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Drainage Used to Control Movements of a Large Rock Slide in Canada

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1. INTRODUCTION

The Jeffrey Mine, owned by Johns Manville Canada Inc. is situated at Asbestos, Quebec. It is currently one of the largest open pit mines in Canada and the biggest producer of asbestos fibre in the free world.

The open pit, approximately 1,000 ft. deep, is immediately west of the town of Asbestos. On the south side of the pit are located the major crushing and milling facilities (see general location plan; Figure 1).

During 1970 and 1971, a major rock and overburden slide developed in the south east corner of the open pit. The eventual slide limits are shown on Figure 2. At that time ore was being hoisted via a skipway to the primary crusher located at the pit crest. The lower section of the skipway and the associated ore pass bridge structure were located within the slide area.

In order to maintain an efficient supply of ore as well as to control the movements of a significant volume of material, stabilisation measures consisting primarily of drainage, were adopted.

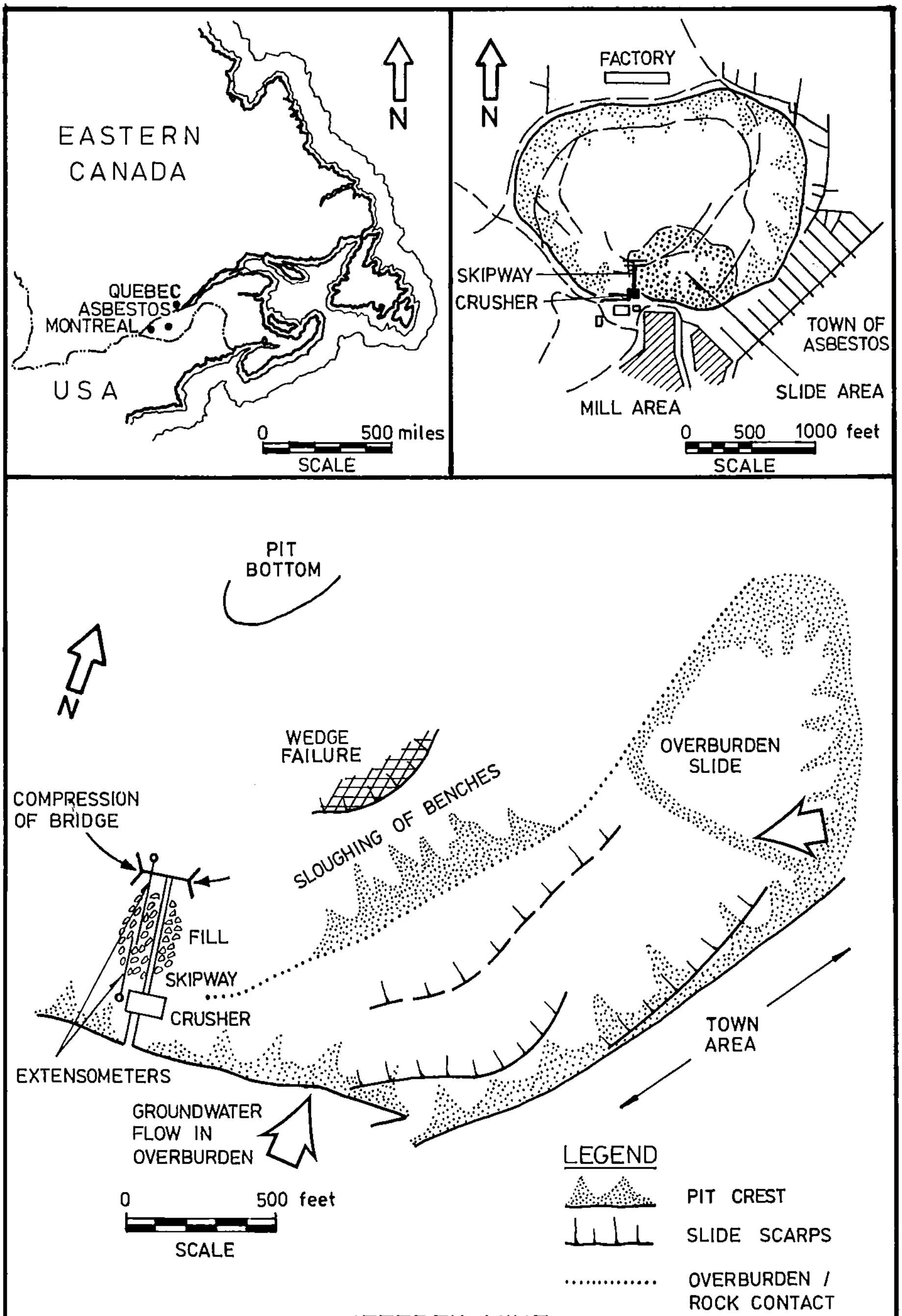

FIGURE 1: LOCATION PLAN AND PLAN OF SLIDE AREA JAN. 1971

JEFFREY MINE
FIGURE 2: AERIAL VIEW OF SOUTH EAST CORNER

2. GEOLOGY OF THE SLIDE AREA

The geology of the south east corner of the pit comprises an ultrabasic rock mass, (serpentinised, peridotite and dunite) which is overlain by some 200 ft. of overburden consisting of silts, sands with gravel layers and glacial tills.

This geology is shown generally in Figure 3. A major shear zone dips into the slope and outcrops on the upper part of the slope. The shear zone consists of highly fractured serpentinised material up to 500 ft. in width. The underlying peridotite is also traversed by other, lesser, generally sub-parallel, shear zones. Fracturing is somewhat irregular and typical of the host rock of an asbestos ore body. Fractures are often infilled with asbestos or brucite fibres and characteristically have a low shear strength.

