8 SUMMARY

1. The damage arising from Cyclone bola at Mangahauini, required a co-ordinated approach to the slope stabilisation with deep drainage, ground shaping, control of surface drainage, construction of walls, and toe loading.

2. The initial recommendations for surface water drainage and counterfort drains could not be completely followed because excavation of deep trenches proved impractical in the lower parts of the landslides.

3. The surfaces of the slip prone areas were completely reshaped with access tracks, benches and surface drains. The slope stabilisation is based generally on control of the ground water level with subsurface blanket drains at three to four metres deep.

4. Investigations with boreholes, test pits and static cone penetrometer tests provided data from across the slip prone areas which allowed the identification of potential slip surfaces and hard/soft interfaces. It was found that recent slips were relatively shallow and strongly influenced by high ground water levels.

5. The installed drainage system taps many of the old springs and is responsive to rainfall. This method proved to be a very economical way of stabilising the slip prone areas.

6. Slope analysis, using the back analysis of failed slopes, indicated that parameters of $C' = 0$ and $\Phi' = 25°$ are the most probable failure parameters; these parameters were applied to other slopes in the areas to find 'safe' ground water levels.

7. Revegetation of the disturbed ground was an important factor in maintaining stability and recommendations for planting were obtained from the Soil Conservation Centre of DSIR.

8. Observation and monitoring of the piezometers over a period of eighteen months has shown that, to date, the drainage installations appear to have been effective.

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REFERENCES

Bulletin No 331. Lime slurry pressure injection bulletin. National Lime Association, Arlington, USA

Greenaway, D.R. 1987. Vegetation and slope stability. Chap.6 Slope stability. Ed. Anderson and Richards. John Wiley & Sons.

Janbu, N., Bjerrum, L. and Kjaernsli, B. 1956. Soil mechanics applied to some engineering problems, Norwegian Geotech. Inst. Publication No 16

Mitchell, K.D., 1991. Deep lime stabilisation applications for roading, East Coast, North Island, New Zealand. Unpublished M.Sc. Thesis, University of Canterbury.

Robnett, Q.L. and Thompson, M.R. 1975. Alteration of soil properties as affected by various lime treatment procedures. Roadbed stabilisation; Lime injection conference. Arkansas, USA

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Jackson Creek landslide stabilisation, New Zealand

M.D.Gillon
Works Consultancy Services, Wellington, New Zealand

C.K.Anderson
Barrett Fuller & Partners Ltd, Wellington, New Zealand

G.S.Halliday
Works Consultancy Services, Cromwell, New Zealand

C.R.Watts
Duffill Watts & King Ltd, Cromwell, New Zealand

ABSTRACT: This paper describes the investigation, design and construction of the stabilisation measures adopted for the active downstream segment of the Jackson Creek landslide situated within the Clyde dam reservoir, New Zealand. The Jackson Creek landslide is one of a number of landslides being stabilised by the Electricity Corporation as part of the Clyde Power Project, New Zealand. It is a 5 million cubic metre, scarp slope, schist landslide that posed wave and blockage hazards to the proposed reservoir. The principal stabilisation measures were a 0.7 million cubic metre buttress and drainage of the groundwater systems from drives.

The investigations permitted a comprehensive assessment of the critical slide parameters, ie. geometry, piezometric pressures and shear strength. During buttress construction unexpected movements of the slide occurred. The subsequent engineering assessment highlighted that the Cromwell Gorge landslides may be sensitive to small changes in stability.

1 INTRODUCTION

The Jackson Creek landslide is located in the Cromwell Gorge in the Central Otago region of New Zealand (Fig. 1). It is one of a number of landslides which will be affected by the filling of Lake Dunstan, behind the recently completed Clyde dam. A general description of the Cromwell Gorge geology, landslides, and the engineering works being carried out, is given by Gillon & Hancox (1992).

The Jackson Creek landslide area is located 7 km upstream of the new Clyde dam and contains two segments which directly threaten the proposed Lake Dunstan reservoir. The active downstream segment, a 4.3 million m^3 schist debris landslide, is described in this paper.

