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An innovative solution to the stabilisation of the Cairnmuir Landslide incorporating reinforced earth

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ABSTRACT: Cairnmuir Landslide is an active schist rock slide, with a volume of 8 million m3, located above Lake Dunstan, the Clyde Dam reservoir in the South Island of New Zealand. Slide velocities of up to 180 mm/year have been measured. Despite underground and surface drainage measures, the slide kept moving, particularly when wet, necessitating surface infiltration protection over an area of 3.8 ha in the toe region of the slide. A unique combination of Reinforced Earth terraces with bitumen sealed runoff surfaces was implemented. The Reinforced Earth technology provided ease of construction, flexibility to follow ground contours, deformation tolerance and natural appearance. Post-construction performance has confirmed the effectiveness of the solution.

1. THE CAIRNMUIR LANDSLIDE

1.1 *Description*

Cairnmuir Landslide is one of a number of landslide areas which required stabilisation measures as part of the Clyde Power Project development (Brown, Gillon & Deere, 1993). The landslide is located on the right bank of the Lake Dunstan reservoir, 15km upstream of the Clyde Dam.

The central portion of the landslide is active and slide movement in response to rainfall has been observed on a number of occasions. The volume and activity of the landslide is such that it was considered necessary to implement stabilisation measures which would isolate the slide from the effects of lake filling and limit deformation response to rainfall in the long term. The stabilisation of the landslide is described in detail by Gillon and Saul (1996)

1.2 *Slide Description and Geology*

The active segment is a relatively planar rock slide 500m wide and 650m long, covering an area of 28 hectares and comprising 8.3 million m^3 of debris. The inclination of the slide surface varies from 20 degrees mid slope to more than 35 degrees in the head and toe.

A full description of the geology of Cairnmuir Landslide is given by Watts and Macfarlane (1996). The slide mass is up to 70m deep and comprises chaotic schist landslide debris (Figure 1).

The main active failure surface is a discrete, slickensided sandy silty clay gouge, 100 to 300mm thick, and is located at the top of a Basal Failure Zone. Multiple failure surfaces within the landslide debris correlate with surface scarps in the Frontal Lobe Area. Prior to drainage, groundwater was both perched on and confined beneath the Basal Failure Zone. Low permeability crushed zones and the Frontal Lobe failure surfaces acted to compartmentalise groundwater in the slide debris while tension zones and sinkholes on the surface provided permeable paths for infiltration.

1.3 *Landslide Movement*

Geological features indicate that the slide has moved at least 600m since initiation. Movement since the most recent glacial outwash terrace was deposited below the landslide toe, approximately 16,000 years ago, is inferred from accumulated debris to be some 30m in the toe. Aerial survey data interpretation indicates total movement between 1949 and 1991 of 2m in the head of the slide and 4m mid slope.

The annual rainfall in the area is 400mm and rainfall initiated movement episodes have been observed in response to rainfall events of 20 to 50mm in 1 to 3 days, which have average recurrence intervals of 2 to 5 years. Total observed movements during individual episodes have been up to 100mm in the toe and 10mm in the head of the slide with rates of up to 3.3 and 0.5 mm/day respectively. Piezometric and drainage flow responses to extreme rainfall events have been observed.