3. PERMEABILITY CHARACTERISTICS OF THE ROCK AND OVERBURDEN STRATA

The rock types comprising the slope have a low mass

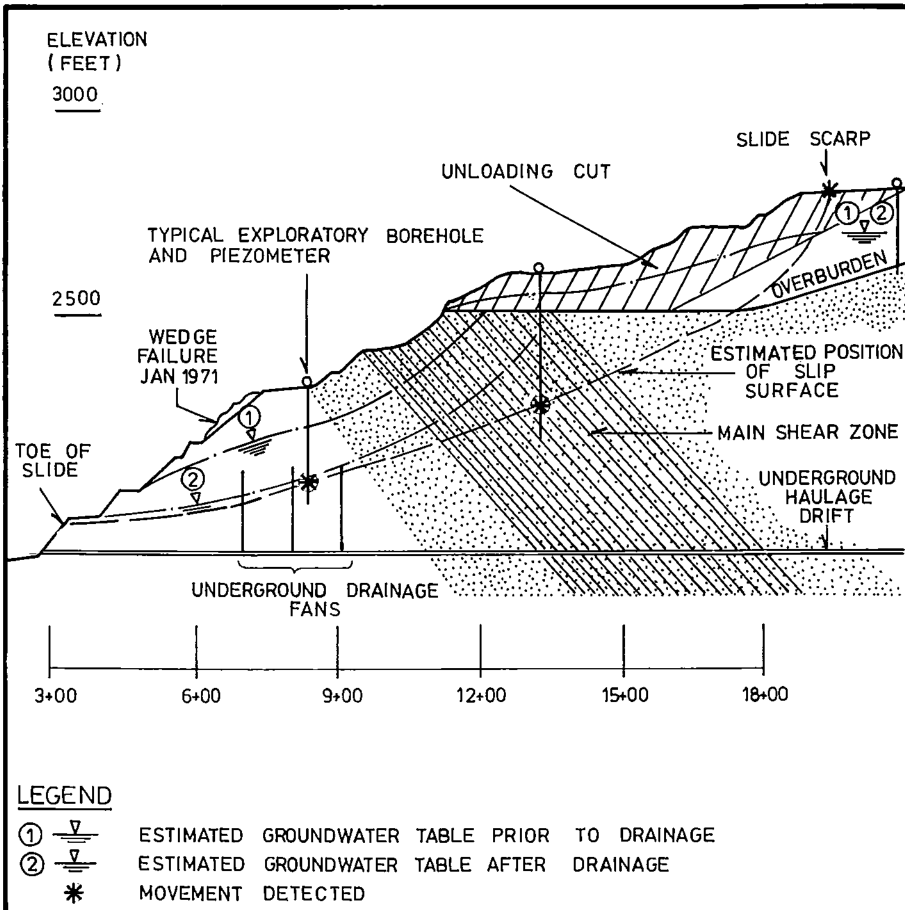

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FIGURE 3: CROSS SECTION THROUGH AXIS OF SLIDE

permeability, typically in the range 10^{-6} - 10^{-9} cm/sec. The shear zones contain a significant proportion of rock flour and are probably less permeable than the more competent rocks.

The overburden is characterised by relatively permeable gravel and sandy gravel strata within lower permeability sands, silts and tills. The gravels often form an infilling to pre-glacial channels in the bedrock and carry significant quantities of water along such alignments.

4. GROUNDWATER CONDITIONS

A significant gravel layer carries water from the south and discharges from the lower overburden slopes to the east of the skipway (see Figure 1). This water source, along with surface precipitation, appears to form the main source of groundwater for the south east corner rock slopes.

Piezometers installed in the overburden and rock slopes have indicated a general groundwater table some 50 ft. below the slope surface.

The potential benefits of slope drainage (reduction in groundwater pressures) in the rock were evaluated in detail prior to the slide occurrence, to determine whether or not steeper slopes could be developed. Such studies concluded that overall drainage of the rock slopes was not economically feasible owing to the following factors:

(1) high recharge from overburden strata that could be only partly controlled by surface drainage/diversion measures.

(2) extremely low permeability of the rock types requiring short drainage paths.

(3) scale of drainage measures required for a potentially 1,000 ft. high slope section.

The conclusions are borne out by the extremely limited and local influence of the extensive underground workings at the 2,000 ft. level on the south side of the open pit.

5. SUMMARY OF THE SLIDE DEVELOPMENT

Between 1965 and 1970 the benches forming the upper rock slopes had failed due to rock deterioration within the relatively weak shear zone material (see Figures 1, 3).

