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# Canal Failure, Ruahihi Hydro Electric Power Scheme, Bay of Plenty, New Zealand

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SUMMARY The Ruahihi power station canal failed on 20 September 1981, one day after being officially opened. Water losses from the canal, and a number of slope failures resulting from these, were observed soon after the canal was filled for the first time.

Progressive slope failures on the embankment and slope immediately upstream of the forebay led eventually to a massive slope failure during which a chasm 500 m long, 100 m wide and 40 m deep was gouged out in about one hour. There was no loss of life and no-one was injured.

The cause of the failure has been attributed to the behaviour of the volcanic soils when wetted and loaded by the canal water. The soils, tephra and ignimbrite, are underconsolidated, low-density and non-plastic materials. They are brittle, both in situ and when compacted, highly erodible and possibly dispersive. Cracks and openings that develop in them are non-healing. When wetted and loaded they tend to undergo a structural change that can lead to a collapse of the soil fabric.

Early investigations indicated the nature of the materials, but the significance of these was apparently not recognised in the planning, detailed investigation and design stages.

The scheme was recommissioned, and was generating power again in June 1983.

## 1 INTRODUCTION

A section of the head race canal supplying water to the Ruahihi Power Station failed early in the afternoon of Sunday 20 September 1981, one day after the station had been opened officially, by the Prime Minister.

Water in the 3350 m canal, in a matter of about an hour, had scoured a chasm, 500 m long, 100 m wide and 40 m deep in the local volcanic soils and fill.

No lives were lost and no-one was injured, but much damage was done to farmland, local roads, a highway and power lines.

The cause of the failure cannot be determined unequivocally, but the evidence available strongly supports the view that after canal filling, ca 7 months before the failure, canal water entered the in situ materials through the placed lining in sufficient quantities to cause internal erosion and soil collapse. A build up of pore water pressure behind a placed fill, which had a lower permeability than the natural deposits, preceded the eventual canal failure. Prior to the canal failure there was convincing evidence of water seepage and minor slope failures in this and other fills along the canal's length.

The failure was investigated by a Committee of Inquiry appointed by the Commissioner of Works, Ministry of Works and Development. The committee's report was presented on 26 February 1982. This paper draws very heavily from this report, and from one prepared for the committee by the New Zealand Geological Survey.

## 2 BACKGROUND

The 20 MW Ruahihi Power Station was the last of four stations to be constructed in the Tauranga joint Generation Committee's (TJGC) Wairoa River Hydro-electric Development, a development programme that extended over a number of decades. All of the various components of the development were designed by private consultants, and built by private contractors.

The TJGC sells power to the local city council and power board. Its system is connected to the national system controlled by Electricorp (previously the Electricity Division of the Ministry of Energy). At times it sells power to the national system, at other times it buys power from it.

## 3 OUTLINE OF SITE GEOLOGY

To utilise the ca 85 m head from Lake McLaren to the tidal reaches of the Wairoa River ca 3.5 km distant (see Figure 1) it was necessary to traverse volcanic terrain built up as successive airfall and pyroclastic flow deposits by episodic volcanic activity from numerous centres over the past two million years.

Rivers and streams draining the Kaimai Range to the west have cut deep (locally ca 40 m) steep-sided valleys in the largely easily eroded deposits. More resistant deposits in places have impeded erosion and produced small waterfalls. Many of the valley sides are potentially unstable, as springs and seepages near their bases have induced local slumping, and produced oversteepened slopes. The interfluves between the valleys are in places only a few hundreds of metres wide.