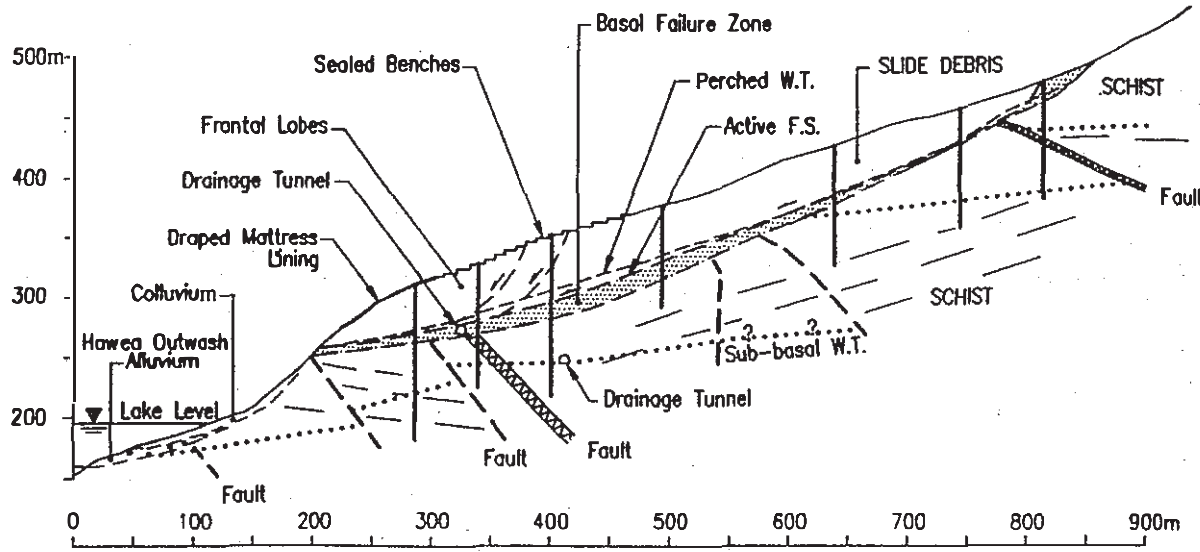

Figure 1. Cross Section of Landslide

2. STABILISATION STAGES

2.1 *Stage 1*

An initial stage of remedial works was implemented to isolate the slide from the potential effects of lake filling. Sub-basal drainage tunnels 600m in length were excavated prior to lake filling with 1300m and 4700m of drainhole drilling were installed to target the sub-based and perched aquifers respectively. Limited surface drainage works were carried out including the formation of some surface run off interception channels and filling of major areas of sinkholes and tension cracks.

Sub-basal drainage influences were detected up to 170m from individual drainholes and the sub-basal aquifers were drained and controlled by the tunnel and relatively small amount of drilling. Drainage of perched aquifers was only achieved within 5 to 15m of each drainhole. Perched ter within the slide mass was drawn down by up to 12.5m locally but up to 7.5m of water remained in the Frontal Lobe area of the landslide.

2.2 *Stage 2*

A second stage of remedial works was implemented to intensify perched aquifer-drainage and to improve surface drainage interception. An additional 2000m of subsurface drainage drilling targeting the perched aquifers was carried out and surface drainage improvements were instigated to fill all tension cracks and sinkholes.

Slide deformation response to a 30mm of rainfall in one day on 5 October 1992 indicated that the initial stages of stabilisation were not sufficient and that more works would be required to limit the frequency and extent of movement.

2.3 *Stage 3*

A third and final stage of stabilisation was implemented to reduce deformation response of the slide to rainfall by controlling perched water in the slide toe. The aim was to limit deformation to creep rates of less than 5mm/year.

The stabilisation work required both surface infiltration protection and more intensive subsurface drainage of the perched aquifers to minimise groundwater seepage into the sensitive toe region of the slide.

3. SURFACE INFILTRATION PROTECTION

The surface works are described in detail by Gillon and Saul (1995) and are summarised below. The works were implemented to limited infiltration into both the more active Frontal Lobe Area near the toe of the slide and the two main drainage gullies.

The development of practical measures to control infiltration required an innovative approach in order to achieve the project requirements without destabilising the slide and to provide a solution which minimised the visual impact.

The aim of the stabilisation measures was to limit deformation to creep rates of less than 5mm/year.

The requirements of the stabilisation method were:

- overall slide mass was to be unaltered
- tolerance of movement
- easily monitored and maintained
- safe working on the steep slope
- reduced visual impact from across reservoir

In addition, difficult construction access favoured simple methods with a minimum of imported materials.

