Stabilizing a Landslide Above Fisher Penstock, Tasmania

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SUMMARY. A landslide occurred in talus above the penstock during commissioning trials for the Fisher Hydro-Electric Scheme. Investigations showed the slide to be initiated by pore pressures generated by a substantial rise in the water table resulting from leakage of tunnel pressure water. Control was effected by drainage by means of an adit system in the underlying bedrock and by establishing a curtain of drainage holes drilled through the talus from the surface and into the adit.

1 INTRODUCTION

Commissioning trials for the Fisher Hydro-Electric Scheme (Ref. 1 & 2) began in December, 1972, and a landslide in talus was observed in progress above the tunnel outlet portal on 4th April, 1973. The slide (Fig. 1) affected an area mostly south of the tunnel 150 to 210 m wide and extending up slope for 360 m. The upper penstock and tunnel were immediately drained, but gradual minor movement continued for about two months. Movement did not seriously affect the penstock though it is sited on talus for a distance of 180 m near the outlet portal.

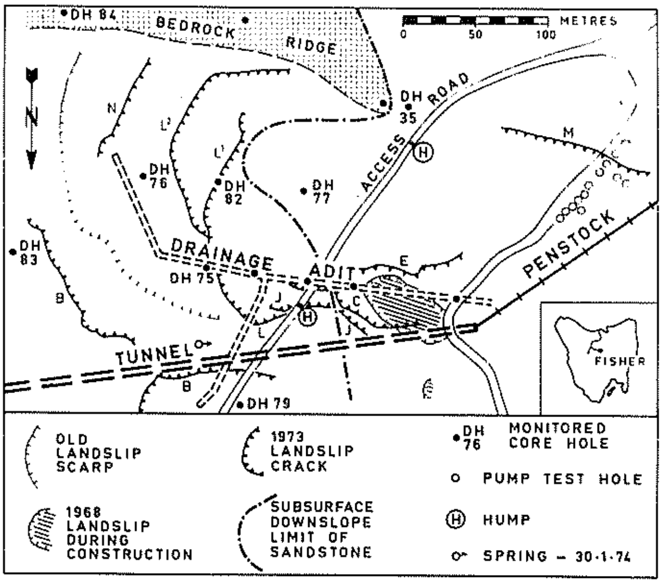

Fig. 1 Locality plan

Water losses from the tunnel were initially at the rate of 0.6 m^3/s but had reduced to 0.1 m^3/s by March, 1973. Because of the possibility that this leakage might initiate a landslide in the talus deposit, an inspection of the area was made on 20th February, 1973, but no leakage was observed although seepage then attributed to rainfall was seen in the road cutting above the access road.

2 SLIDE MATERIAL

The slip was a slow debris slide with the potential of becoming a debris flow. It took place in Pleistocene periglacial solifluction talus that had accumulated as a result of debris slides, flows and avalanches from the plateau slopes in a near glacial climate.

The bulk of the deposit is composed of up to 60% of sound to slightly weathered dolerite boulders and gravel to 1 m diameter in a matrix of sandy clay of medium plasticity, but some of the deposit is also derived from Permian sediments. The permeability is mostly low. The talus water table remains high, and when the slip took place the talus was saturated. The deposit is not forming under the present climate and tends to be stable except where disturbed by development or changes in the groundwater state.

In the affected area the deposit lies on Permian sediments, the surface of which has considerable local relief. This surface is variably weathered and exhibits shearing in drill core, and core loss was high at the interface. The underlying Permian sediments are flatly-bedded and extensively jointed on a rectilinear system. Primary permeability is low, but secondary permeability along joints, faults and bedding planes varies greatly.

3 FISHER SLIP

During construction some 60 000 m^3 of unconsolidated material was removed from the vicinity of the outlet portal and minor slips occurred above the cut. A small slip also occurred along the access road above this cut. In addition, the whole of the area above the access road was badly disturbed by logging activities and left as a maze of tracks, dozer scars and timber debris.

