Beanpole Corner Landslide Stabilisation

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1. INTRODUCTION

The Pukerua to Paekakariki Bay section of the North Island, New Zealand main trunk railway line and State Highway One (Figure 1) are located at the foot of a steep 45^{o} coastal rock slope with a 35^{o} head slope. The rock slope was formed by wave action during the last interglacial period and comprises weathered and closely jointed greywacke and argillite which rises 250 to 300m above sea level. The 35^{o} head slope comprises colluvium derived from greywacke. Extensive old scree deposits have accumulated at the toe of the slope and several localised areas of slope instability have caused problems in the past.

The Beanpole Corner slide is a shallow rockslide located on a broad spur above the railway line. The landslide has a long history of instability which started before 1940 as a small slide just above the railway line and progressed 65 m up the rock slope in a series of failures, one of which derailed a freight train in the 1960s. During 1979, a portion of the slide is believed to have moved up to 5 m during or following a period of heavy rainfall and continued creep movement was observed. This portion was located on the head slope immediately above the steeper rockface, 65 m above the railway. The positions of these features are shown in Figure 1.

Preliminary slope assessment concluded that without stabilisation there was an unacceptably high risk of complete and rapid failure closing the railway below. This paper describes the investigation, monitoring, stability assessment, design and construction of engineering works to stabilize the slide. Design and construction supervision of the remedial works was carried out by Works Consultancy Services Ltd for New Zealand Rail Ltd.

2. SCOPE OF INVESTIGATIONS AND INSTRUMENTATION

An engineering geological assessment of the slide was carried out in 1981 by New Zealand Geological Survey, (DSIR), based on surface mapping of the geology and a review of previous instability problems. Site investigation and monitoring was recommended to enable remedial works to be designed.

The following techniques were used to investigate and instrument the slide mass.

(a) A topographical survey.

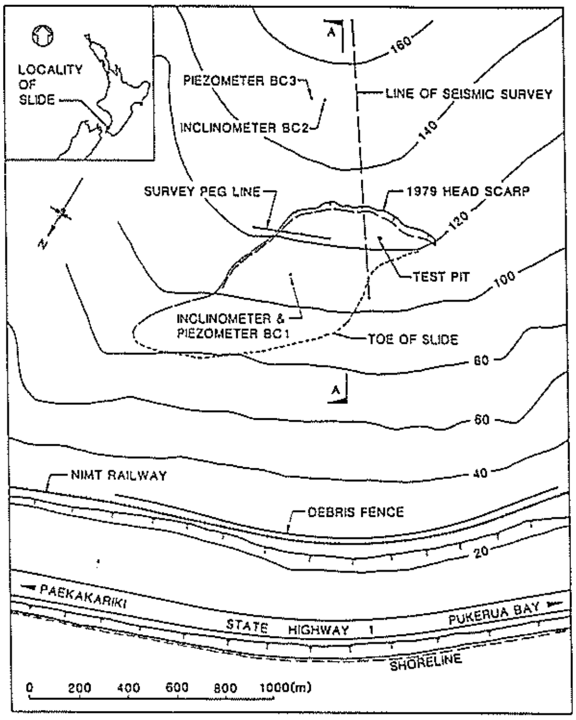

Figure 1 Plan of Landslide Area.

(b) A seismic profiling survey.

(c) One cored borehole to 16 metres depth (BC 1).

(d) A test pit to 9 m depth within the slide mass.

(e) A line of survey pegs across the head scarp area.

(f) Two inclinometers (BC 1 and BC 2).

(g) Two pneumatic piezometers (BC 1 and BC 3).

(h) Installation of a rain gauge.

The position of these investigations and instruments is shown on Figure 1.

3. SITE OBSERVATIONS

The geological mapping, test pit and cored bore hole indicated that the sliding mass was a greywacke derived colluvium comprising loose to moderately dense, angular gravel in a silty matrix with occasional clay lenses. A 4.5 m thick, silty clay layer with greywacke gravel sized inclusions was located at the base of the slide mass. Total slide thickness was approximately 10 m. Bedrock below the slide comprised closely jointed, argillaceous greywacke with irregular beds 200-500 mm thick, dipping 45° to 65° into the slope.

