WAIKAREMOANA POWER SCHEME - PIRIPAUA PENSTOCK ENGINEERING GEOLOGY STUDY AND RISK ASSESSMENT

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SUMMARY

ECNZ wished to have an engineering geology and risk assessment study undertaken of the Piripaua penstock slope, part of the Waikaremoana Power Scheme. This paper presents the method used for geological and statistical analysis and the conclusions made. Aerial photograph interpretation as well as field mapping of the geology was carried out to determine the likely failure mechanisms. Three dimensional analysis of wedge failures was performed with variation of input parameters such as strength and earthquake magnitude to allow probabilities of failure to be evaluated.

INTRODUCTION

Piripaua Power Station is the third in a line of three stations on the Waikaretaheke River, draining Lake Waikaremoana. The lake is 610 metres above sea level and the valley below it falls over 448 metres within 8 km, making it ideal for hydro-electric purposes.

The scheme uses Waikaremoana's water by firstly carrying it through a tunnel then two pipelines to Kaitawa power station into Lake Kaitawa. The water then passes through another tunnel to a surge chamber, and down penstocks to Tuai station and is discharged into Lake Wakamarino. A further tunnel leads to a surge chamber and then through penstocks to Piripaua station. The water finally discharges into the Waikaretaheke River.

The combined power output of the three stations is 132 MW.

STUDY REQUIREMENTS

ECNZ Tokaanu Hydro Group wished to have a study undertaken of the Piripaua penstock area. The brief was to undertake and complete a study of the engineering geology of the Piripaua surge chamber, penstocks and powerhouse area to investigate slope stability hazards and assess possible failure scenarios and their impacts upon structures. The study was to consider the risk probabilities of the possible failure scenarios and to identify engineering options that would mitigate the impacts of the failures.

ENGINEERING GEOLOGY

Regional Geology

The regional geology of the area is well understood considering the studies that have been undertaken on the massive landslide that formed Lake Waikaremoana approximately 2000 years ago, and the construction activities which have taken place with the Waikaremoana Power Scheme. The Huiarau and Ikawhenua Ranges, west and northwest of the lake respectively are formed from axial-range greywacke which is part of the mountain chain that extends from eastern Bay of Plenty, southeast to Cook Strait. The lake is surrounded by younger sedimentary rocks of Tertiary age.

The rock in the immediate area of Piripaua is Tertiary aged sedimentary consisting of interbedded sandstone, siltstone and mudstone. The sedimentary sequence has been uplifted and tilted gently at dips up to 20° to the southeast.

Aerial Photograph Interpretation

Vertical aerial photographs from 1952 and 1965 were available from NZ Aerial Mapping for the area. In addition, a set of nine aerial photographs were borrowed from ECNZ in Tokaanu, taken during construction of the Piripaua power scheme (1942).

Aerial photograph interpretation suggested that two historic landslips were present in the immediate area of the Piripaua penstocks and powerhouse, as shown on Figure 1. The construction photographs showed that one of these slips was linked to construction of the tunnel leading to the surge tank, as the photo shows that the tunnel spoil was dumped on the north facing side of the ridge leading to the west from the surge chamber. Some of this spoil appears to be pushed over the ridge line and has slid down the slope towards the powerhouse.

Field Mapping

Detailed field mapping of the area surrounding the penstock slope and powerhouse was carried out. Two days were spent walking over the penstock slope and surrounding countryside to develop a localised geological model. Rock exposures were logged in the hills behind the penstocks, in the Piripaua Stream bed and cuttings for SH38. A large amphitheatre was also inspected from the left bank access road leading from Piripaua to Tuai.

Rock Mass Description

At the surge tank the rock mass consists of dark grey soft weak slightly to moderately weathered thinly interbedded mudstone and siltstone with minor interbedded sandstone. This lithology extends downstream on a farm track at or above the surge tank level. Bedding, dipping 22°, N110° is the most well developed defect in this lithotype with defect sets orthogonal to bedding prominent. Upstream of the surge tank unweathered light grey to grey fine, weakly commented, moderately strong sandstone appears to be the dominant lithology outcropping on the surge tank access road and on SH38 below the old hydro village. The more erosion resistant sandstone may form the prominent ridge to the north of the site.

Defect surveys were carried out at the surge tank and at nearby road exposures. The defect orientations are summarised on Table 1 and Figure 2. The main defect set is bedding (set F) with the orthogonal defect sets C, D and E.