When inspected on 5th April the main cracks (Fig. 1) were B, N, L, L_2, C and D. The humps in the road were present and en echelon cracks were visible between B and L and between N and the lower hump on the road. A mudflow had formed where crack D crossed the construction slip area and water flowed freely from it. This area was oversaturated and completely cracked. Another spring flowed from a crack south of the lower hump on the road and smaller seeps were present especially along the Permian outcrops. The whole area was interlaced with surface cracks from a few metres to 30 m apart, with crack widths up to 200 mm and vertical movement of up to 1 m. The general heading of the slip was towards the penstock. Further movement extending crack J and causing a drop of 600 mm along crack L occurred on 10th

April. No further major movement has occurred.

4 TUNNEL LINING AND LEAKAGE

The 'Mole' bored section of the tunnel (Chainage 254 to 3 000 m, Fig. 2) was generally lined with unreinforced concrete of 230 mm nominal for structural reasons. A major fault at ch. 709 m received a steel liner and the thickness of the adjacent concrete lining was increased to 550 mm. An open jointed and seamed area at ch. 1 905 to 1 926 m also received the thicker lining. A steel liner was installed for a distance of 820 m from the outlet portal, and at the upstream end of the steel liner the first 73 m was grouted to form a 'cut-off'. The rock-concrete interface of the cut-off was grouted through holes drilled 1.5 m into rock at up to 2 800 kPa, and a curtain was established at each end of the grouted section by grouting at 700 kPa via holes drilled 6 m into rock. About 17 t of cement was used.

diameter holes drilled 1.5 m into rock in patterns of three at 3 m intervals over a distance of 247 m upstream of the end of the steel liner. During the 1st stage 374 drainage holes were grouted using 50 t of cement, and during the 2nd stage 222 holes were grouted using 1.5 t of cement. In view of the considerable reduction in grout take no further pressure test was made.

Tunnel water can reach the slip area via the interlocking joint and fault systems. The path in the dolerite is long permitting dissipation over a wide area, thus the most probable path is via joints and faults in the sandstone beds situated from ch. 2 174 to 1 714 m. The tunnel grout cut-off was established in the main sandstone bed and grouting had been carried out for a distance of 247 m upstream. This left some 212 m of tunnel within the sandstone bed with free connection with joints and seams by means of the pressure relief holes. The major fault at ch. 709