The location and geometry of the active failure surface was determined from the results of both investigations and monitoring, with inclinometer results providing the most valuable data. The failure plane was believed to lie within the thick silty clay layer. Figure 2 illustrates the section geometry adopted for slide analysis and remedial works design. Total slide volume was estimated at 30,000 m^3.

Accurate determination of piezometric levels was not achieved from the pneumatic piezometers although water was observed seeping into the test pit at 5 m depth. Correlation of slide movement with rainfall indicated that increased movement occurred when the three month average rainfall exceeded 100 mm, which suggested a link between slide stability and piezometric pressure. Figure 3 illustrates the correlation developed.

above the slip scarp was not moving. However, materials above the scarp were similar to the actively sliding materials and there was the potential for progressive slope failure.

4. HAZARD AND RISK ASSESSMENT

Continued creep movements of the slide, coupled with the observed acceleration of movement following wet weather, indicated that there was an unacceptably high risk of a rapid failure blocking the railway. This could have led to disruption of commuter and freight traffic and possible loss of life.

A 4 m high debris fence had been installed just above the track through the slide area in the 1960s and had proved effective in preventing boulders and small rock falls from reaching the track. After the 1979 failure, an alarm was installed on the fence to trigger signal lights should the fence be deformed by falling material.

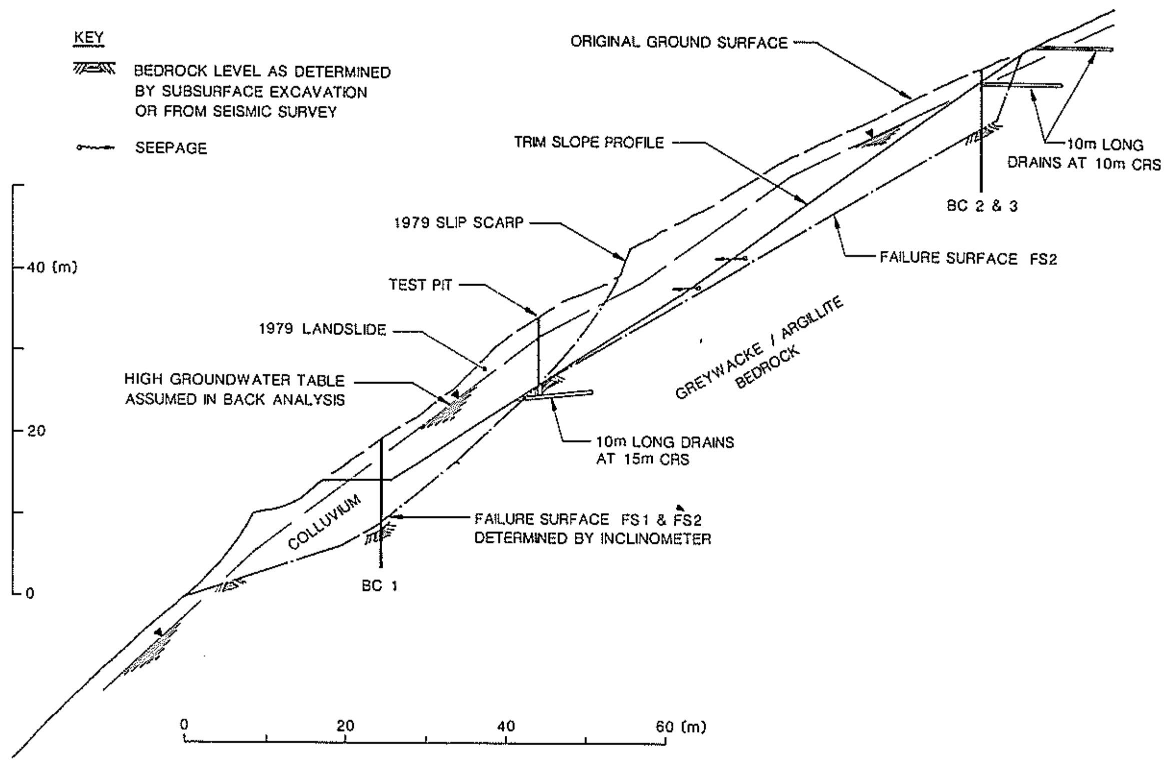

Figure 2 Cross Section A-A through Landslide.