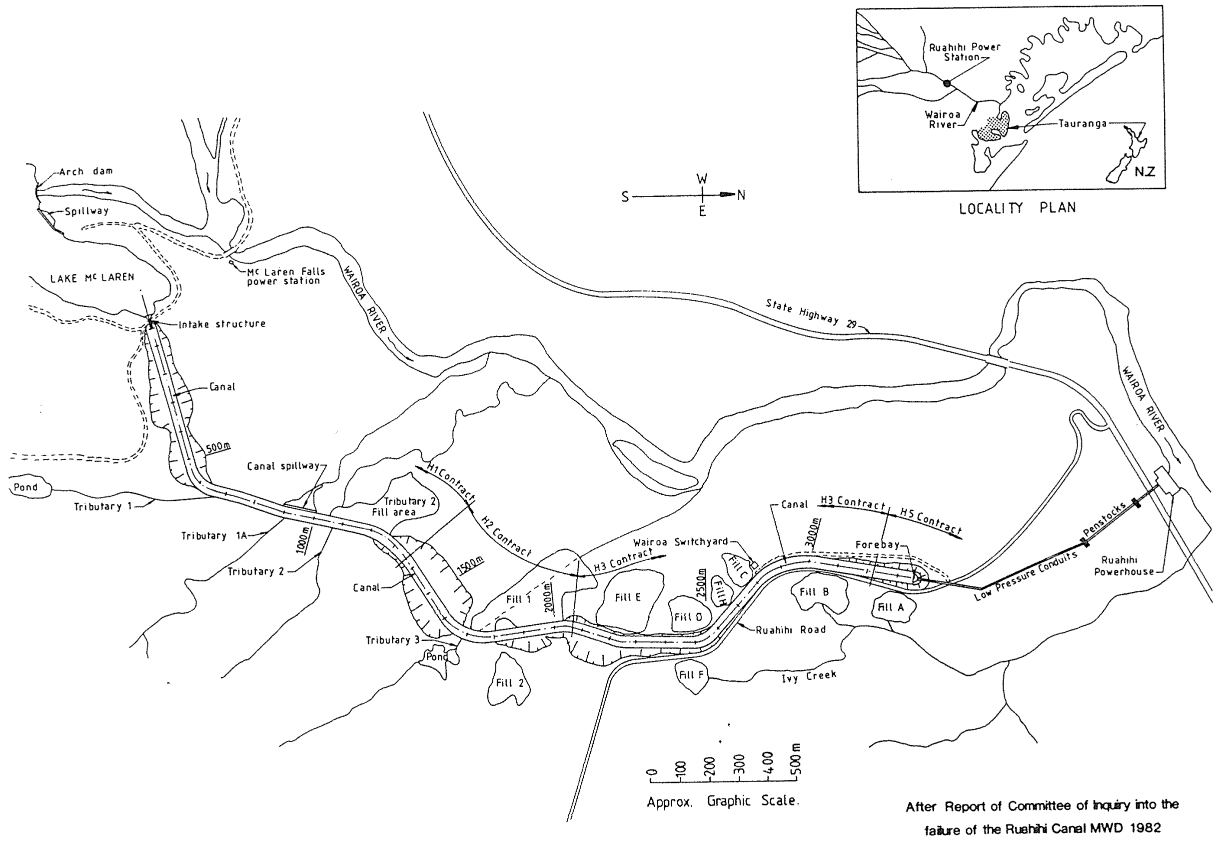


FIGURE 1 MAP OF RUAHIHI HYDRO-ELECTRIC SCHEME

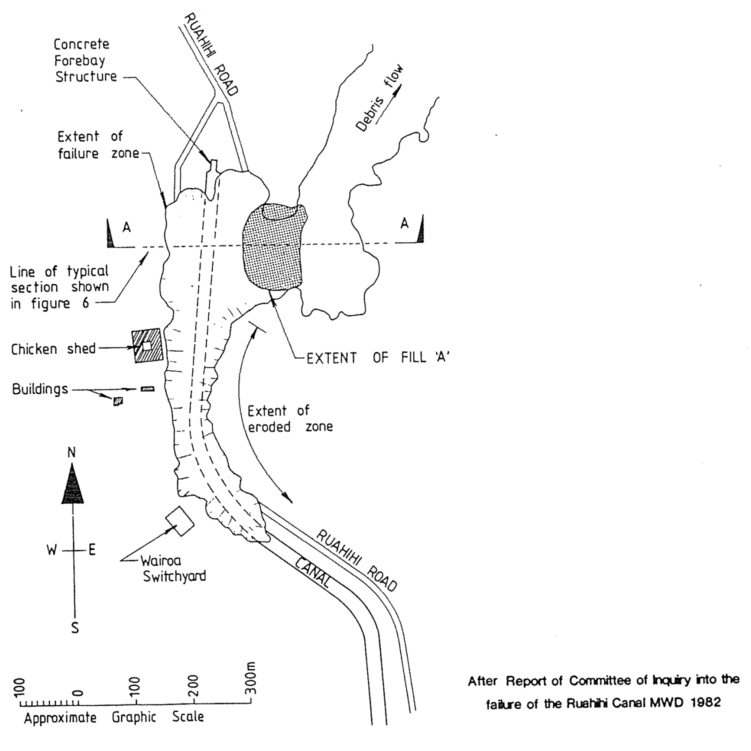


FIGURE 2 PLAN OF FAILURE AREA

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The section of the canal that failed (see Figure 2) was near the eastern margin of an interfluve, both sides of which were formed, and continually modified by natural valley side and head erosion.

The sequence of deposits in the area, which is composed entirely of airbourne volcanic materials, is summarised in Table 1 (see also Figure 3).

Apart from the upper 4 to 5 m of tephra (volcanic ash, see Appendix 1) the deposits near the failed section of the canal are Mamaku Ignimbrite (in this area essentially unwelded, in places weathered, loose and uncemented pumice breccia - see Appendix 1). The subsurface configurations of the ignimbrite are highly irregular because of its mode of deposition, and of Post-depositional erosion. Apart from these features, there are few mappable geological structures. Topographic lineaments can be identified in a few places on aerial photographs, and these may be related to past faulting, but this has not been demonstrated. None of these passes through the failed area. Jointing has been identified in the Mamaku Ignimbrite elsewhere in the region, but none was observed in the chasm cut by the failure.

The mineral composition, fabric and physical properties of these various deposits, and the ground water regime are relevant, and probably had a significant bearing on the failure. All deposits exposed in the chasm, except the locally - derived intraformational bedded sands, are airfall or pyroclastic flow deposits.

### 3.1 Composition

The clay content of the ignimbrite is generally quite low, and of the tephras, only the well-weathered Hamilton Ash has a high clay content. The dominant clay mineral in the ignimbrite is hydrated halloysite. Allophane (see Appendix 1) is abundant in the overlying ashes, although the clays of the Hamilton Ash are mainly halloysite and Kaolinite, and for these reasons this is the most stable tephra.

### 3.2 Fabric

The subaerial mode of deposition of the ignimbrite and ashes has resulted in a loose packing of particles in these deposits, and this combined with low particle density, has produced materials of low bulk densities. Such soils, when unsaturated, are susceptible to large decreases in volume when they become saturated. This soil collapse may be triggered by wetting alone, or by wetting and loading acting together. Judged on the range of water contents, liquid limits and densities given by Parton and Olsen (1980) for "ignimbrite" and brown ash", these soils have high porosity and collapse potential. The greatly accelerated rate of settlement of the forebay structure after canal filling, may well indicate a response to wetting after loading.

### 3.3 Sensitivity

Many of the deposits near the canal have a high ratio of peak undisturbed strength to remoulded strength, that is, they are highly sensitive. Sensitivities of ca 60 were indicated by limited field shear vane tests (Oborn et al 1982). Undisturbed material can be still and brittle, but on remoulding it behaves almost as a fluid, with strengths of 1 to 2 kPa only.

These properties posed problems during investigations and construction. Sand and pumice breccia which appear to be moderately compact in outcrop, can appear to be plastic when still wet in a core barrel. The metastability of these materials is presumably a consequence of their fabric.