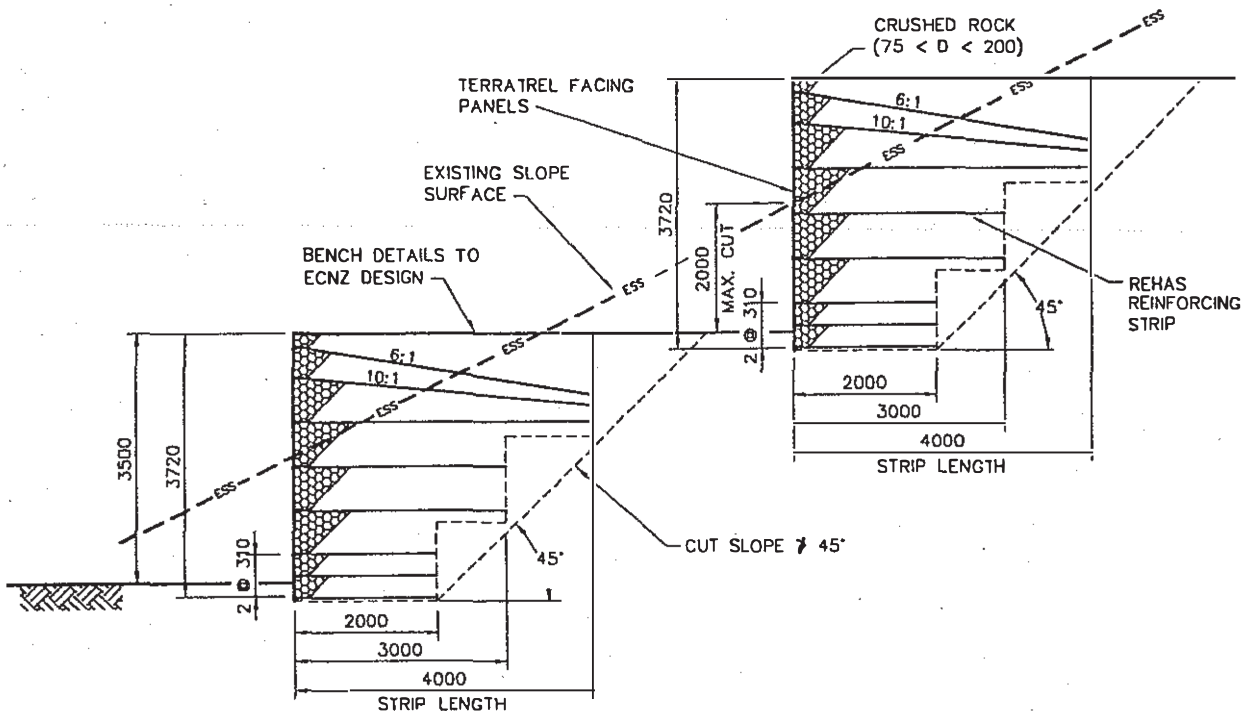

Figure 2 Cross section of Reinforced Earth terraces

Sealed benches were adopted in the 2.8 ha region of the Frontal Lobes (Figure 1) where the slope was typically 20° and up to 30° locally. Reinforced Earth walls, typically 3.5 or 4.2m high were formed with plain steel mesh facing panel (Figure 2). The benches were constructed from the bottom up for safety reasons and for stability.

A layer of coarse 100 to 200mm uniform rock was placed against the steel mesh to provide a natural looking finish and a steel flashing plate was located near the base of each wall to intercept rainfall penetrating into the wall face. The walls followed the original ground surface to preserve the mass balance.

The 5 to 15m wide benches between walls were shaped to contain and direct water for discharge to the adjacent gullies via 300 to 600mm diameter HDPE pipes draped over the ground. The surface of each bench was sealed with a specially developed bitumen geomembrane and a chip coating to prevent traffic damage. Walls were constructed using slide debris from the excavation for the wall above. A number of different fill material gradings were produced on site from the slide debris using nearby crushing plant and mobile skid mounted screens, towed behind the excavator. The essential elements of the Reinforced Earth terrace walls are shown in Figure 3.