The movement history prior to 1982 is uncertain although there is anecdotal evidence of a slide displacement of up to 5 m following heavy rain in 1979. Monitoring of slide movement using survey pegs began in mid 1982 and Figure 3 illustrates the history of movement up to 1985, and the relationship between rainfall and slide velocity. Movement continued at an average rate of 70 mm/year from 1985 to 1989. Inclinometer BC1 in the active slide was sheared off in December 1983 after 70 mm of deformation in 11 months. Inclinometer BC2 confirmed that the slope

5. STABILITY ASSESSMENTS

Observations of slide movement and the relationship between slide movement and rainfall were used to establish that the slide was at limiting equilibrium prior to stabilization works. No clear correlation was established between piezometric level and slide movement.

The history of slide movement and the consequences of failure were considered sufficient reason to establish the requirement for stabilization works. Remedial works

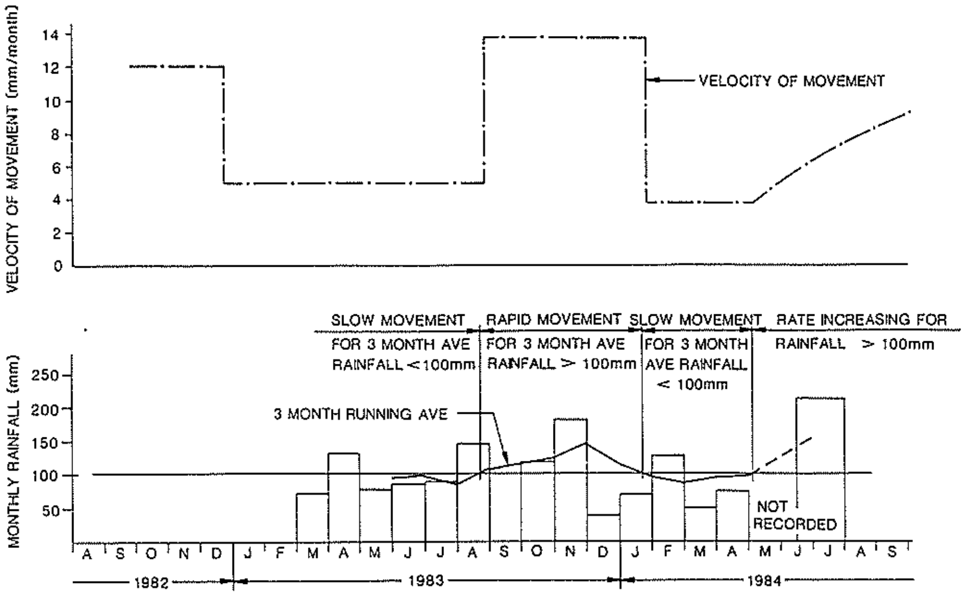

Figure 3 Graphs of Slide Velocity and Rainfall.

including slope trimming and drainage to control groundwater rise during prolonged wet periods were considered.

A geotechnical model of the slide was established which comprised ground topography based on available survey data, the failure surface geometry derived from subsurface geological observations and surface mapping together with material strengths derived from back analyses of the slide. The model is illustrated in Figure 2.

Back analysis assuming limiting equilibrium conditions and two different piezometric cases (dry and wet slopes) were used to determine material shear strength parameters for remedial work design because materials in the slide base were variable and it was not possible to establish shear strength with certainty. The piezometric line assumed for the wet slope case is shown in Figure 2. These analyses yielded the following effective shear strength parameters:

Dry Slope c' = 0 kPa $\phi' = 31^{o}$

Wet Slope c' = 27 kPa $\phi' = 35^{o}$

The "wet slope" parameters were considered more representative of the materials observed and high ground water conditions inferred during the movement event. Therefore the wet slope strength parameters were adopted for final design. Limit equilibrium analysis of the geotechnical model was carried out using the vertical slice Sarma method (1). The stability analysis results for the following three cases are presented in Table 1:

(a) The Original Situation; which models the original level as shown in Figure 2.