A slickensided bedding plane shear surface was observed on the amphitheatre to the northwest of Piripaua. The defect was filled with 5 to 10 mm of gouge and indicative of movement in the past. The amphitheatre may have been formed by a wedge failing on the bedding plane shear surface.

Table 1. Rock mass strength estimation

SET	DIP	DIRECTION	CONTINUITY	WAVINESS	SEPARATION	SPACING
A	70°	350°	2+ m	planar	<0.1 mm	0.3-1.0 m
B	70°	100°	5+ m	planar	<0.1 mm	1.0-2.0 m
C	70°	270°	2+ m	planar	<0.1 mm	0.3-1.0 m
D	70°	240°	2+ m	planar	<0.1 mm	0.3-1.0 m
E	75°	210°	2+ m	planar	<0.1 mm	0.3-1.0 m
F	75°	310°	20+ m	planar	<0.1 mm	0.05-0.2 m

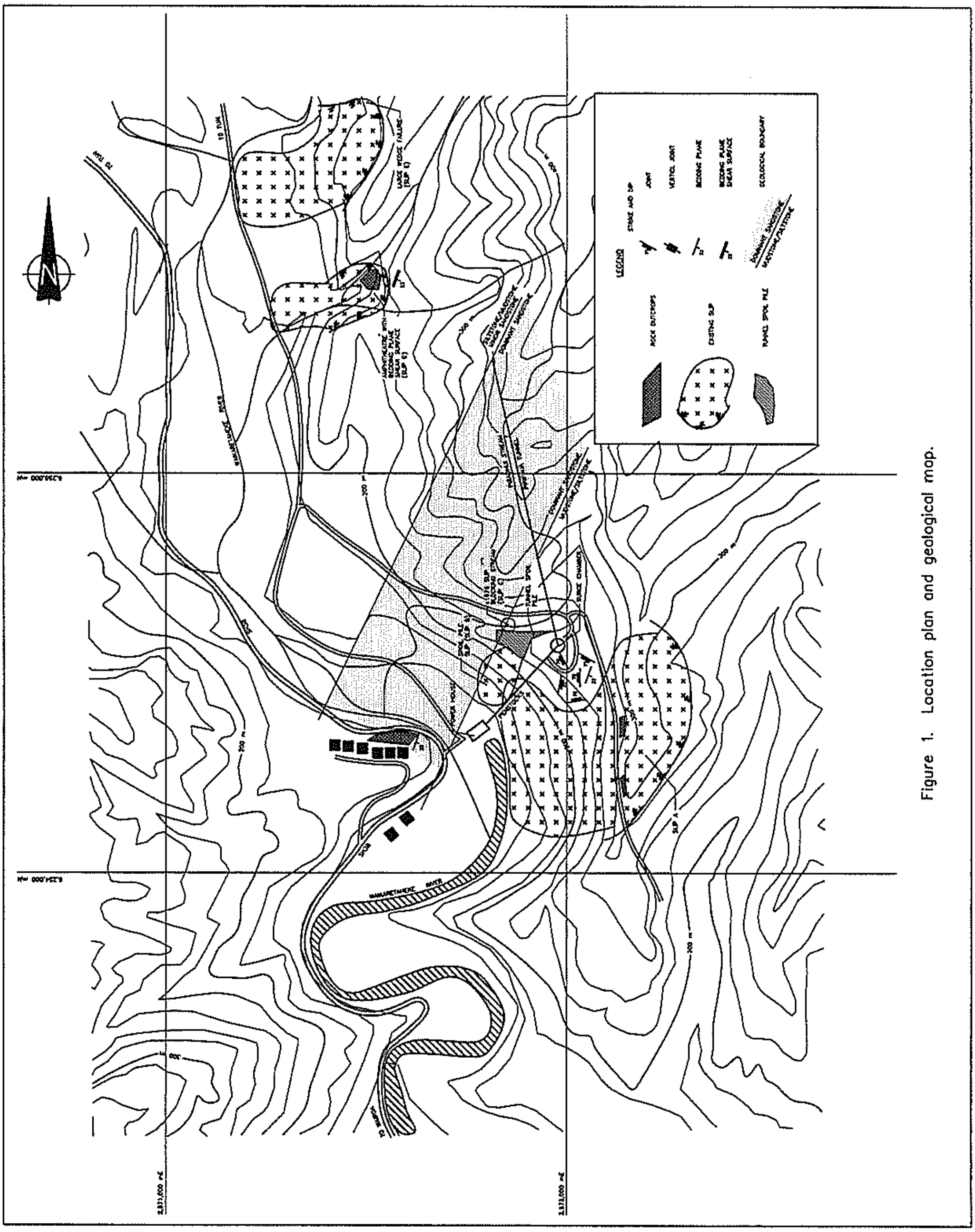

Figure 1. Location plan and geological map.