Early in 1989, geological and geotechnical investigations on this segment identified a potential hazard to the proposed reservoir with an unacceptable risk of failure after lake filling. The landslide dam or creating a wave that would overtop the Clyde dam if rapid failure occurred at full lake level.

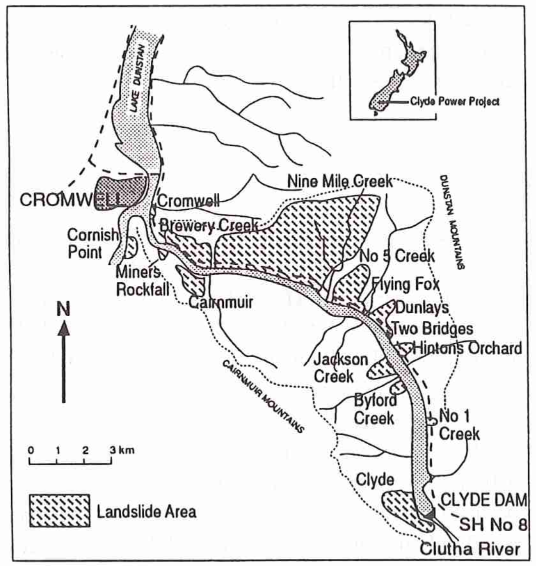

Figure 1. Location diagram

Stabilisation measures were deemed necessary prior to lake filling. This paper describes the process which led to the selection of the adopted measures of buttressing and drainage. Also described is the movement incident which occurred during buttress construction.

2 GEOLOGICAL AND GROUNDWATER MODEL DEVELOPMENT

2.1 *Investigations*

The slide geology was investigated using a combination of geological mapping, geophysical surveys, core drilling and drives as described in Gillon, Denton & Macfarlane (1992).
— the geology and geomorphology was mapped at 1:2000.
— seismic refraction traverses totalled 4 km.
— downhole geophysical logging was utilised in some holes.
— cored investigation holes totalled 1960 m.
— the drainage/investigation drives totalled over 450 m.
— 40 standpipes and 1 string of Westbay piezometers were installed in 29 holes.
— 2 km of resistivity surveys were carried out.
— isotope, temperature and chemical analyses were carried out on the groundwater.
— deformation was monitored by 15 inclinometers and 21 surface pillars.

2.2 *Geological Model*

The downstream segment is an active translational feature consisting of a layer of schist debris sliding on bedrock (Fig. 2). The slide is up to 46 m thick, and the ground surface has an average slope of 30°. It occupies the floor of a large topographical depression and appears to be the remnant of a former larger feature.

There is clear geological evidence of current slide activity. Fresh active scarps extend around the slide perimeter. Internal scarp systems are also present and divide the slide into a series of lobes. The slide toe (lobe 1) has overridden the last glacial advance outwash terrace, indicating at least 30 m of movement in the last 16,000 years.

Current movement rates have been determined using surface pillars and inclinometers (Gillon, Foster, Proffitt & Smits 1992). The subsurface geometry of the active failure surface has been defined by the zones of inclinometer deformation.

The landslide material consists of chaotic schist debris, with fragments ranging from fine gravel to large blocks, tens of metres in diameter. The active failure surface lies within a 0.5 - 2 m thick zone of crushed schist material, along the slide base. The crushed material is generally a mixture of fine gravel, sand, silt and clay with discontinuous thin lenses of clayey-silt gouge.

The basement rock is textural zone IV schist consisting of alternating laminae of quartz/feldspar and mica. Foliation dips at 20 - 40° into the slope beneath the lower part of the slide. The attitude changes abruptly to a 20 - 40° downstream dip above a fault that trends beneath the slide at midslope (Fig. 2).

Bedrock geology has controlled slide development. The slide is believed to have originated as a rock slope failure along pre-existing tectonic defects. Crushed zones dipping obliquely out of the slope have provided a wedge failure mechanism in the lower part of the slide.