Prior to 1970 several relatively shallow slides had occurred within the overburden. These were probably caused by adverse groundwater conditions and the undercutting of the slope toe due to ravelling of the upper benches in rock.

During 1970, a gradual extension of the skipway rails was observed (see Figure 1). The skipway rails had been laid on a 45 degree slope partly composed of fill. (The movements were originally attributed to localised creeping of the fill material.)

Because of the critical role of the skipway installation in the mining operation, extensometers were installed from the crest of the slope and from the ore pass bridge level to check the depth of movement (see Figure 1). The instruments became operational at the end of 1970.

During December 1970 and January 1971, a major overburden slide developed in an adjoining area to the north. The slide involved an area behind the pit crest forming part of the town of Asbestos. Groundwater pressure build-up resulting from the winter freeze-up was shown to be a contributing factor to the initial slope movements. Further movements were influenced by the leakage of service water from the town into existing cracks forming the rear scarp to the slide.

In January 1971, a localised wedge failure between the 2,220 ft and 2,320 ft elevations occurred below the main haul road in relatively competent rock (see Figures 1, 3). The failure was controlled by localised jointing although some extension of fractures may have occurred due to small, overall movements of the slope.

Mining was being carried out near the toe of the slope (approximate elevation 2,100 ft.) at the beginning of 1971. Relatively rapid progress had been made during the latter part of 1970 with mining below the 2,300 ft level.

In February 1971, a massive, overall movement of the south east corner was suspected. A movement monitoring programme was initiated to determine both the overall extent of surface movement and the depth to which movements were occurring. Approximate piezometric conditions within the slope were also determined from the same boreholes. Both monitoring and investigation were difficult due to the significant snow cover that existed.