Near the toe of the slide a 0.6 hectare area of the surface with slopes of 30 to 35° was protected with a 2.5mm thick HDPE geomembrane. The geomembrane was held in place with a gravel filled welded wire mattress (100mm thick) supported by cables and anchors. Wire cables were used to prevent the mattresses from sliding off the slope and were linked to grouted anchors using steel spreader beams.

The ground surface was unloaded by 300mm prior to construction to ensure a mass balance was maintained in the long term. Three cross slope run off interception drains were incorporated, discharging to an adjacent gully.

The two adjacent gullies receiving runoff from the protected slope areas were themselves lined with HDPE geomembrane so as to convey water off the slide area without infiltration. The geomembrane was overlain by Reno baskets. In steep areas, the Reno baskets were stabilised with anchored cables.

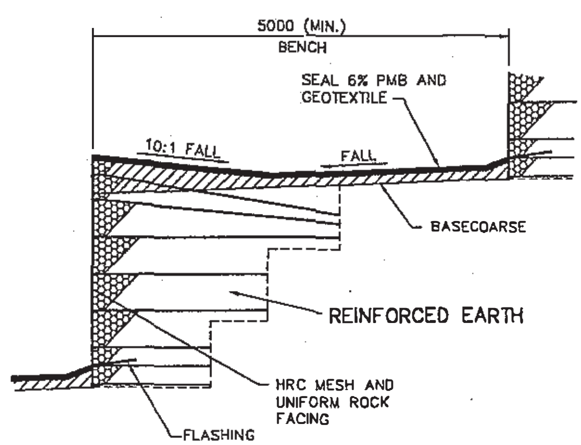

Figure 3. Details of Reinforced Earth terraces

4. THE REINFORCED EARTH SOLUTION

4.1 *System*

The Reinforced Earth system used for the terracing of the slope incorporated the steel mesh faced *Terratrel* system. This system was chosen on the basis of its strength and ability to tolerate deformation, as well as its speed of construction.

The *Terratrel* facing system comprises a steel mesh facing panel made up of 8mm bars at 100mm centres connected to Reinforced Earth's High Adherence steel (Rehas) strip reinforcement, 50 x 5mm in section. The mesh is backed by crushed rock (25 < D < 200mm) for an average depth of 500mm behind the face (Figure 4)

4.2 *Design*

Specific design of the Reinforced Earth structures was carried out by Reinforced Earth Pty Ltd in Australia, on behalf of Reinforced Earth Ltd in New Zealand. The design brief was prepared for the Clyde Power Project by the main design consultant, Works Consultancy Services Ltd. Specialist consultancy in relation to the seismic design of the Reinforced Earth structures was provided by Phillips and Wood Ltd in Wellington.

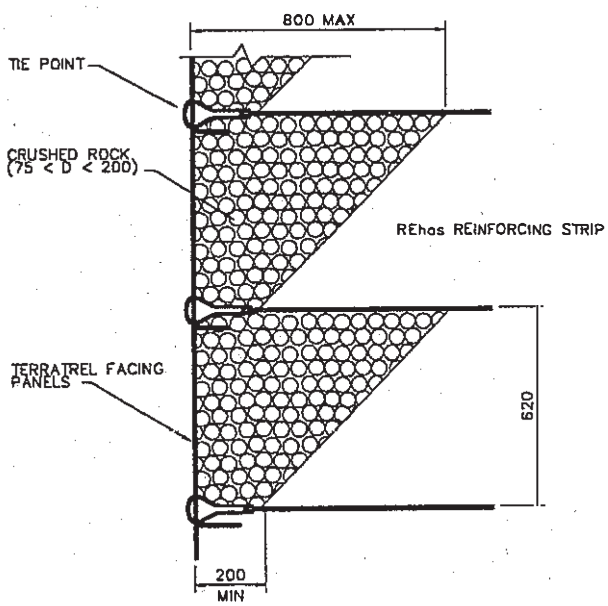

Figure 4. Details of *Terratrel* facing system

The structure was designed for a service life of 50 years. A corrosion allowance was incorporated into the steel elements of 2mm on exposed (black) steel and 1mm on buried (black) steel.