Another series of faults and crushed zones dip at 20°-40° into the slope beneath the slide base (Fig. 2). These strongly influence the fundamental watertable as described below.

2.3 *Groundwater model*

Springs existing at the toe, and piezometer measurements indicate that a perched watertable lies in the trough of the slide, above the low permeability basal crushed zone. The watertable rises steeply across the rear of the lobe 1, due to the ponding of water behind the lobe failure surface (Fig. 2).

The sub-basal watertable in the schist lies close to the slide base. Vertical steps occur across those faults dipping into the slope. Local confinement is present beneath the slide base above faults at mid-height of the slide.

3 GEOTECHNICAL STABILITY ANALYSIS

3.1 *Analysis methods*

The principal algorithm used for stability analysis

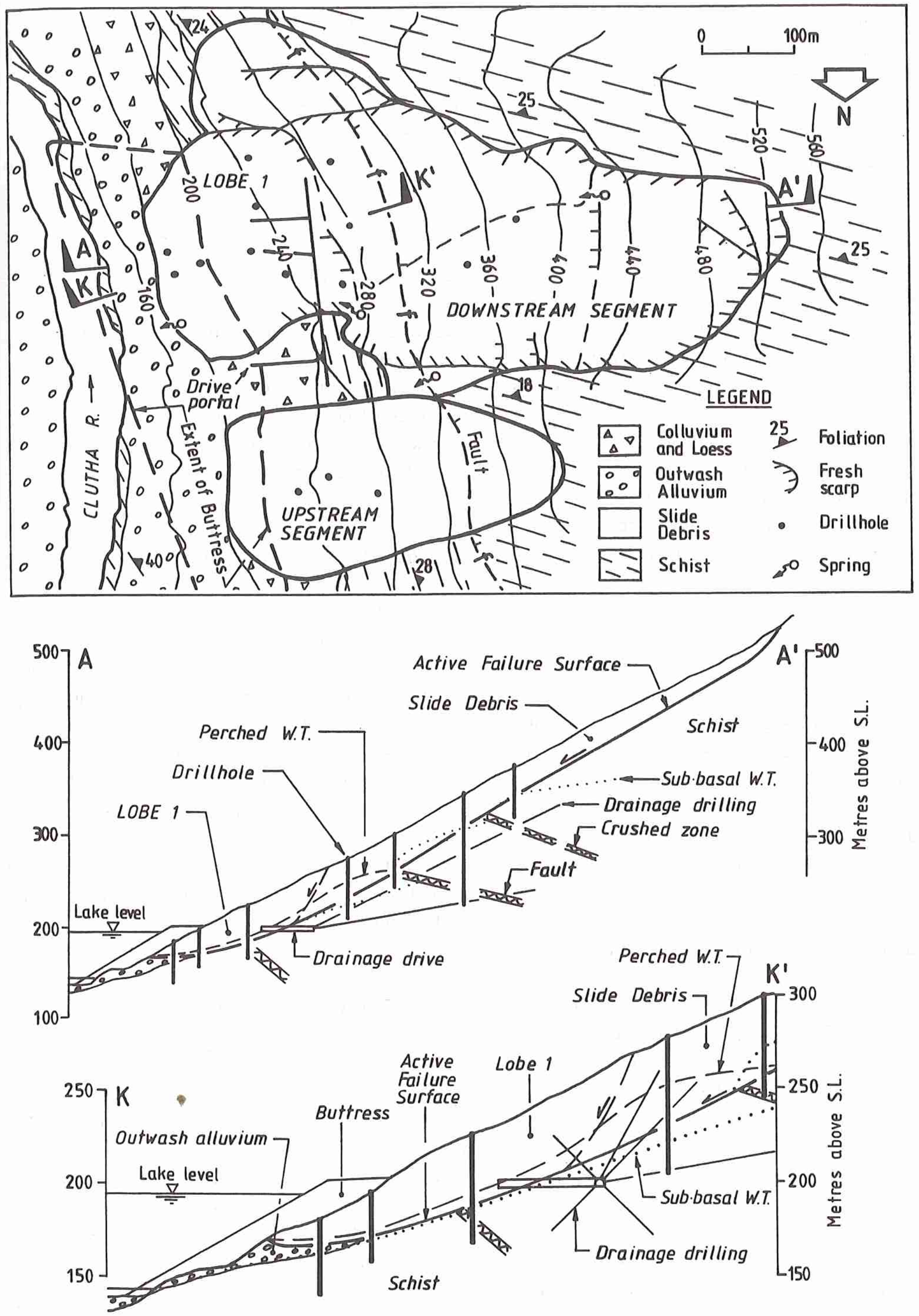

Figure 2. Geological map and cross sections

was that of Sarma (1975), with checks being performed using the Spencer algorithm (Spencer 1967). Relative factors of safety were used to assess the magnitude of stability changes between load cases, to reduce uncertainties in accurately predicting absolute factors of safety.

3.2 *Selection of analytical model geometry*

The analytical model geometry closely follows the geometry of the slide determined during the geological investigations. Critical sections aligned with slide movement direction were selected for analysis. Section KK′ and AA′ in Fig. 2 are representative of lobe 1 and whole slide stability respectively. Critical failure surfaces were defined by inclinometer movement. Both active and inactive surfaces were examined to determine relative risks of long term movement.

3.3 *Piezometric surfaces*

The original distribution of piezometric pressures was inferred from the geological investigations and standpipe water levels. Post-lake predictions of piezometric pressures were made using simple assessments of the likely rise in the sub-basal and perched groundwater systems, for different degrees of drainage and positions of flow boundaries. Computer modelling was limited to the sub-basal system. Modelling of the perched system was not considered practical due to the complex distribution of aquicludes.