Measurements on the surface and in boreholes (inclinometer measurements or casing deformation observations using sondes) indicated movement of the entire slope some 700 ft. in height, 2,000 ft. in width and at depths up to 250 ft. (see Figures 3, 4). Approximately 20 m tons of material were contained within the slide zone. The observations

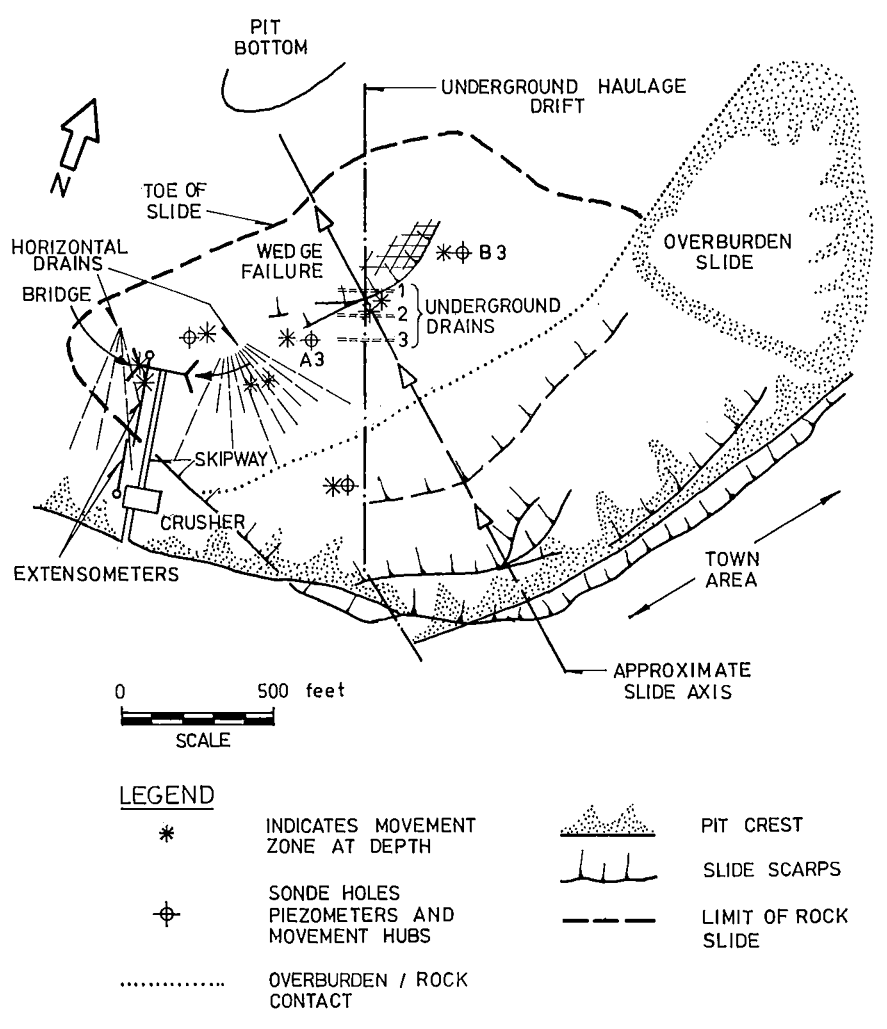

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FIGURE 4: PLAN OF SLIDE AREA MARCH 1971

correlated with extensometer data obtained in the vicinity of the skipway.

Maximum movements were evident at the centre of the slide area with a gradual decrease in movement rates towards the lateral slide boundaries. As movements continued, scarps at the slide crest became evident. Regular maintenance of the skipway track was required to counter the overall extension of the rails. The skipway bridge was subjected to an increasing degree of compression with time, owing to its orientation with respect to the movement axis. Shear displacement of the support columns and jamming of the horizontal ore pass doors were evident.

During April and May 1971, a period during which the main spring run-off occurs, major movements of the slope occurred causing ravelling on the lower rock benches, disruption of the haul road across the upper slope area and extensive damage to the skipway bridge structure.