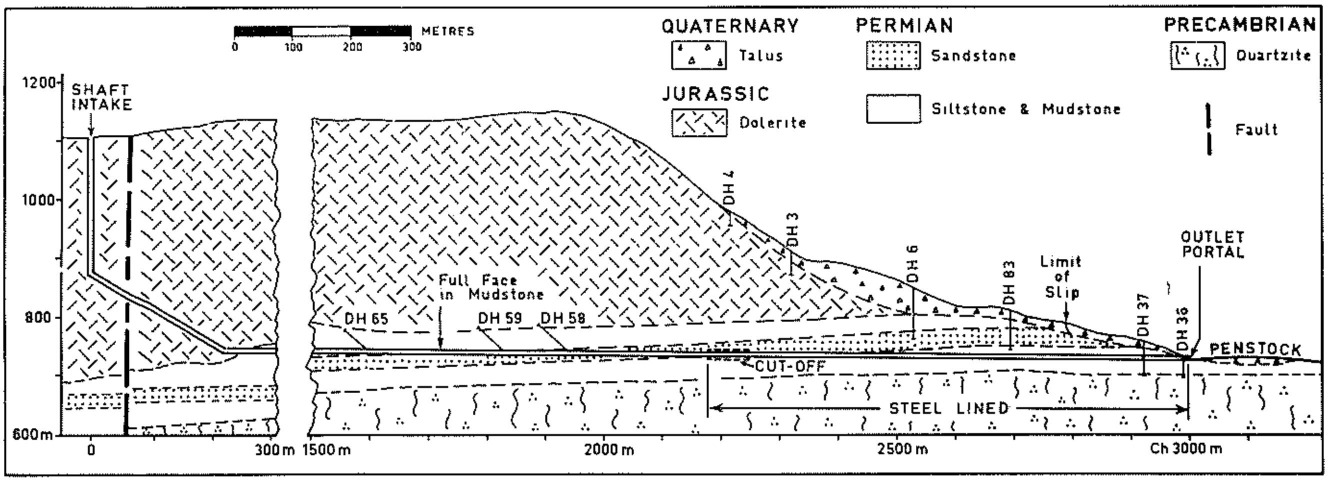

Fig. 2 Geological section of Fisher Tunnel

Throughout the 'Mole' bored section of the tunnel weep holes were provided in the concrete lining to prevent unduly high stresses developing in the lining following upon sudden dewatering. Without the weep holes the lining would extensively crack when subjected to internal pressure, so that leakage from the tunnel would be similar with or without weep holes. In all, 4 340 weep holes of 30 mm diameter were drilled 150 mm into rock either on a regular pattern or to intersect some 230 seams or open joints. Open jointed sections of the inclined tunnel received a nominal 230 mm concrete lining with weep holes, but the major fault at ch. 85 m was lined with 450 mm thick concrete without weep holes. The sound tight dolerite was left unlined, but the vertical shaft was lined throughout.

After completion of the 'cut-off', and before the steel liner was installed, a pressure test was conducted by erecting a test bulkhead at the upstream end of the grouted section and then filling the system. The inflow into the shaft was 0.23 m^3/s and the leakage from the tunnel was 0.21 m^3/s while the head was maintained at 213 m. Leakage from the tunnel was thus recharging the country drained during construction. Of this leakage approximately 0.01 m^3/s flowed around the cut-off and entered the then unlined section of the tunnel over a length that extended for 76 m downstream of the cut-off. In order to reduce this leakage, and to consolidate a cylinder of rock immediately around the lining, grouting was carried out in two stages at 1 400 kPa and then at 2 800 kPa in 30 mm

to 715 m probably also provides a leakage path from the tunnel to the joint systems in the main sandstone bed, which at this location is below the tunnel. The upper limit of the slip was located within the contact zone of the major sandstone bed with the base of the talus, and springs occurred above the contact of the lower boundary of the bed with the base of the talus.

Inspection of the tunnel after the slide took place indicated a substantial flow back into the tunnel, and on 24th May, seven weeks after dewatering, the inflow was 0.04 m^3/s. Most of the water entered the tunnel in the vicinity of the major fault at ch. 709 m, with the remainder entering via the weep holes drilled into sandstone beds and via joints and faults in the dolerite in the inclined tunnel.

5 RAINFALL

The rainfall pattern for the area is shown on Fig. 3. The commissioning period from December, 1972 to April, 1973 was wet, but the following months were wetter and this produced no significant change in groundwater activity or ground movement. Clearly the landslip was not the result of exceptional rainfall.

6 CORRECTIVE MEASURES AND INVESTIGATION

It was suspected early that leakage from the tunnel was causing sufficient uplift on the talus at the Permian surface to initiate the slip.

TOTAL YEARLY RAINFALL (mm)

YEAR	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973
RAINFALL	1969	1397	2331	1642	1604	1211	2064	1200	1881	2432	1309	2425

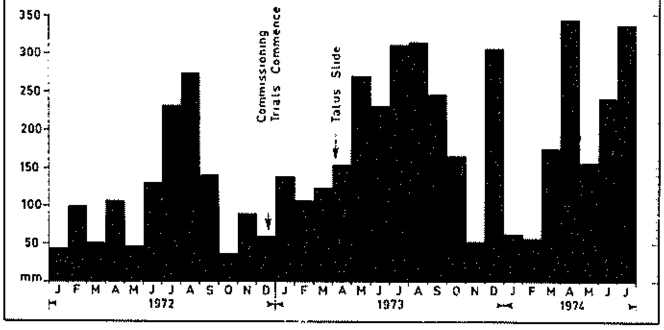

Fig. 3 Cumulative rainfall at Lake Mackenzie

Therefore, the tunnel and upper penstock were drained immediately and survey to supply movement data was commenced. Measures to prevent debris flow were started before investigation results were obtained. These included drainage behind the old construction slip area, building a crib wall and placing rock to prevent flow into the portal cut, collection and removal of surface water and filling of surface cracks.