Critical combinations of piezometric levels from the perched and sub-basal systems were used in analysis. Sensitivity studies were made to determine relative effects of different piezometric levels on stability.

3.4 *Shear strength assessment*

Landslide movement during buttress construction defined the failure surface geometry and enabled accurate back analysis of lobe 1. The average effective stress parameters were $c' = 0$, $\phi' = 24.8°$.

These results compare closely with those from shear box tests used to measure residual strengths on undisturbed samples of the crushed zone from the active failure surface ($c' = 31$ kPa, $\phi' = 20°$ and $c' = 10$ kPa, $\phi' = 23.5°$). The small difference between back analyses and laboratory shear strengths is attributed to waviness and asperities.

3.5 *Loading cases*

The load cases analysed consisted of the following basic types:
— pre-construction, pre-lake
— pre-construction, post-lake
— post-construction, pre-lake
— post-construction, post-lake

3.6 *Results of stability analysis*

The results of the stability analysis are given in Table 1, which illustrates the following points:

— Post-lake stability of the downstream segment without stabilisation could have been reduced by 25% to FoS = 0.75. Failure would have occurred.

— A large 15% post-lake improvement of lobe 1 stability has been created by buttress and drainage but the total effect on the whole slide is much smaller.

— Without drainage control of the sub-basal groundwater system low levels of post-lake stability could result.

4 REMEDIAL MEASURES

4.1 *Design philosophy*

The design philosophy for the remedial measures was to:
— offset effects of lake filling
— prevent confinement of sub-basal watertable
— include allowance for uncertainties
— reduce surface infiltration

This philosophy resulted in the adoption of buttressing and subsurface drainage as the principal stabilisation measures.