The primary causes of instability were attributed to the following:

(1) Mining operations at the toe of the slope. The excavation of material in this area reduced the volume of the competent lower rock buttress (See Figure 3) to a critical state where it was no longer able to support the overlying shear material and overburden. The build-up of shear stress through the buttress probably led to the extension of unfavourable oriented shallow dipping joints to form a continuous shear surface at the base of the buttress. This was accompanied by significant internal shearing along fractures oriented roughly parallel to the main shear zone as well as a general break-up and dilation of the more competent rock.

(2) Adverse groundwater conditions as detailed below.

In order to control further movements and to protect the crusher installation at the pit crest, stabilisation measures were required.

6. GROUNDWATER AND ITS INFLUENCE ON THE STABILITY OF THE SLIDE AREA

The investigation of the slide area confirmed the existence of adverse groundwater conditions within the slopes. In addition the break-up of the slope surface and the creation of tension cracks and scarps in the upper slope area led to an increased infiltration of surface water particularly during thaw periods. The overburden area to the north had by early 1971 broken up considerably and was found to contain a large quantity of water within open cracks and fissures much of which was derived from the town area. This water was free to flow southwards into the rock slide.

Later investigations have also shown that a considerable quantity of water probably flowed into the rear scarp of the slide from the buried, gravel-filled channel to the south.

Groundwater sources are illustrated generally in Figure 1. The phreatic surface within the slope profile is shown in Figure 3. The shape of the surface illustrates the high degree of recharge from the overburden slopes as well as the relatively impermeable nature of the major shear zone.

After the geometry of the slide had been defined, preliminary stability analyses using a technique of back analysis were carried out. These analyses demonstrated that the stability of the area could be improved significantly by the adoption of drainage measures that would lead to a reduction in groundwater pressures within the slope at depth.

In addition, unloading of the upper slope area by stripping of the overburden, would further improve stability following drainage.

It had been previously concluded that drainage of the intact rock would be of little benefit in reducing groundwater pressures on a significant scale. However, with the generation of a fairly well defined shear zone at the base of the slide that was likely to be quite permeable (due to dilation of the rock on shearing) it was decided to attempt drainage into the slide base.

7. STABILISATION MEASURES

Drainage measures to reduce groundwater pressures at the base of the slide consisted of:

(1) drainage of the overburden materials to minimise recharge into the underlying rock.

(2) horizontal drains from the skipway bridge area into rock (see Figure 4).

(3) drain holes drilled upwards from the underground haulage drift into the base of the slide (see Figures 3, 4).

The drainage measures were commenced in June 1971.

Drainage of the overburden was carried out using an 'Aardvark' drilling rig used to install perforated PVC casings into the slope. Drilling difficulties occurred when the more permeable gravels were encountered and the programme was only partially successful. Surface water diversion using drainage ditches and sumps was also carried out within the overburden area.

Initial drainage measures within the rock consisted of two fans of horizontal holes drilled from either side of the skipway bridge structure. A heavy duty diamond drilling rig was used to overcome potential difficulties with the highly variable and often sheared rock. Drillholes that did not penetrate the basal zone of the slide yielded little or no water and had only a localised effect on the groundwater pressures.

Most of the drillholes to the east of the bridge penetrated the base of the slide and encountered considerable quantities of water under pressure. The water was contained within a broken, sheared zone at the base of the slide. Dilation resulting from shear movements of several feet had produced an extensive, permeable zone that appeared to be well connected across the base of the slide.

One of the main holes in the east bridge series was temporarily closed off at the outlet after completion. A rapid increase in water pressure to approximately 75 psi was observed indicating high water pressures at a relatively shallow depth. The initial flow magnitudes (of the order of 50 gpm) and the rapid increase in pressure with time indicated the high permeability of the rock at the slide base. After approximately 3 months, the installation was again closed off and pressures were found to have decreased to about 5 psi.