A programme of investigation to determine subsurface conditions and the source of the water was undertaken using percussion and diamond drilling, seismic refraction surveying and tritium dating. A series of pump and recorder holes were drilled through the talus near the toe of the slip adjacent to the penstock. These were used to determine the permeability of the talus as a guide to the feasibility of drainage into an adit and of drainage by siphoning and pumping. The holes were then pumped continuously to lower the water table and reduce uplift pressures near the penstock.

Because of the expense and the uncertainty as to the mechanism of slippage placing of rockfill to control the whole slide was not considered practical and drainage was deemed to be the most suitable corrective method. In view of the saturated nature of the talus, surface trenching was considered impractical and likely to be ineffective if the water flow was mainly through the bedrock. Considering the mass of material moving, it was decided to commence an adit (Fig. 4), not as usual within the slide with the object of reducing weight in the sliding mass, but in the underlying bedrock with the object of creating a barrier to waterflow across the head of the slip by draining the talus and the talus-bedrock interface by holes drilled from the surface to the adit. The adit would also provide a means of draining the Permian sandstone beds if these could be shown to be carriers of tunnel water. Because of the urgency, the adit was driven on two faces by two shifts.

The drainage curtain through the talus was established using a Mayhew Down-Hole Hammer. The system adopted for most holes was to penetrate the talus with a 150 mm diameter hole drilled 1 m into bedrock and to install steel casing. A 115 mm diameter hole was then drilled through the bedrock to adit level. Slotted PVC casing of 64 or 77 mm diameter fitted with centralising collars was installed to the bottom of the hole and filter material (10 mm chips) was placed between the casings in the talus section while the outer casing was progressively withdrawn.

7 RESULTS OF INVESTIGATIONS AND CORRECTIVE MEASURES

(a) Bedrock Profile and Talus Thickness

Because there was little velocity contrast between the water saturated talus and the Permian bedrock, the seismic survey was unsuccessful in delineating the bedrock profile and data was obtained by drilling. Beneath the penstock the bedrock surface has a 1^{o} slope developed on mudstone and siltstone, but a steep change in slope occurs along the line of the subsurface downslope limit of the main sandstone bed (Fig. 1), and above the change the surface has a 10^{o} slope developed on sandstone.

The thickness of the talus is greatest below the change of bedrock slope where it reaches 27 m at Drill Hole 35, and above the change the thickness ranges from 7 to 15 m. There is a confining ridge of bedrock south of the tunnel, which was not located by the original investigation for the tunnel portal.

Drilling clearly defined the sub-surface position of the moderately weathered, open jointed major sandstone bed and showed it to be essentially sub-horizontally bedded and to be underlain by fresh, tight intercalated sandstone and mudstone. Separate water tables were found in the talus and the bedrock.

(b) Pumping Tests

An average value for the transmissibility of the talus of 6.4×10^{-1} m^3/day/m (permeability 6×10^{-8} mm/s) was obtained by pumping from holes drilled near the penstock. The results of continuous pumping were variable but for over half of the area pumping had little effect on the level of water in holes 3 m away.

(c) Talus Drainage Curtain

All but three of the holes draining the talus into the adit are only damp or drip steadily, and the initial flows of two of the other three holes were much reduced by holes drilled into bedrock from the adit. The flow from the holes responds slightly to rainfall but not to tunnel filling. Jetting and surging to improve hole performance produced no improvement, and indicates a permeability similar to that of the talus near the penstock.

(d) Test Filling

The system was refilled on 13th September, 1973 and dewatered on 25th September when it was clear that the water level could rise to the base of the talus over a considerable area. A plot of water levels in some of the holes recording levels in the bedrock is shown on Fig. 5. It shows the marked and rapid rise following filling and the slow fall subsequent to dewatering.

The mechanism causing the landslide was then clear. Continued leakage from the tunnel raised the water table in the bedrock joint systems such that near artesian conditions were created owing to the presence of an overlying blanket of talus of low permeability. The pore pressures generated were sufficient to initiate basal shearing, presumably along the talus bedrock interface.

(e) Refilling

The system was refilled on 3rd January, 1974.