Table 1. Results of stability analysis

		Pre-Lake			Post-Lake		
		Original State	Buttress Only	Buttress + Drainage	Original State	Buttress Only	Buttress + Drainage
Lobe 1	Absolute FoS	1.00	1.30	1.36	0.75	1.01	1.15
Whole Slide	Absolute FoS	1.00	1.09	1.13	0.88	0.93	1.06

4.2 *Buttressing*

The principal purpose of the buttress was to stabilise the lower portions of the slide, particularly lobe 1 which was known to be moving independently of the rest of the slide mass. Drainage for lobe 1 was not a feasible option, as the pre-lake pressures were too low to provide a nett increase in stability post-lake. The size of the buttress was based on stability assessments, with a nett increase in stability to allow for the geological and modelling uncertainties. This resulted in a buttress with a nett volume of 0.7 million m^3 (40 m crest width, 60 m high, 400 m long) which included buttressing of the upstream slide segment.

Buttress materials were free draining alluvial gravels excavated from the Clutha River terraces upstream of the slide.

4.3 *Drainage*

A series of drives was excavated below the slide in sub-basal rock, with two stubs being excavated back through the slide basal surface (Fig. 2). The drives provided access for the drilling of drainholes and also provided valuable investigation data on sub-surface geology.

Drainage was installed to lower and control both sub-basal and perched slide piezometric pressures. Sub-basal drainage was achieved by the drilling of a regular pattern of 30 m holes off the main drive (diagonals at 10 m centres), and the drilling of holes up to 280 m long into specific target areas identified from the geological investigations. Drainage of the perched watertables was achieved by targeted drilling from the cross drive and stub drives up into the slide mass.

Control of the sub-basal system was very important, as the position and nature of aquitards was such that it could possibly permit development of confined groundwater systems after lake filling.

Drainage of existing piezometric heads acting on the slide could not produce large increases in stability, as these pressures were small. Jacksons Creek slide has aspects in common with the other low watertable slides in the Gorge, where the achievement of large stability increases by drainage is not possible.

5 EFFECT OF REMEDIAL MEASURES ON SLIDE MOVEMENT

5.1 *Movement history*

During construction of the buttress a series of movements took place on lobe 1, and to a lesser extent the upper slide mass, which illustrated the sensitivity of the Jackson Creek slide (and possibly other Cromwell Gorge landslides) to small changes in stability. These events led to a reassessment of the level of work required on the other slides in the gorge prior to lake filling. The sequence of events and calculated changes in stability are presented in Figure 3 and Table 2, and described in the following sections. Figure 3 illustrates lobe 1 deformation and velocity changes calculated from inclinometer and surface pillar data.

Prior to the start of buttress construction in April 1989 inclinometers indicated that the slide was moving at 1.4 mm/mth. The initial pre-construction

Table 2. Stability changes on lobe 1 during buttress construction

Factor of Safety	Construction Stage					
	Pre Const.	Remove 4000 m^3	Temp Butt RL 173 m	Remove 750 m^3	Temp Butt RL 175 m	Full Butt
Absolute FoS	1.00	0.98	0.99	0.98	1.00	1.36
Velocity mm/Day	0.05	8	1	2 - 3	decreased	ceased

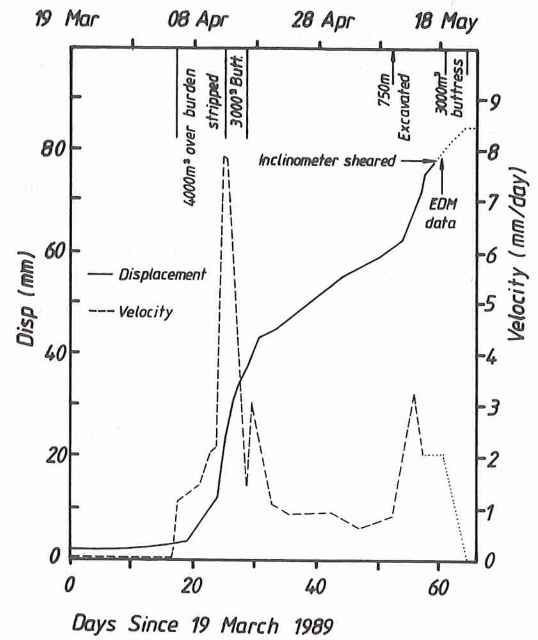

Figure 3. Displacement and velocity of lobe 1

stripping of loess and topsoil from the lower part of lobe 1 resulted in the removal of 4,000 m^3 of material from the slide toe. This was calculated to have decreased lobe 1 stability by 2% (Table 2).