Based on the success of the drillholes from surface, it was decided to rehabilitate the underground haulage drift and drill into the lower central slide area. Twelve drainage holes were drilled from a rail mounted rig so as to intersect the slide zone as illustrated in Figure 5. As expected a highly permeable zone, several feet thick was intersected and water under pressure flowed initially at some 200 gpm. This flow decreased after several months to about 20 gpm after pressures had dissipated across the base of the slide. The relative permeability of the intact and failed material was demonstrated by the lack of influence

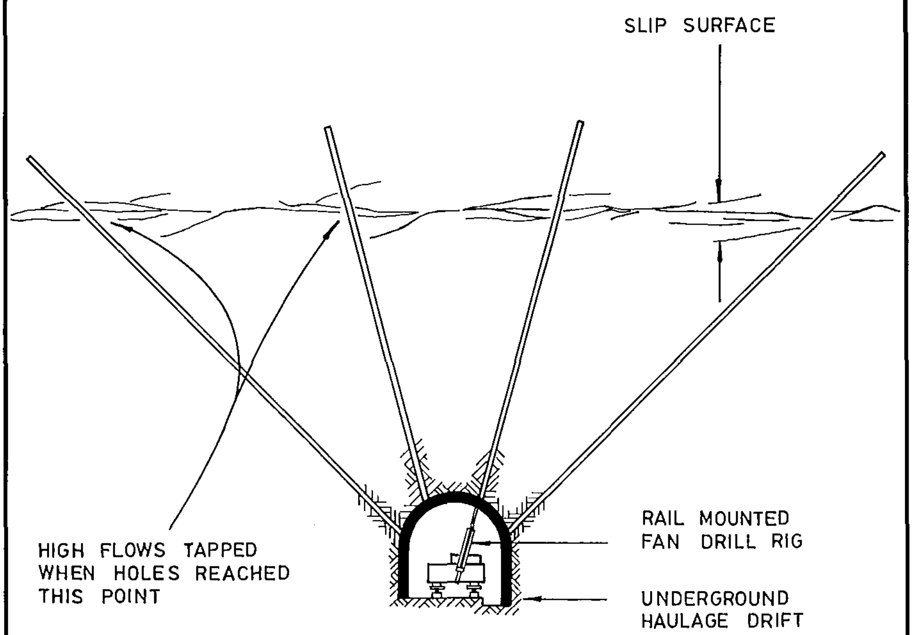

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FIGURE 5: CROSS SECTION THROUGH UNDERGROUND DRIFT SHOWING FANNED DRAINAGE HOLES

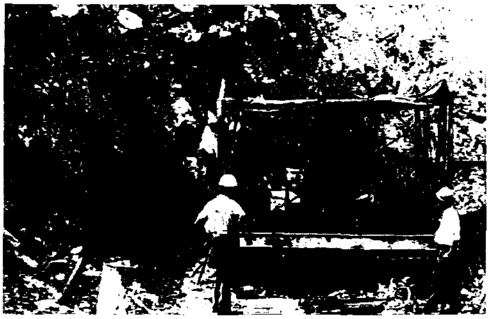
that the adit alone had had on reducing pressures in the overlying slide.

Details of the drainage arrangements are shown in Figure 6.

Concurrently with the skipway bridge drainage programme, some 1.5 m tons of overburden was mined from the upper slide area.

8. RESULTS OF THE STABILISATION MEASURES

Surface monitoring of the slide area showed a significant reduction in movement rates following completion of the initial drainage measures and the overburden stripping (see Figure 7).



SKIPWAY DRAINAGE RIG

DRAIN OUTLET AND PRESSURE GAUGE

FIGURE 6: DRAINAGE ARRANGEMENTS

Complete stabilisation was achieved after completion of the drainage from underground in August 1971.

A similar movement trend was observed from the lower skipway extensometer as illustrated in Figure 7.

The sensitivity of stability to minor changes in groundwater conditions is indicated by a significant increase in movement rates in early August 1971 following approximately 5 in. of rainfall within 24 hours (see Figure 7).

Since drainage was installed during the summer period when groundwater conditions were more favourable, the success of the measures could not be immediately judged. No re-occurrence of movement was however observed during the winter and spring periods of 1972 and 1973 and groundwater pressures within the slide remained minimal. After 1973 significant changes in the slope geometry occurred and further conclusions on continuing stability trends could not be drawn.