The configuration of the structures was dictated by the slope and the need to minimise excavation. Base width was kept to an absolute minimum to limit cut and to found each structure on existing material. This resulted in an exaggerated block shape (see Figure 2), which required specific analysis.

Wall heights varied from 7.4m to 2m. Total wall face area was 13,280m2.

The unusual configuration of the structures required that the individual wall design and the interaction of walls be analysed by appropriate global stability methods. For individual walls, the narrow base width (2.0m min) required particular attention. Reinforcing strips in this area were placed at closer (vertical) spacing to ensure that there was sufficient interception of potential failure surfaces. For adjacent walls and slopes, stability was checked for both construction (factor of safety 1.1 min) and long term (factor of safety 1.4 min) conditions. This is illustrated in Figure 5. These analyses were undertaken using a modified Bishop analysis which provides for the inclusion of reinforcements.

The structure was also checked for stability and deformation under peak ground accelerations of 0.197g and 0.256g for risk factors of 1.0 and 1.3 respectively. The effect of variation of the earth friction characteristics were also checked as the structures were to be built using the existing slide material, which potentially exhibited friction angles between 28 and 36 deg. See Figure 6.

For stability, the critical backfill friction angles were 33deg for ground acceleration of 0.197g (risk factor 1.0) and 36deg for ground acceleration of 0.256g (risk factor 1.3).

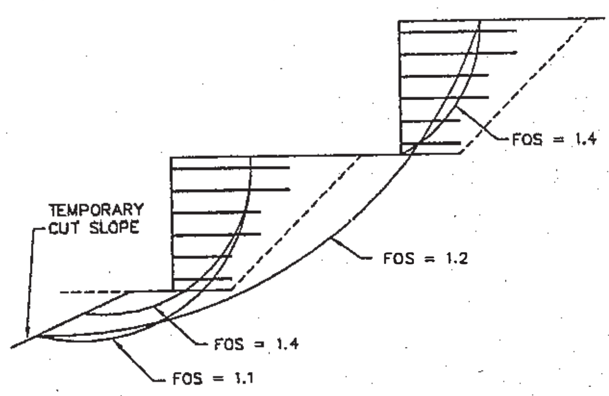

Figure 5. Global Stability Analyses

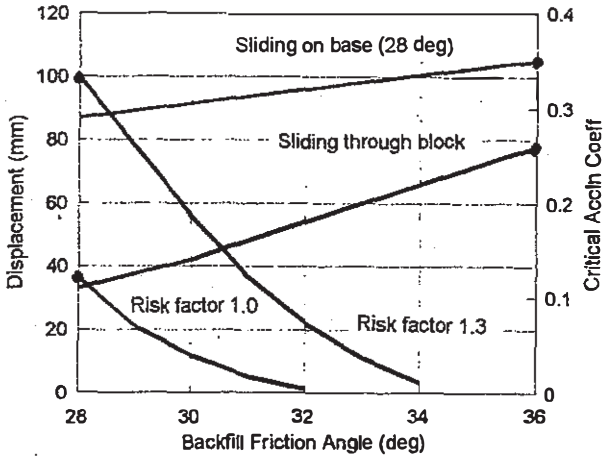

Figure 6. Dynamic Stability Analysis Results

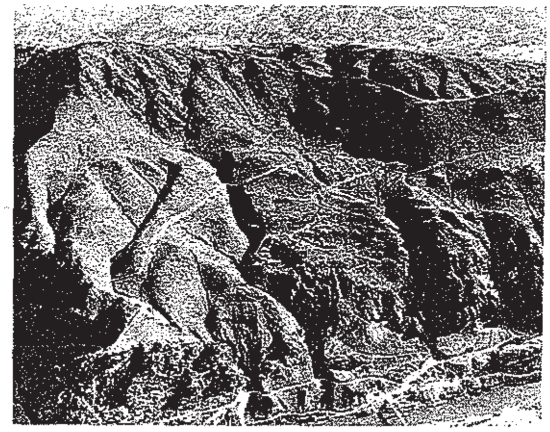
Photo 1. Aerial view of original slope

For deformation, no displacement of the structures was predicted for backfill friction angles of 32deg (at 0.197g) and 34deg (at 0.256g)

These results indicated that there was an acceptable range of backfill properties which could be accepted for the structures.