Slide response was immediate. Within a few days movement rates increased to nearly 8 mm/day on lobe 1 (a 160 x increase in rate).

The engineering response was to place a 3,000 m^3 temporary buttress (RL 173 m) after quickly stripping failure zone material from the active surface. Replacement of weak shear surface material with higher shear strength alluvial gravels permitted a higher stabilising force to be mobilised. The small buttress is estimated to have restored slide stability to near the original state (Table 2). Movement rates on lobe 1 slowed quickly to 1 mm/day.

The slide continued to move at this rate for 3 weeks, until excavations to widen a haul road resulted in removal of a further 750 m^3, representing a 1% stability decrease. Movement rates increased sharply to 2-3 mm/day necessitating the addition of 3,000 m^3 to the temporary buttress (RL 175 m). At this stage the inclinometers on lobe 1 sheared, movement rates after this time are based on monitoring of the surface pillars by EDM, which had a lower level of accuracy. These surveys indicated a major decrease in velocity following temporary buttress placement (Fig. 3).

When the main buttress construction reached the toe of the active portion of lobe 1 and began to act on it, movement essentially ceased. Total movement measured during this phase was 83 mm on lobe 1.

An increase in velocity also occurred in the upper part of the slide, but it was less than on lobe 1. This resulted in a tension zone between the two areas. The upper slide mass continued to move at 0.5 - 2 mm/mth for the next two years until the tension zone was closed.

5.2 *Groundwater response to drainage*

The overall response of the perched groundwater system to drainage has been one of rapid initial drawdown, followed by more gradual reductions in piezometric head with time.

Achieving effective areal drainage of the slide volume upslope of lobe 1 has been frustrated by the compartmentalisation of the perched groundwater system. Internal shear zones form aquitards, which appear distributed at random orientations within the slide mass. Drainage drilling sometimes produced good drainage flows, but with no response from nearby piezometers.

Drainage of the sub-basal rock proved to be much easier than the internal slide mass. Overall permeability of the rock mass is governed by rock joints and numerous faults dipping into the slope.

6 CONCLUSIONS

The principal conclusions drawn from the current work on the Jackson Creek landslide were:

(i) Without stabilisation, failure of the down-stream slide segment would have occurred as a result of filling Lake Dunstan.

(ii) Movement rates of the active Jackson Creek landslide are very sensitive to small changes in stability, and the same may be true of other Cromwell Gorge schist slides.

(iii) Careful field investigations and testing of shear surface material can produce good correlations between back analysed field strength and laboratory strength testing.

(iv) The residual effective field strength for the Jackson Creek slide failure surface is approximately $c' = 0$, $\phi' = 25°$. Laboratory strengths ranged from $c' = 10 - 31$ kPa, $\phi' = 20 - 24°$ with the difference attributed to asperities.

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REFERENCES

Gillon, M.D., B.N. Denton, and D.F. Macfarlane 1992. Field Investigation of the Cromwell Gorge Landslides: *Sixth International Symposium on Landslides*, Christchurch, New Zealand.

Gillon, M.D., P.F. Foster, G.T. Proffitt, and A.P. Smits 1992. Monitoring of the Cromwell Gorge landslides: *Sixth International Symposium on Landslides*, Christchurch, New Zealand.

Gillon, M.D., and G.T. Hancox 1992. Cromwell Gorge Landslides - a general overview: *Sixth International Symposium on Landslides*, Christchurch, New Zealand.

Sarma, S.K. 1975. Seismic stability of earth dams and embankments. *Geotechnique 25: No. 4.*

Spencer, E. 1967. A method of analysis of the stability of embankments assuming parallel inter-slice forces. *Geotechnique 17: No. 1.*