### 3.4 Erodibility and Dispersivity

The materials in this area, dominantly low-density silt and sand grade with little clay content, offer little resistance to erosion by flowing water. The erodibility of these materials was demonstrated by the presence of fine grained particles in suspension in water issuing from the numerous 'tunnels' exposed in the failure chasm. the undisturbed pumiceous sandy silts are not self healing, as many other soils of low clay content are. This may be because in the undistrubed state they are brittle. In the remoulded state they have very little, if any, coherence. There is evidence to support the view that some of the soils are also dispersive.

### 3.5 Permeability

The permeability of both the tephra and ignimbrite varies over several orders of magnitude, particularly between the in situ and recompacted states. Most materials are judged to have quite a high permeability in situ, in part because of

| PROVISIONAL NAME: FORMATION NAME | FIELD NAME | STRATIGRAPHIC COLUMN | THICKNESS (m) | ENGINEERING GEOLOGICAL DESCRIPTION | FIELD PROPERTIES AND NOTES |
|---|---|---|---|---|---|
| | Top Soil | | 0.1-0.3 | Black, organic top soil | |
| | Ashes | | 2-2.5 | Yellowish brown, firm, friable silty sand with rare clay | Undifferentiated ashes, younger than c.43,000 years |
| Rotoehu Ash | | | 0.2-1 | Light yellow, firm to loose sands to gravelly sands | Prominent marker bed, mantles pre-existing topography, dated c.43,000 years |
| Hamilton Ash? | Brown Ash | | 0.7-1.5 | Dark brown, firm to stiff massive clay | Low permeability beds locally perch groundwater in Rotoehu Ash |
| Upper Manaku Ignimbrite | Upper Ignimbrite (pumice breccia) | | 7-8 | Brownish-white and pink with distinctive black mottles, clayey SILT or clayey SILTY FINE SAND with pumice clasts up to 150 mm diameter, especially near the base. Variably weathered.<br>Very soft to firm, depending on moisture content and weathering, which vary laterally and vertically. Sensitive, highly plastic soil with apparent low bulk density and very high moisture content. | Sensitive soil with signs of "disturbance" where saturated especially in the canal area. Wet specimens are very soft and liquefy when disturbed. Water can be squeezed from moist hand specimens.<br>Degree of weathering varies laterally and vertically |
| | Base surge deposit | | 0.2 | White to brown GRAVELLY COARSE SAND | Distinctive marker bed |
| | Ignimbrite "or rock" | | 15-2 | Reddish brown, clayey SILTY FINE SAND stiff, moist | Some iron cemented joints, resistant to erosion and serves as a cap rock in places. Thickest in pre-existing topographic depressions |
| | Bedded Sands | | 0-4 | Greenish to reddish brown, bedded (in places distinctly current bedded) unweathered COARSE SANDS. "Loose" although limonite cemented in places, especially near the base, moist and permeable. | Variable thickness, absent in some sections. Stands vertically, low cohesion, easily excavated with a spade. Mantles undulating topography - airfall tephra reworked locally by water (current bedded) and wind. |
| Lower Manaku Ignimbrite | Lower Ignimbrite (pumice breccia) | | 2-15 | Light grey, unweathered, pumiceous COARSE SANDS, with some pumice clasts ranging in size from pebbles to (rarely) 150 mm diameter. Occasional small rock pebbles. Carbonised logs with fossil fumaroles above filled with coarser sands.<br>Faintly bedded in places and iron stained at base<br>"Loose", moist, and permeable. Easily eroded by running water. | Structure destroyed on handling when wet, crumbles when moist. "Loose" when dry. Shows extensive piping, and gullying where exposed to running water. Numerous carbonised logs (150-200 mm diam, 1-2 m long). The impressions left after the carbon has been removed act as drains, and water has flowed freely from these, especially from lower levels.<br>Low cohesion, density varies laterally - from material that requires effort to excavate with a spade, to "loose". Considerable lateral and vertical variations in soil properties. |
| | Ignimbrite | | 0.5-1 | Pinkish-white, CLAYEY SILT. Stiff, moist, sensitive | Water seeps from base of formation. Stiff, sensitive. |
| | Bedded tephra | | 1.5-2 | Light brown interbedded COARSE SAND and CLAYEY SILT. Sands "loose", silts firm; moist and permeable. | Excavated by spade. CLAYEY SILTS may perch water locally. |
| | Ignimbrite | | >2 | Brown (Fe and Mn stained) CLAYEY FINE SAND. Very stiff to hard, mosit. | Material can be chipped out with a spade. |

GENERAL NOTES

1. All materials exposed at the Ruahihi site are described as soils as they are natural aggregates of mineral grains that can be separated by gentle mechanical means.
2. The descriptive term ignimbrite has previously been applied to silicic pyroclastic flow deposits erupted as dense incandescant clouds of volcanic glass and shards to form a hard rock. The usage has now been expanded to include all volcanic materials deposited from pyroclastic flows. and includes a wide range of materials varying from hard rock (welded ignimbrite and lenticulite to non-welded, loose aggregates of blocky and fine pumice (pumice breccia).
3. Soil descriptions are as observed several weeks after canal collapse.
4. Sensitivity is defined as the ratio of undisturbed to remoulded strength.

After N.Z. Geological Survey unpublished report EG 361.