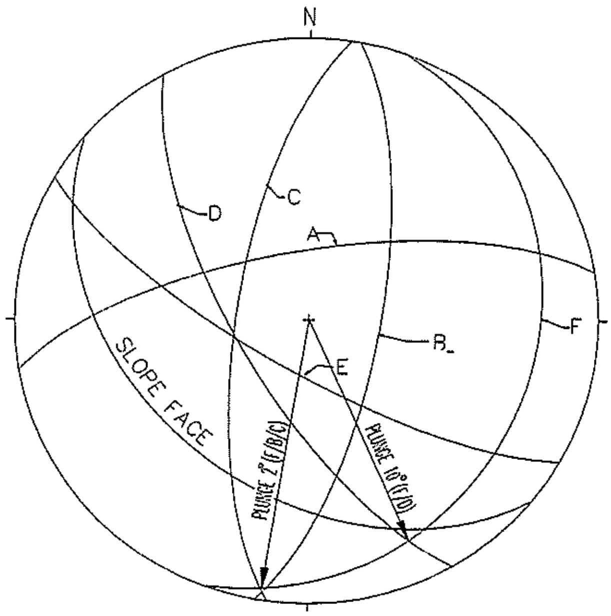

Figure 2. Design stereoplot data.

Rock Defect Strength Parameters

For use in the risk analysis, the 5th and 95th percentile limits for a non-linear Mohr Coulomb envelope were estimated, based on the method presented by E Hoek et al [1]. Table 2 presents the values selected using Hoek's method.

Table 2. Rock mass strength estimation.

FACTOR	5% VALUE	5% SCORE	95% VALUE	95% SCORE
Intact Rock Material Strength	3 MPa	1	10 MPa	2
Drill Core RQD	50%	8	75%	17
Joint Spacing	0.3 m	15	1 m	25
Joint Condition		12		25
Ground Water		7		7
TOTAL		43		76
GSI		43		76
CONDITION		VERY BLOCKY		BLOCKY
m_b/m_i		0.16		0.40
S		0.003		0.062
a		0.50		0.50
Em		9000		40 000
ϑ		0.25		0.20
GSI		48		75
m_i		6		19

The shear strength versus effective normal stress curve was plotted for the 5th and 95th percentiles, and power curves fitted to them. The mean and standard deviations of c' and ϕ' were developed and are presented in equations 1 to 4 below.

$$\varnothing'_{MEAN} = \frac{ATAN(8.480.\sigma_n^{-0.2821}) + ATAN(3.789.\sigma_n^{-0.3220})}{2} \qquad ...1$$

$$\varnothing'_{STD} = \frac{ATAN(8.480.\sigma_n^{-0.2821}) - ATAN(3.789.\sigma_n^{-0.3220})}{4} \qquad ....2$$

$$c'_{MEAN} = \frac{3.332.\sigma_n^{0.7179} + 1.799.\sigma_n^{0.6780}}{2} \qquad ....3$$

$$c'_{STD} = \frac{3.332.\sigma_n^{0.7179} - 1.799.\sigma_n^{0.6780}}{4} \qquad ....4$$

The above curves are presented in Figure 3.

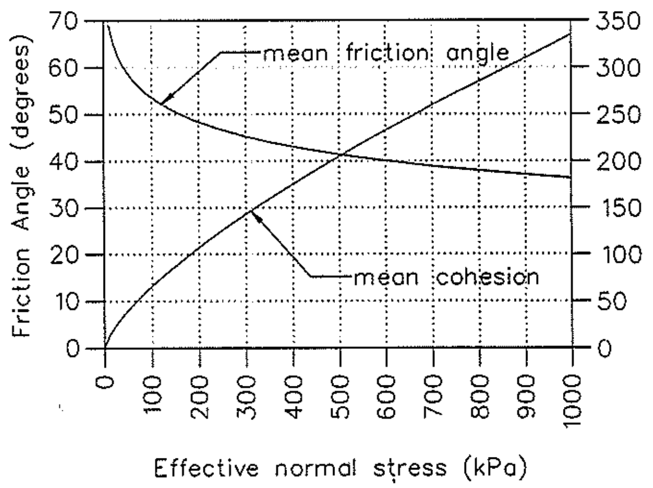

Figure 3. Mean Strength Parameters.