4.3 *Construction*

The supply of materials and construction of the works proceeded very rapidly due to the short construction period available. More than 2,000m2 of wall materials were delivered on site within three weeks of order, followed by 6,000m2 in the following month. This allowed construction to proceed at rates up to 364m2 per day with total completion of the 13,280m2 of walls in 17 working weeks. The walls were completed in mid-April 1994.

Aerial views of the original slope and the completed stabilisation works are shown in Photo 1 and 2.

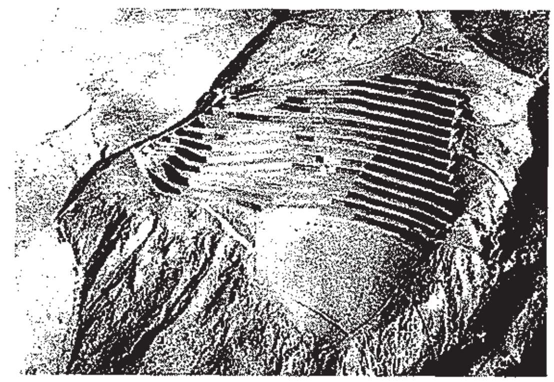
Photo 2. Aerial view of completed stabilsation works

5. PERFORMANCE

Since completion in June 1994, the slide area has been subject to several large rainstorm events, including an extremely wet period with severe flooding in December 1995. By the end of 1994, the slide had slowed to less than 5mm/year and has since continued to slow. There has not been any detectable increase in slide movement in response to rainfall since completion.

6. CONCLUSIONS

The stabilisation of the slide initially concentrated on sub-basal drainage with more extensive internal drainage being implemented as continued deformation response to rainfall was observed. Finally, a combination of extensive subsurface drainage and surface infiltration protection of the more active Frontal Lobe area in the slide toe was installed.

Reinforced Earth technology offered a means of building benches on the relatively steep slopes which were tolerant of displacement, as well as variation in fill and foundation materials. The system was flexible, enabling the ground contours to be followed easily and alignments to be adjusted to maintain mass balance of the slope. The open mesh facing and uniform rock fill zone behind resulted in a natural coloured appearance of the structure. The benches enabled a flexible geomembrane to be constructed using local materials, labour and technology. The benches facilitate in- spection and maintenance of the geomembrane.

The lack of slide movement in response to rainfall, combined with the limited and slowing creep movements (less than 5mm/year) observed since completion of the stabilisation works demonstrated their effectiveness.

7. ACKNOWLEDGEMENT

The permission of the Electricity Corporation of New Zealand to publish this paper is acknowledged.

REFERENCES

Brown, R., Gillon, M.D. and Deere, D.U. 1993. Landslide Stabilisation at Clyde Power Project, New Zealand. *ASCE Conference on Geotechnical Practice in Dam Rehabilitation,* Raleigh, North Carolina, April, pp 299-319

Gillon, M.D. and Saul, G.J. 1995. Cairnmuir Landslide Infiltration Protection stabilisation works. *Proc. IPENZ Annual Conference,* Palmerston North, New Zealand. pp 235-239

Gillon, M.D. and Saul, G.J. 1996. Stabilisation of the Cairnmuir Landslide,New Zealand. *Proc. Seventh Int. Symposium on Landslides,* Trondheim, Norway .

Watts, C.R. and Macfarlane, D.F. 1996. Engineering Geology of the Cairnmuir Landslide, New Zealand. *Proc. Seventh Int. Symposium on Landslides,* Trondheim, Norway