TABLE 1 GENERALISED GEOLOGICAL STRATIGRAPHIC COLUMN

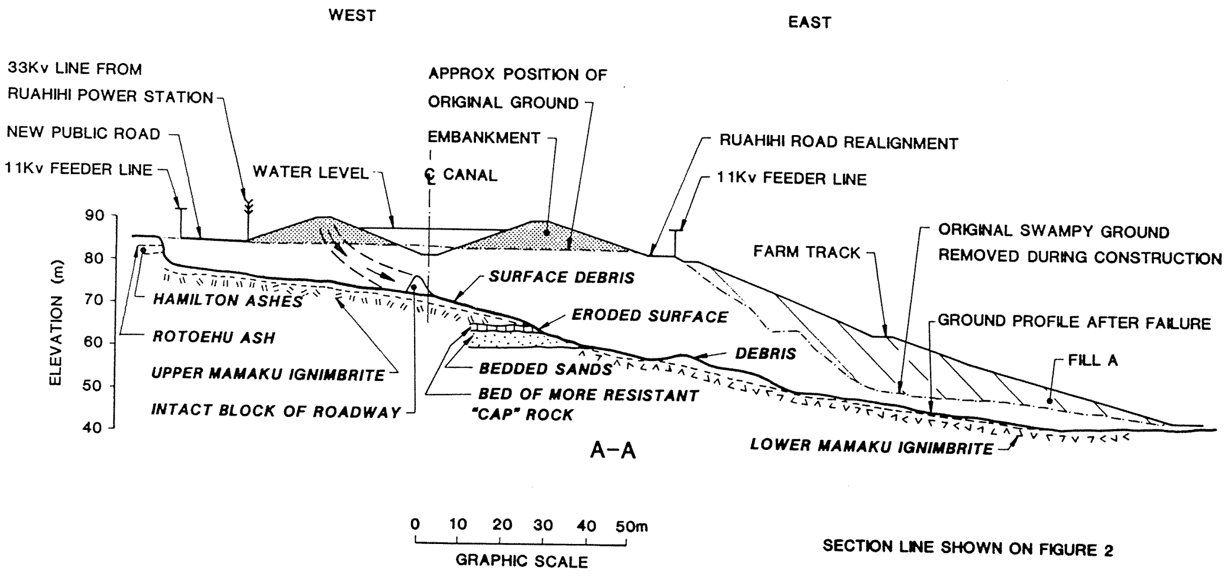
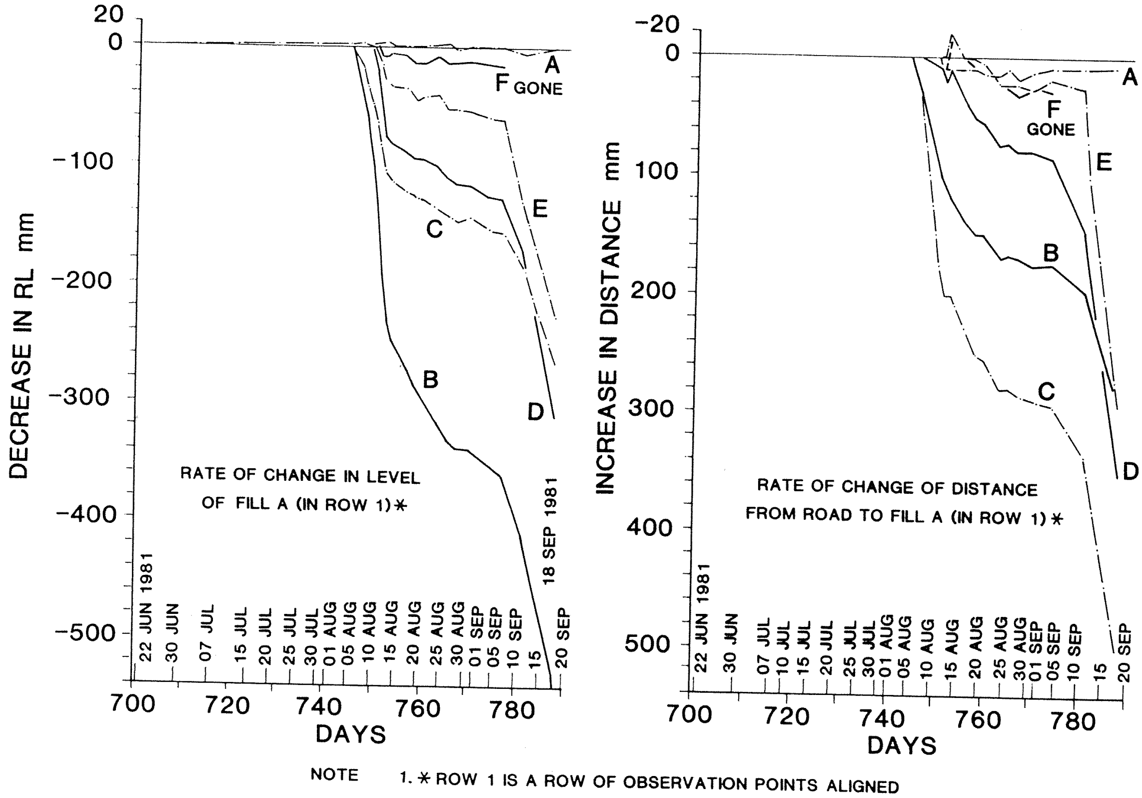



FIGURE 3 PROFILE OF FAILURE AREA SHOWING PRE AND POST FAILURE SURFACES AND GENERALISED GEOLOGY




NOTE 1. * ROW 1 IS A ROW OF OBSERVATION POINTS ALIGNED DOWNSLOPE AT THE NORTHEN END OF FILL A.

2. A,B,C,D,E &F ARE OBSERVATION POINTS ALONG ROW 1.

FIGURE 4 TYPICAL VERTICAL AND HORIZONTAL MOVEMENTS OF FILL A their brittle nature and the maintenance of cracks, fractures and erosion channels in them.

### 3.6 Ground Water

Preconstruction water-table levels in the area that failed, (the only section that is discussed here), were low, possibly 17 to 20 m or more below canal invert level. Shallower water tables may have occurred locally where water was perched on less permeable beds, but these it is judged, would have been of small areal extent. Small springs that flowed from the lower levels of the eastern flanks of the ridge provided some indication of the position of the main water table. Preconstruction drainage activities in the valley east of the ridge downstream from Fill A met strong flows from "buried springs" issuing from in situ materials. On the western flank of the ridge a spring in a cave between two more resistant beds provided a permanent domestic water supply.