Likely Failure Mechanisms

Based on interpretation of the defect data, kinematically the most likely failure mechanism involves wedges formed by defect sets: F & C, F & B, F & D and C & D with rear release joints A or E. Large wedge failures have the most significant consequence of failure in that they would be likely to cause considerable damage to the penstocks and possibly to the power house, surge chamber and feed tunnel. Evidence of large wedge failures are reasonably common in the area, with the landslide which formed Lake Waikaremoana being an example.

Large wedge failures are likely to be formed by bedding plane defects in combination with two or more other joint sets. Ground water pressures reduce the frictional component of strength. The large wedges are likely to be stable under static conditions, aided by discontinuous joints, and require a trigger such as a very large seismic event to cause failure. The energy involved in such an event would cause massive damage to the affected area.

RISK ASSESSMENT

Methodology

Various failure mechanisms were investigated and the critical mechanism was established as a three dimensional wedge failure. To allow evaluation of the wedge failure mode a spreadsheet was developed to calculate the factor of safety of a wedge based on the method developed by Hoek & Bray [2] using various plane geometries, material parameters and groundwater conditions. All of the combinations of defect orientations were evaluated to find the critical combination.

Risk modelling was carried out on the critical failure geometry using the risk analysis and modelling software "@RISK" in conjunction with the "LOTUS 123" spreadsheet. The upper and lower non-linear Mohr Coulomb envelopes previously developed were adopted as the 5th and 95th percentile values of a normal probability distribution. The mean value (relative to effective normal stress) for cohesion and friction angle was taken as the median of the upper and lower strength envelope, while the standard deviation was taken as one quarter of the range of upper and lower values. A power function was fitted to the resulting curves and non-linearity with respect to the normal effective stress was incorporated into the spreadsheet.

The second parameter with a possible variation was the horizontal seismic acceleration. A cumulative distribution function was evaluated based on the probability of a certain level of earthquake shaking occurring within one year. The probability distribution was programmed into the spreadsheet. The resulting model allowed normal variation of cohesion and friction angle, with the mean and standard deviation dependent upon the calculated effective normal stress, and variation of earthquake magnitude. The program "@RISK" was then used to run 1000 simulations of the model, randomly choosing variable values from their appropriate probability distribution. The factor of safety was calculated for each simulation and recorded for statistical analysis. Using this method, the probability of failure for various size wedges could be evaluated.

Results of Risk Analysis

The critical failure mode was found to be a three dimensional wedge formed by the defect sets F and C with a tension crack formed by defect set A. Failures essentially wholly within the 40° penstock slope were found to be most likely. The height and hence dimensions of the wedge were varied with mean factor of safety calculated under static conditions presented in Table 3 below.

Table 3. Mean static factor of safety for critical wedge geometry.

Wedge Height (m)	Factor of Safety
5.0	8.07
10.0	6.60
15.0	5.88
20.0	5.42
30.0	4.84
40.0	4.46
50.0	4.20

As shown in Table 3 wedge failures become more critical with depth. Failures were limited to 50 m height by the height of the penstock slope.

Using "@RISK" in conjunction with "LOTUS 123" a full probability analysis was carried out allowing for variability in cohesion, friction and PGA. The results of this analysis are summarised in Table 4.

Table 4. Seismic probabilistic analysis of 3D wedge stability.

Wedge Height (m)	Probability of Failure
5.0	0.17
10.0	0.22
15.0	0.20
20.0	0.40
30.0	0.40
40.0	0.30
50.0	0.47

To further highlight the effect seismic acceleration has on the probability of failure (failure is defined as a factor of safety less than 1.0) a relationship between PGA and probability of failure was developed. The results are presented in Table 5 and plotted in Figure 4 for large volume slips (i.e. full wedge height).

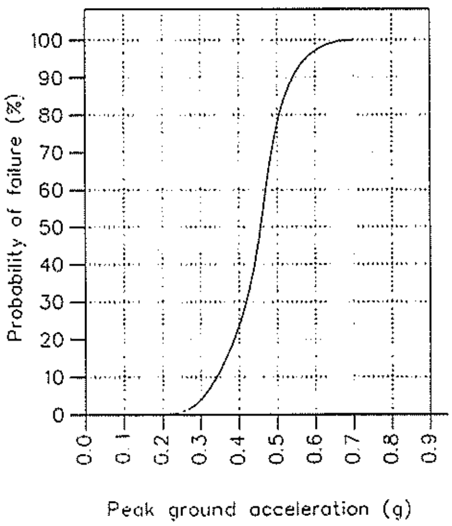

Figure 4 Effect of seismic acceleration.