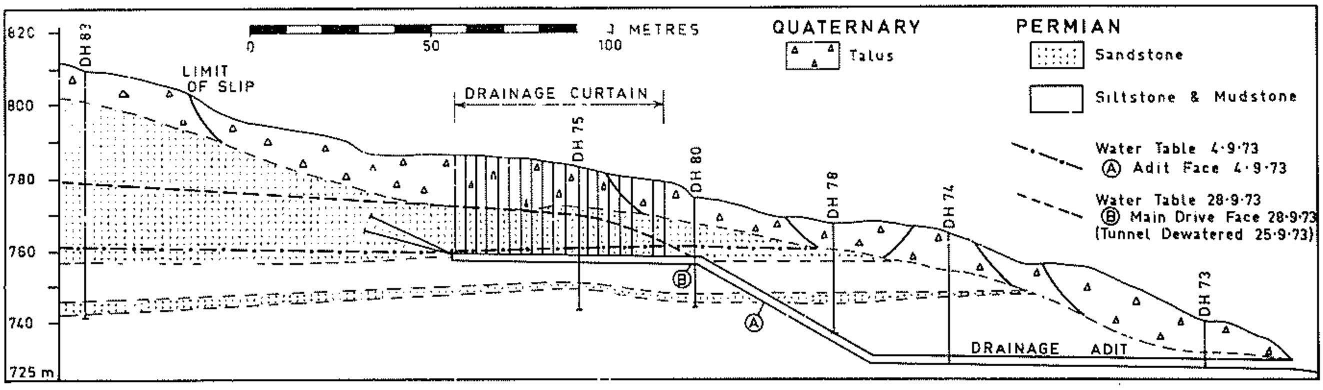

Fig. 4 Geological section along the main drive of the drainage adit

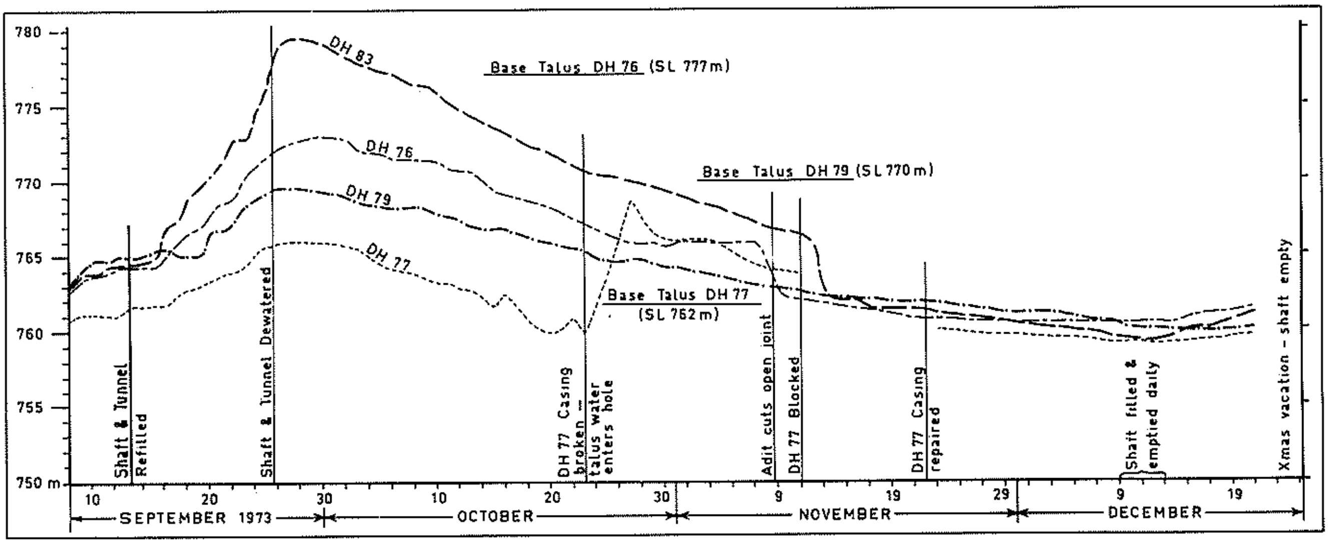

Fig. 5 System test filling - plot of water levels in some of the drill holes recording water movement in the Permian bedrock

As shown on Fig. 6 the level in each recording hole reached a maximum, then remained relatively steady with a flow of about 270 l/min into the adit. A spring (Fig. 1) appeared upslope from the position of the north branch of the adit on 30th January, just before the maximum level was reached, and this has built up and maintains a flow of 70 l/min.