Investigation holes drilled near the chicken shed (see Figure 2) indicated that the water table here was below canal invert level, but the holes were not deep enough to reach water. Holes drilled, and piezometers installed beneath Fill A were destroyed very early by construction activities, before any useful records could be obtained.

Four days before the canal failed, four piezometers were installed in Fill A. The water level in these was observed to rise each day, but it is not known whether this was in response to rising water levels in the fill, or whether the water levels were still approaching equilibrium.

## 4 OUTLINE OF SITE ENGINEERING

Water from lake McLaren passed along a canal that was in some places in cut, and in others in fill (see Figure 1).

In a number of places along the canal, buttress fills were placed to support the canal embankments.

The forebay, immediately downstream of the failed section of the canal, was a conventional concrete structure fitted with control gates, trash screens, and alarms and safety trips appropriate for the protection of the conduits and penstocks from hydraulic accidents.

The twin 2.54 m diameter, 600 m long low pressure concrete pipes with flexible joints which lead water from the forebay to the penstocks were laid in a trench that was subsequently back filled.

The 2 m diameter, ca 150 m long high pressure steel penstocks descended steeply to the powerhouse silted adjacent to, and in the upper tidal reaches of, the Wairoa River.

### 4.1 Canal

The canal was fed directly from Lake McLaren through an intake gate that was used to control he water level in the canal when operating, and to shut off the flow when necessary. The gate could be operated by remote control, or in emergencies by hand. As a prevention against overfilling, the canal had a 140 m long broad-crested side spillway capable of discharging 50 cumecs. The velocity of flow at a full load of 28 cumecs was 0.3 m/sec. At the lowest operating level the flow velocity was 0.65 m/sec. The flows from the four tributary streams that were crossed could be discharged directly into the canal through intakes, or when required could be passed under the canal through permanent culverts.

All soil materials needed for canal construction, apart from concrete and roading aggregate, were won on site. A mixture of so-called 'brown ash' was used as the low permeability canal lining. This was a mixture of the tephras that mantle the uppermost 4 to 5 m at the site (see Table 1). It includes the ca 1 m thick Hamilton Ash that lies ca 4 m below the surface. The design thickness of the lining was 500 mm. Where special conditions had to be met, the design thickness was increased to 1000 mm.

The lining material was required to meet specified standards of density and water content. Placement procedures were also specified, and these were aimed to ensure, among other things, that the lining did not dry out before canal filling.

The design acknowledged the need for adjacent materials to be compatible, but it made no specific provisions for general underdrainage of the lining. Nor did it provide a cut-off system below the lining to isolate the underlying deposits from the effects of canal seepage.

Special drainage was provided in a few areas, more particularly those were there was active inward seepage, high ground water levels or erodible material. In a few places where the underlying material was of low permeability and erosion resistant, no lining was placed.

The design slopes for ground lower than the canal was there horizontal to one vertical. Natural ground steeper than this was supported by buttress fills constructed with the same design slope angle. Local materials were used to form the buttresses. Drains were installed beneath these fills.

The designer's intention was that these buttresses would improve the stability of the natural slopes, by both reducing the steepness, and adding mass to the toes. In practice, because of the effects that the water lost from the canal had on the underlying in situ material, a greater loading was placed on these buttresses and embankments.

In the section of the canal that failed (see Figures 2 and 3), the north-trending canal was sited towards the eastern margin of the broad crest of an asymmetrical ridge. The eastern flank of the ridge has been shaped, and was continually being modified by natural erosion. Water issuing from the base of the slope induced slope instability by slumping. The process, although continuous, was not rapid on an engineering time scale. The canal invert was in cut, a few metres below ground level. The canal embankments and lining were all constructed of "brown ash".

A buttress fill (Fill A) was placed against the natural slope in layers subparallel to the natural slope. Before placement, drains were installed at the interface of the natural slope and the fill.

During construction the original swampy ground at the base of the natural slope was removed.

The contractor constructing the section of canal upstream from the forebay encountered unsatisfactory materials while excavating for the invert. These were removed over a distance of ca 75 m, and the thickness of the lining increased. It was noted, prior to the placing of the lining, that the underlying, in situ materials were permeable. The rate of infiltration of storm water was presumably consistent with the designer's expectations.

Only about 2 m of the canal bottom were excavated into original ground. The embankments on both sides, which form the canal trough, sit on original ground. The ash lining was placed on that part of the trough section that was below the original ground level; there was an overlap on to the embankments.

4.2 Events immediately prior to and following commissioning

Canal filling began on 25 February 1981. On 30 March seepages were observed at the toe of Fill D (see Figure 1). Flows increased, seepage water became discoloured, and, on 31 March, an arcuate crack 20 m long was seen above the seepage area. By 1 April the crack had widened, and the ground down stream of the crack had slumped 300 mm. Seepage at this time was ca 1 litre/sec. Remedial work was undertaken.

Seepages were observed also, and remedial work undertaken at Fills B, H, C, E and A. There seems to be little doubt that any one of these fills could have failed, but for this discussion, only the events at Fill A, which did fail catastrophically, will be considered.

A small, possibly-shallow slip occurred near the centre of Fill A in 1980. Cracks were observed at the top of the fill in April 1981. On 5 August 1981 a major crack opened up at he head of the fill. This was described as being 30 m long and about 50 mm wide: the downstream side had settled ca 150 mm. In places it may have opened up to 125 mm and settled 300 mm judged on photographs taken at the time. By the next day the crack had extended the full length of the fill (ca 100 m): later another crack appeared.

On 9 August a drain at the northern end of the fill, which up until this time had been discharging water at ca 20 litres/sec, suddenly had its flow reduced to about half this amount. Water then started flowing from the ground surface, presumably from in situ material. This flow was traced to a ca 2.5 m hemispherical cavity in the in situ material.