Table 5. Relationship between PGA and probability of failure for a large volume slip.

PGA (g)	P(FOS<1.0) (%)
0.0	0.0
0.1	0.0
0.2	0.0
0.3	0.6
0.4	21.4
0.45	41.5
0.5	89.6
0.6	99.6
0.7	100.0

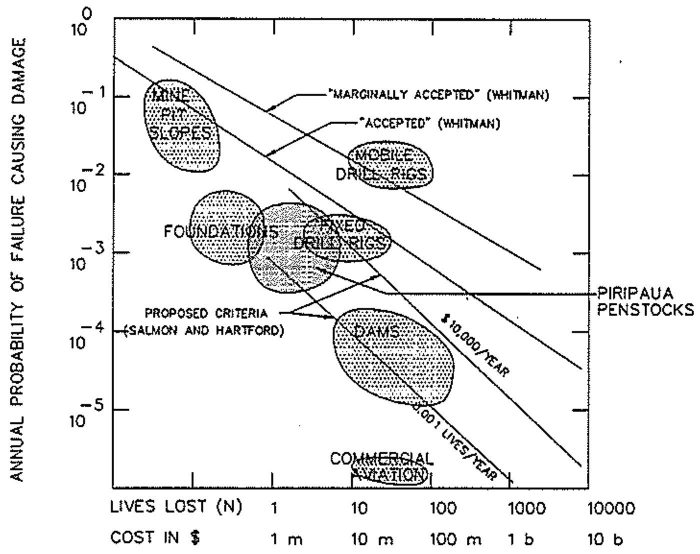

Figure 5. Risks for selected engineering projects.
(Whitman, 1984 and Salmon and Hartford, 1995)

Comparison With Acceptable Risks and Mitigation Measures

The risk of a large wedge failure with current assumed ground water conditions (full saturation) is approximately 0.5% for any one year. Figure 5 shows some examples of normally acceptable risks for selected engineering projects as suggested by Salmon and Hartford. As shown by the area shaded for the Piripaua project, the present risk is generally within the lower range of commonly accepted risk criteria.

The critical failure analysed is a large wedge failure that is limited by the height of the slope. Due to the volume of rock involved conventional methods of reinforcement such as rock bolting or anchoring are not feasible. The only feasible option for reduction in the degree of risk is drainage of the penstock slope.

The degree of risk was calculated on the basis of fully saturated conditions, which is consistent with field observations. If the level of risk was considered to be unacceptably high, the first course of action would be to verify the groundwater regime by monitoring groundwater levels in the penstock slope. Analyses have been carried out for a range of groundwater conditions expressed as r_u (r_u is the ratio of water pressure head at the slip surface to the weight of soil/rock above: r_u of 0.42 is full saturation). The results are summarised in Table 6 and show that a significant improvement in risk level can be achieved by small reductions in r_u.

Table 6. Variation in risk with groundwater conditions.

r_u	Probability of Failure (%)
0.0	<0.005
0.04	<0.005
0.08	<0.005
0.13	<0.005
0.17	0.02
0.21	0.05
0.25	0.08
0.29	0.14
0.33	0.25
0.38	0.30
0.42	0.47 (present condition)
0.63	2.86 (confined aquifer)

If groundwater levels proved to be high after monitoring, two main options would be available for drainage of the slope. As a relatively inexpensive option, horizontal bored drains could be installed in the slope. A drainage adit would be more effective but would involve considerably more cost.

CONCLUSIONS

The main conclusions from the study were:

- The toe of a landslip on the south eastern side of the penstocks has been eroded by the Waikaretaheke River (Slip A). The slip was likely to have been caused through undercutting and over-steepening of the rock mass by the Waikaretaheke River resulting in a defect controlled release surface. The slope now appears to be in equilibrium with slip debris at the toe of the slope and providing a buttress which will reduce the likelihood of further failures.

- The major source of risk to the scheme consists of large wedge failures triggered by seismic events. With the current assumed high groundwater conditions, there is an approximate risk of failure within any one year of 0.5%. This is within the lower range of commonly accepted risk/cost relationships. If the risk was considered to be unacceptable the groundwater assumptions should be verified by installation of piezometers. If saturation is proved by monitoring, significant improvements in risk level could be achieved by slope drainage.

ACKNOWLEDGEMENTS

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REFERENCES

1. Hoek, E 1994. Strength of Rock and Rock Masses, *International Society for Rock Mechanics News Journal*: 4-16.

2. Hoek, E and Bray, J W 1977. *Rock Slope Engineering*. The Institution of Mining and Metallurgy.

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