Aditing ceased on 18th February and activity was then concentrated on improving the flow into the adit by drilling holes from the adit to intersect open joints. A series of 6 m long blast holes were drilled from the spring line to intersect any perched water. Some produced good flows but most were only damp. Diamond drill holes up to 30 m long were drilled upwards and downwards. These produced good flows and raised the flow into the adit to 410 l/min. It is interesting to note that it was the last series of holes, those drilled downwards into a minor sandstone bed, that produced the most dramatic fall in level in the recorder holes and lowered the level in holes downslope from the adit branches to below the base of the talus.

(f) Ground Movement

The maximum movement of the penstock after the initial slip was 9 mm downhill and a rise of 6 mm in level. The horizontal movement varied within a 6 mm band width and the vertical movement within a 4 mm band, but the variations possibly reflect soil moisture and pipe temperature changes.

(g) Tritium Dating

The results of tritium analyses of water samples by the Australian Atomic Energy Commission are given in Table I.

TABLE I

TRITIUM ANALYSES OF WATER SAMPLES

Sample	Tritium Units
Lake Mackenzie; June, 1973	16.4 ± 0.4
Tunnel ch. 700 m; upstream of fault - from Permian sediments; June, 1973	6.7 ± 0.3
Tunnel ch. 720 m; downstream of fault - from Permian sediments; June, 1973	14.9 ± 0.4
Talus spring above slide; June, 1973	25.7 ± 0.4
Adit ch. 150 m; September, 1973	1.7 ± 0.3
Adit ch. 170 m; October, 1973	1.6 ± 0.4

The weighted mean tritium content of rainwater at Hobart in 1972 was 25.0 tritium units. Water originally having such an average tritium content would have to be removed from the hydrological cycle for 12.3 years to give a present tritium content of 12.5 tritium units, because the half