On 11 August, before remedial work could be completed, a "sinkhole" surfaced partway up the fill. this increased in size over two or three hours from ca 0.6 to 5.0 m in diameter. The depth of this hole is uncertain, one estimate was 6 m. Water rose, and soon "muddy" water was discharged at the surface of the fill. Remedial measures were undertaken to stabilise the sides of the cavity, and to prevent the discharge of fines.

The remedial work on the crack that had been formed on 5 August had been completed by 10 August. By this time also a network of observation points had been installed, and daily measurements on fill movements had been initiated: seepage was also being monitored. Observations showed that Fill A continued to move, both vertically and horizontally (see Figure 4). The movements suggested to the Committee of Inquiry that the failure was not a typical slip circle failure, but involved a shortening of the fill.

About 10 days before the collapse a second crack appeared. A third one was observed the night before the collapse. Yet another one developed just 80 or so minutes before the collapse occurred.

The failure occurred on Sunday 20 September. No one witnessed the initial stages of the collapse of Fill A, although a number of local residents, possibly alerted by the noise, did see, later, and vividly describe, the early and later stages, as viewed from various vantage points.

Faults on the local electrical system defined the time of the event. A local 11 KV power line which ran alongside the eastern bank of the canal lost power at 1.52 pm. Shortly afterwards an electrical trip at the Ruahihi Power Station, possibly caused by the loss of a 33 KV line along the western side of the canal, put the station out of operation. The tripping operated the relief valve which discharged water into the river (it was learnt subsequently that the penstock in use at the time had been partly drained before the turbine shut down). At 2.20 pm all generation in the TJGC system failed as the 33 KV line that crossed the canal was cut by collapsing canal banks (see Figure 5).

4.3 Factors possibly contributing to the failure

Many factors could contribute in some way to a failure such as this. the overall concept of power development in the area was discussed 25 years previously. In the intervening years there were many changes of staff, specialist advisers, and of layouts. Changes in all these were taking place until shortly before approval was given to construct. Pressures of one kind or another did not help either (particularly financial, time, and the complex constructural interfacing in the final stages of construction of the canal, near Fill A, and of the the forebay structure). These various factors were not unique to this project, but they may have played a minor part in the failure.

Without doubt the technical factors that contributed most to the failure were the physical properties of the volcanic soils. In particular the high erodibility of the pumice silts, their brittle and non-healing nature that enabled them to maintain a 'tunnel' without this collapsing, and the effect that the addition of water and loading had on their structure.

The steepage losses from the canal became evident soon after this was filled for the first time. Slopes failed, and subsurface erosion produced "tunnels" and "sinkholes" (locally known as