9. CONCLUSIONS

The adoption of drainage measures in an extremely impermeable rock mass to control the movements of a large rock slide proved to be successful owing to the existence of a permeable basal zone that resulted from the overall slope movements.

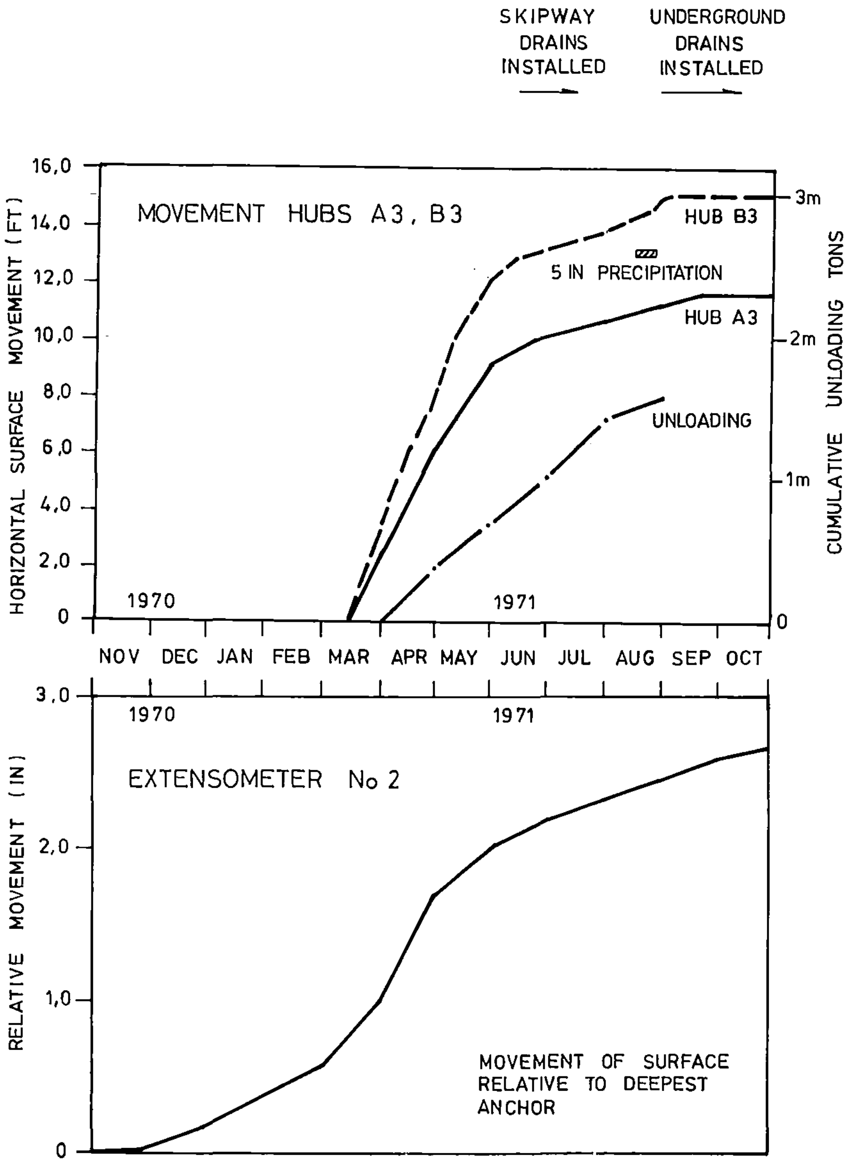


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FIGURE 7: SURFACE MOVEMENT AND EXTENSOMETER DATA

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FIGURE 8: SLIDE AREA FROM WEST

The stabilisation measures allowed the operation of critical installations to be maintained. Major break-up of the slope was also prevented thus ensuring access across the slope and safe conditions in the pit bottom below.

Although in low permeability rock masses, drainage may be impracticable (uneconomic) as a means of improving the stability of intact slopes, it can be successfully used as a remedial stabilisation measure.

ACKNOWLEDGEMENTS

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