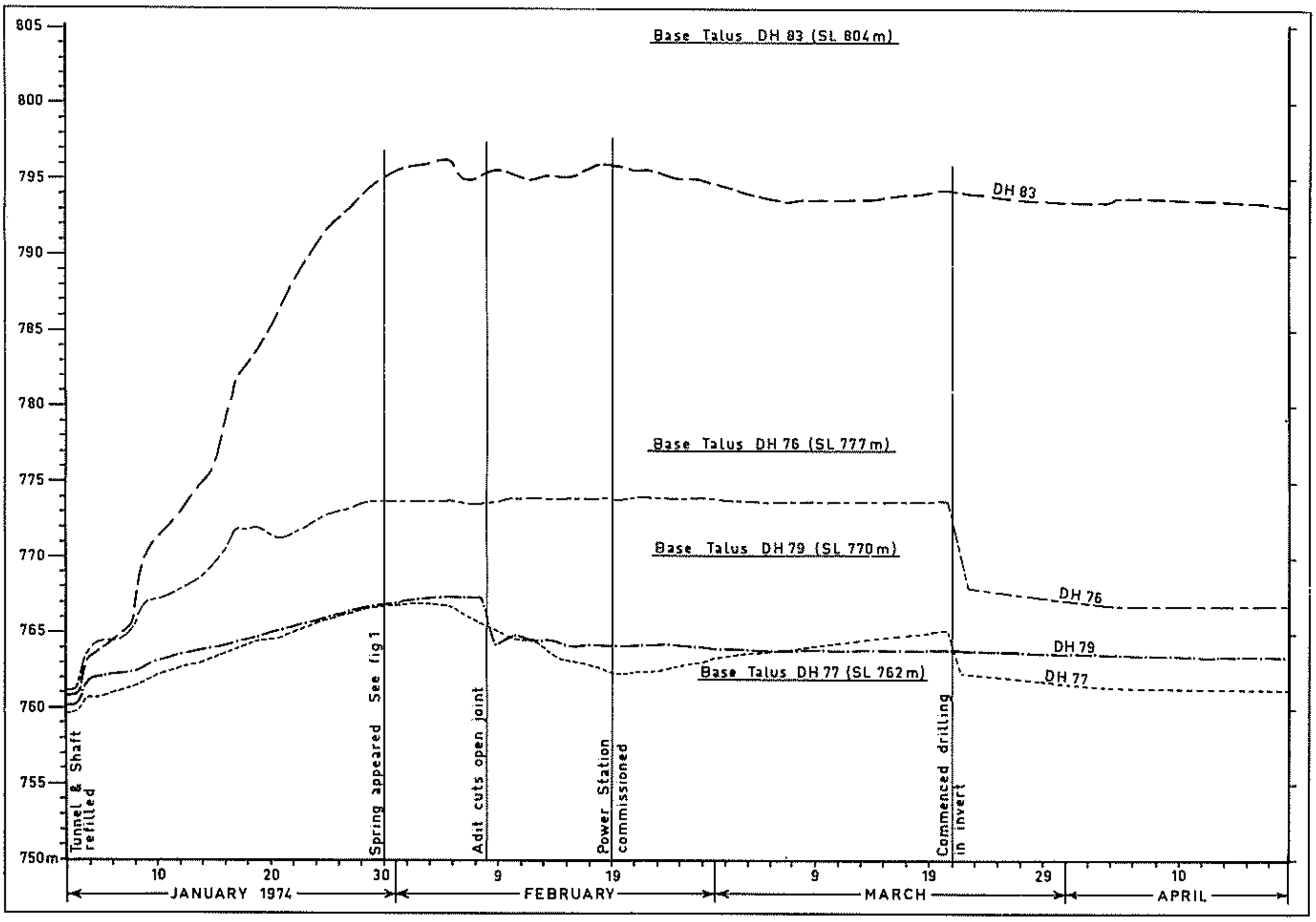

Fig. 6 System re-commissioning - plot of water levels in some of drill holes recording water movement in the Permian bedrock

life of tritium is about 12.3 years. The results, therefore, indicate that the water providing uplift under the talus has been removed from the hydrological cycle for considerably longer than either that in the nearby springs (rainwater) or the water supplied to the tunnel from Lake Mackenzie or from the Permian rocks. From these observations it is deduced that although tunnel pressure was transmitted to the slide area rapidly after the shaft was filled, there is no evidence that tunnel water had then reached the slide.

(h) Stability

No tests were carried out on the material thought to have sheared at the base of the talus. However according to Skempton and DeLory (Ref. 3) the factor of safety against sliding for a continuous slope can be expressed as:

$$F = \frac{c' + (\gamma - m\gamma_w) z \cos^2\beta \tan\phi'}{\gamma z \sin\beta \cos\beta}$$

where β is the slope of the slip surface; c' is cohesion; γ is unit weight of soil; γ_w is unit weight of water; z is depth to slip surface; mz is height of water above slip surface and ϕ' is angle of shearing resistance.

For a cohesionless material this becomes:

$$F = \frac{\gamma - m\gamma_w}{\gamma} \frac{\tan\phi'}{\tan\beta}$$

For the average bedrock slope of 10° in the slip area, sliding would occur (F = 1) if $\phi' = 20^\circ$ with the water table at the surface (m = 1) or if $\phi' = 30^\circ$ with artesian conditions (m = 1.33). These conditions are consistent with the observations that the talus is relatively impermeable and the bedrock joints are open.

8 CONCLUSIONS

The slide probably occurred as a shear failure at the base of the talus initiated by development of near artesian pore pressures, which resulted from a substantial rise in the water table following upon leakage of tunnel pressure water and transmission of system pressure via joints and faults in the Permian sandstone beds. The results of the tritium analyses of water samples suggests that although system pressure was transmitted rapidly, the actual transference of tunnel water was slow and had not reached the slip area by 3rd October, 1973. This reflects the general low primary and secondary permeability of the sandstone beds and explains the fact that although leakage from the whole tunnel was 0.1 m^3/s when the slip occurred, removal by the adit of only 410 l/min has lowered the piezometric line below the base of the talus. The slip is considered to have been stabilized and no other remedial measures are contemplated.

9 REFERENCES

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