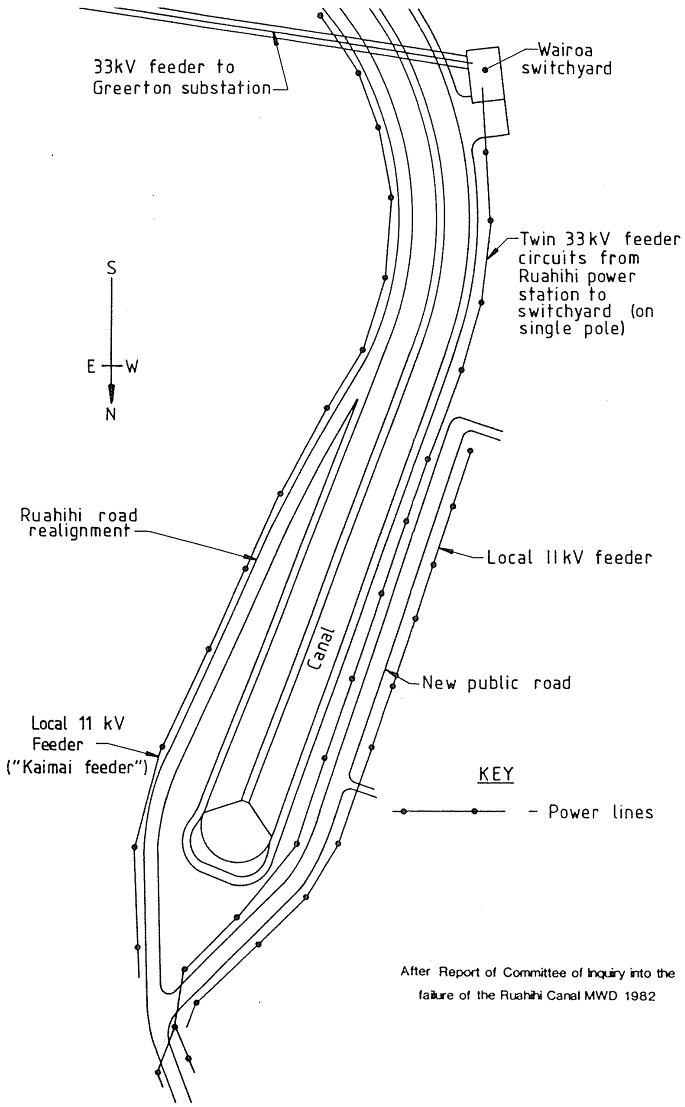



FIGURE 5 LOCATION OF POWER LINES IN FAILURE AREA

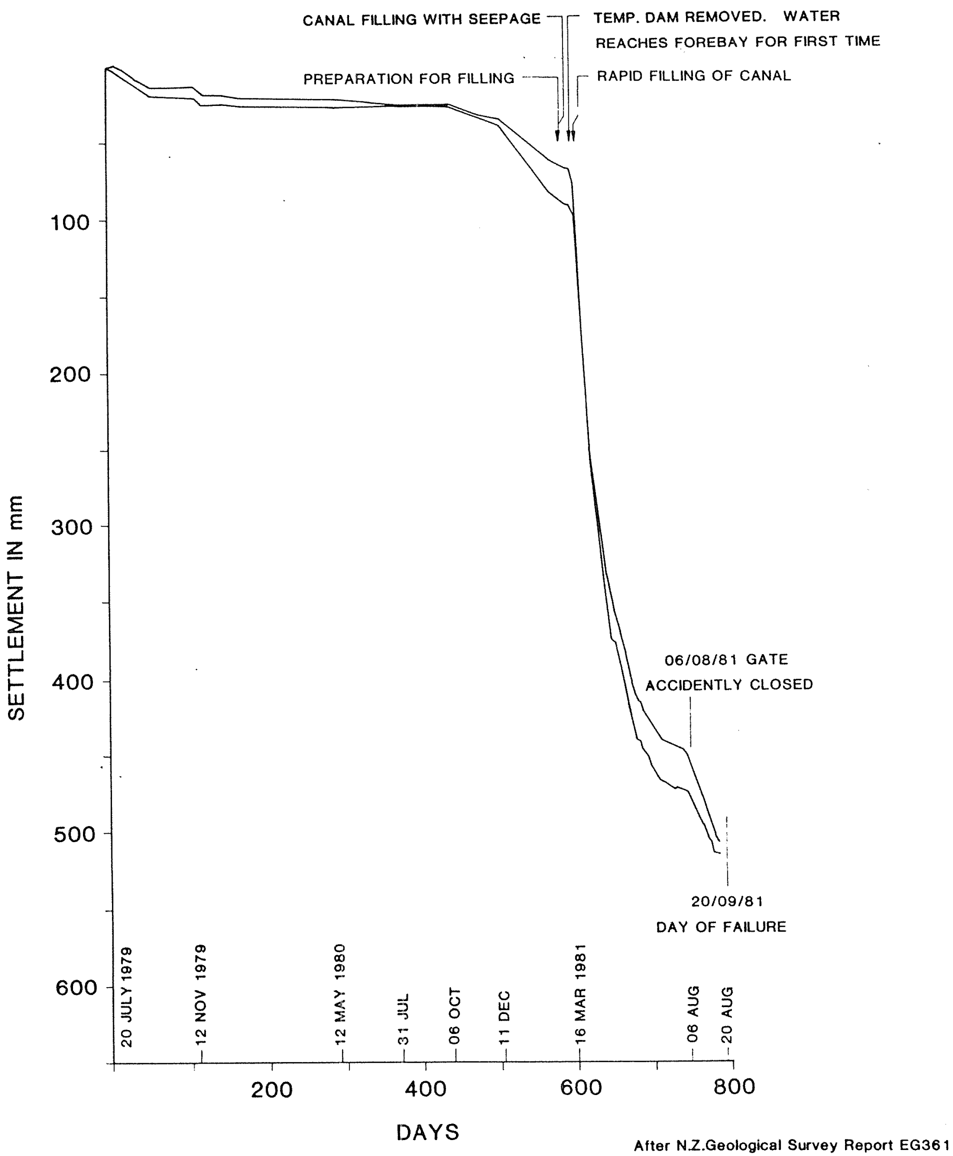



FIGURE 6 RATE OF SETTLEMENT OF INTAKE (FOREBAY) STRUCTURE

"tomos"). The effect of canal filling was demonstrated unequivocally by the rate of settlement of the forebay structure (see Figure 6).

Geological investigations in 1956 and again in the mid seventies had indicated the nature of the volcanic soils. The full significance of these indications was apparently not recognised by the designers, and appropriate additional detailed studies were not undertaken to confirm the distribution and likely behaviour of these brittle, highly structured, sensitive, erosive and possibly dispersive soils. The potential instability of the natural country was not identified, and there was no exploratory drilling between canal distance 2500 m and the forebay, the area that eventually failed.

## 5 CONCLUSION

The only merit in reviewing a past disaster lies in the hope that this will lead to a greater understanding of the factors that caused it, that these maybe identified in the future, and that a similar disaster may be avoided.

The main causes of the failure of the Ruahihi canal on Sunday 20 September 1981 have been attributed to the nature of the volcanic soils, and the canal lining which did not (and was not designed to) effectively isolate the underlying materials from canal water.

The soils along the canal route were underconsolidated, low density and non-plastic materials that were brittle, both in situ and when compacted. they were also highly erodible, so that cracks and "tunnels" once formed were readily extended and enlarged, but were not readily self-healing.

Water seeping through the canal lining caused subsurface erosion and soil structure collapse. Hydrofracturing may have occurred beneath the canal.

Geological investigations in 1956, and again in the mid seventies indicated, in descriptive terms, the sensitivity of these materials, and advised additional detailed studies. The significance of these indicators and of this advice, was either not recognised by planners and designers, or in the numerous staff changes over the years these concerns were not passed on or followed up.

The need is emphasised for thorough investigations and active and close co-operation of planners, engineering geologists and geotechnical engineers at all stages of a project, and periodical reviews of previous and current work and objectives.

During construction, monitoring systems of ground movement, water seepage, water levels and structural behaviour should be well designed, carefully measured and documented. it is most important that the records obtained are frequently and carefully studied by qualified people who can detect trends, investigate relevant anomalies, and initiate precautionary or remedial action when necessary.

When anomalous conditions and ground responses are met, planners and designers need to be able to stand aside from the usual pressures (such as financial, contractural and time constraints) and consider technical problems. Where the problems met are beyond the experience of staff, specialists should be consulted.

In summary, it is judged that early investigations gave an adequate indication of the materials to be expected. The engineering significance of these indications, especially of the effect that water loss from the canal would have on the country well above the piezometric surface, was not, however, recognised in the planning and design stages. Even after canal filling, the significance and causes of the loss of water, the slope instability of the fills, the development of the "sinkholes" and the rate of forebay settlement were not understood, known or further investigated. Yet information on the properties and likely behaviour of these kinds of soils was available in the country.

