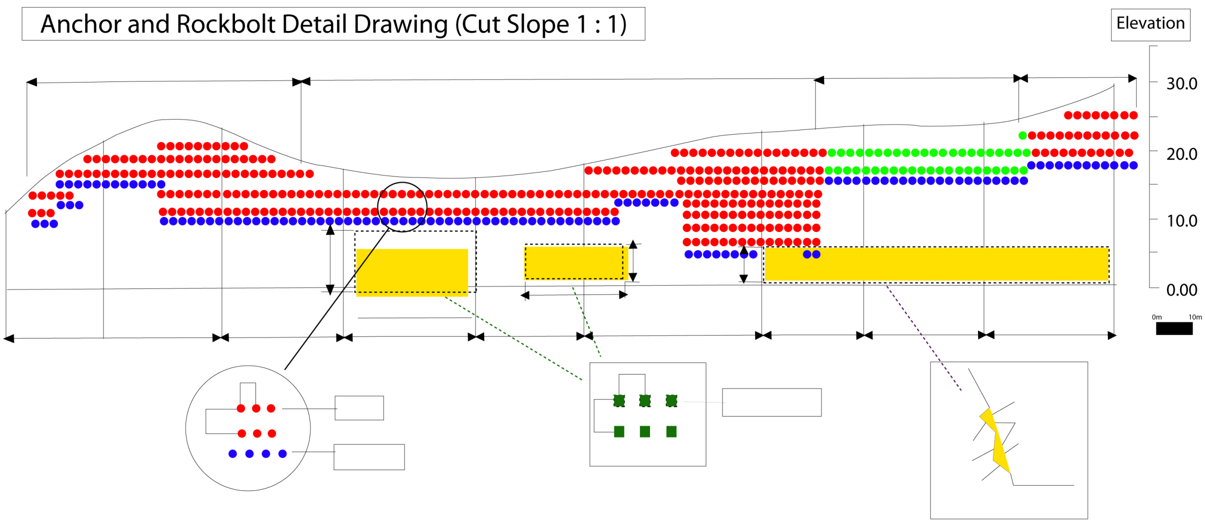


Figure 23: Anchor and rock bolt detail drawing with reduced gradient (1:1) (Lee 2000)

Lessons

It is inappropriate to carry out design solely on the basis of borehole data from a distance of 300 m from site as was done for the original design. It was inappropriate, as was originally done to only consider circular failure within the upper soil as a mechanism for failure. The potential for more extensive circular failure should have been considered continuing into the lower stratum of MW rocks with RQD = 0%. Other potential modes of failure should also have been considered; in this case the original failure occurred on the essentially planar interface between soil and rock and indeed additional preventive works needed to be designed for that failure mode as demonstrated by the additional investigations and analysis (Lee 2002; Lee & Hencher 2007).

CONCLUSIONS

In general, the mountainous areas in Korea have relatively favourable stability conditions because soils are thin and the rocks are not deeply weathered. However, inattention to geology during investigation and the lack of proper design standards means that, with the cutting of more and more large scale cut slopes as mountain areas become industrialized, the failure of large cut slopes is an increasing trend. As the standards for geological survey and design are not properly specified and inspections performed after failures are often insufficient, the causes of collapses are not properly identified. Most are attributed to "natural disaster". As a consequence, the costs of remedial works are paid for by the Korean government and those affected by failures are not indemnified in the majority of cases. It is of prime importance that the system is improved with clear identification of responsibilities for design, construction, and guidance for inspections. Guidance is also required for appropriate geological survey with technical training and the development of consistent technological systems for geological survey, design, construction work, and maintenance. Attention is also required for management with interaction between those responsible for maintaining the stability of cut-slopes. There is a need for establishing a database of cut slope information and an integrated management system.

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