(b) Post Construction - Wet Slope; which models the design slope trim as shown in Figure 2 with a groundwater level coincident with the design trim profile.

(c) Post construction - Drained Slope; which models the slope trim with fully effective drainage (an ideal but impractical situation).

TABLE I

STABILITY ANALYSIS RESULTS

Failure Surface	Original Situation	Post Construction Wet Slope	Post Construction Drained Slope
Active Portion (SF1)	1.0	1.3	2.5
Headslope (FS2)	1.0	1.6	2.2

Drainage is generally more cost effective than earthworks for improving slide stability. However, because of the uncertainties in the specific relationship between slide movement and groundwater pressure, uncertainty in level of control achievable with drainage, and the possibility of progressive upslope development of the slide, slope trimming was adopted as the primary means of stabilizing the slide. The extent of slope trimming was determined from analysis and is illustrated in Figure 2.

Analysis indicated slope trimming gave an acceptable stability increase for the two modelled surfaces (FS1 and FS2). However nominal drainage was considered desirable to stabilise the head slope (FS2) because of the difficulty in predicting future mechanisms for progressive development of the slide and because access to the site would be difficult after remedial work.

6. REMEDIAL WORK

Access to the site was constructed from farm land above the landslide and only tracked vehicles were able to negotiate the steep gradients along the access track.

Slope trimming involved a 60 m high cut at a grade of 1.4H to 1V and removal of 31,000 cubic metres of material. Earthworks were carried out between January and September 1990. Material was cut from the slope and pushed into and down a side gully to the railway line where it was loaded out by trucks to a dump site. Excavation was halted while trains passed beneath the site because of the steep slopes and potential for boulders loosened by construction to roll downslope and bounce over the debris fence onto the railway.

Three rows of 10 m long horizontal drains at 10 m centres were planned from benches formed in the head of the cut slope to control ground water in the slope above the active portion. The lower two rows of five drain holes were dry and the highest row was not installed. A seepage area was observed during excavation at about mid slope and a bench was formed just below the seepage to channel increased flow during wet weather into the side gully. The seepage continued beneath the bench and a row of 10 m long horizontal drains at 15 m centres was installed from a bench formed at RL 110 m. A slight flow (less than 1 li/m) was encountered from only one of these drains.

7. POST CONSTRUCTION PERFORMANCE

Geological conditions encountered during construction were similar to conditions predicted from investigation results. The previous monitoring instrumentation was removed during earthworks and surface survey marks have been installed since. Regular repeat surveys of these marks is planned with a re-survey after the first winter.

8. SUMMARY AND CONCLUSIONS

The Beanpole Corner slide presented a potential hazard to the operation of a busy section of the main trunk railway line. The slide was investigated and monitored, remedial works were designed and constructed. Principal conclusions from the work follow:

a) A possible relationship between slide movement and rainfall was identified but with the limited instrumentation it was not possible to establish a relationship between groundwater pressure and movement.

b) The design standard for the active part of the slide and the slope above was based on an assessment of geological and groundwater uncertainties, the hazard and the difficult site access after remedial work was completed.

c) Slope stabilisation measures were able to be designed accounting for uncertainties of material strengths and piezometric pressures.

d) Drainage was installed but no appreciable flows from these holes were observed. Drainage holes may act as a control on future groundwater rise during prolonged wet periods.

Post construction monitoring has been installed but at the time of paper preparation results of resurvey were not available.

9. ACKNOWLEDGEMENTS

The authors wish to thank New Zealand Rail Ltd and Works Consultancy Services Ltd for their permission to publish this paper. Acknowledgement is made to the many members of Works Consultancy Services Ltd (formerly the Ministry of Works and Development) and DSIR who worked on the investigation, design and construction of this project.

10. REFERENCES

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