A decision to restore power generation was made in April 1982. The remedial works were completed in May 1983, at a cost of NZ$17 million, and power generation was restored in June 1983 (see Appendix 2).

## 6 ACKNOWLEDGEMENTS

The writer is pleased to acknowledge the helpful and constructive comments made by Dr R.D. Northey, and by Messrs G.T. Hancox and R.D. Beetham of the New Zealand Geological Survey, who read the text.

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## APPENDIX 1

### TEPHRA, IGNIMBRITE, ALLOPHANE

#### Tephra

The tephras (volcanic ashes) that mantle the region surrounding Ruahihi, and indeed much of central North Island, are the products of plinian eruptions. In these, gaseous blasts drive volcanic ejecta upwards, and the prevailing winds distribute them widely. When deposited they are cool or warm, well-sorted materials with angular pumice, minor rock fragments and free crystals. The thickness of the deposits decrease exponentially with the distance from the source. Ash showers are commonly distinctive and serve as good time-stratigraphic marker beds. They mantle the topography without greatly changing landforms.

Ignimbrite

Ignimbrite is the product of a pyroclastic flow from a plinian or ultra plinian eruption. The erupted column, probably many tens of kilometres high, collapses, causing a very high velocity surge radially from the source. Ignimbrite can be a poorly sorted mixture of pumice blocks, ash, minor rock fragments and free crystals. The flow conserves heat energy, and the material can be hot enough to fuse the glass matrix. When this happens the resultant deposit is a welded ignimbrite. More distant from the source, flows may be too cool to facilitate fusion but still hot enough to convert trees growing in their path into charcoal. These flows can greatly modify landforms, filling in valleys and hollows, and producing planar upper surfaces.

Allophane

Allophane is a clay mineral with a very small particle size, but with a large surface area. It is a very small, hollow, generally water-filled spherical structure surrounded by tunnel-like pores. When they dry the pores tend to collapse, preventing the escape of water. Allophanic soils commonly have high natural water contents and low bulk densities. They have a firm friable consistency when undisturbed, but break abruptly to a soft paste when remoulded. When air dried, the clay particles aggregate, and there are marked irreversible changes in physical properties. There is, for example, a decrease in water retention and an increase in permeability and erodibility. These soils are not suitable as impermeable lining materials for water retaining structures that are subject to widely fluctuating water levels, as they could exhibit significant deterioration in performance after exposure to drying above the water level.

APPENDIX 2

POST - FAILURE REMEDIAL WORKS (1982 - 1983)

Contributed by Beca Carter Hollings and Ferner, Consulting Engineers

The remedial works comprised some 300 000 cubic metres of remedial earthworks, the fabrication and installation of 1100 metres of 3 metre diameter pressure pipeline, the repair and lining of two kilometres of canal and the construction of a new penstock intake structure.

Penstock Works

The original penstock structure comprised 600 m of twin 2.54 m diameter low pressure buried concrete conduit connecting into ca 150 m of twin 2.0 m diameter high pressure steel penstock descending a steep escarpment above ground to the powerhouse. The new penstock route was selected to skirt to the west of the collapse area in natural ground, being extended some 1050 m upstream from the original penstock intake structure and comprising:

* 75 m of the original twin 2.54 m concrete conduit relaid below ground.

* 520 m of single 3.06 m diameter reinforced concrete pressure pipe laid below ground.

* 560 m of single 3.20 m diameter steel pipe in typical lengths of 20 m connected by Dresser couplings and supported above ground on concrete pad footings.

* Each underground concrete pipe joint in the new penstock structure is monitored for leakage by means of a pipe/manhole system.

Penstock Intake Structure

The penstock intake structure was sited upstream of the collapse area in natural ground. Special attention was paid to the design of connection details between the structure and the adjoining canal and penstock to accommodate expected differential settlements. Drainage is provided beneath the structure to control natural groundwater levels and to monitor any leakage from the canal in the vicinity of the canal-forebay transition. Provision is made for the monitoring of the settlement of the structure and the adjoining penstock foundations.

Canal Repair

The original canal was 3350 m long with a trapezoidal cross section, the sides and base being generally lined with a 0.5 m thick layer of ash. In some sections of deep cut a lining of filter material and rip-rap replaced the ash material. Pipe diversion systems were installed in the embankment fills constructed across the four tributary streams traversed by the canal, and extensive subsoil drainage systems were installed in the base of these fills.

The repaired canal is 2261 m in length, and has a U-shaped cross-section with its invert raised to 1.5 m above the original (pre-collapse) design level. Extensive repair to the original ash lining was required over most of the canal length due to slumping and cracking induced by the rapid drawndown during the collapse. The slump material was spread and compacted in the invert, the slump scarps benched out and filled with compacted ash and hardfill, and the remaining ash lining reworked and compacted to form the new U-shaped profile.

An impervious lining system was installed over sections of the new canal where fill embankments adjoin the canal and where natural ground slopes steeply away from the canal, totalling 1650 m in length. The lining installation comprises:

* an undercanal drain along the canal invert to intercept ground water flow or leakage through the lining, and consisting of a perforated pipe laid in drainage aggregate and enclosed in filter fabric.

* a filter fabric layer covering the subgrade.

* a drainage layer to convey groundwater seepage and leakage water to the underdrain and consisting of a 12 - 2 mm clean graded 'pea gravel'.

* an impermeable membrane of high density polyethylene (HDPE) sheet 2.0 mm in thickness, the sheets being jointed by an in situ heat welding process.

* a protective layer of precast concrete paving slabs, generally 50 mm in thickness.

In the two main cut areas totalling 700 m in length, the canal surface was protected against erosion and drawndown slumping by a lining layer of rip-rap material.

Flows from the extensive undercanal drainage system are monitored regularly, as are flows from subsoil drainage systems installed in embankment fills adjoining the canal. Groundwater levels and settlements are also monitored at critical locations along the canal sides and in cut and fill slopes adjoining the canal.