MANUHERIKIA IRRIGATION SCHEME

GEOTECHNICAL STUDIES

FOR FEASIBILITY REPORT

Prepared by

Ministry of Works and Development

Civil Engineering Directorate

Special Projects Design Office

Head Office

Special Projects Report 1986/28

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CONTENTS

APPENDIX A

1. INTRODUCTION

Dunedin MWD District Office are carrying out a study to determine the feasibility of headwork repairs for the Manuherikia Irrigation Scheme. Special Projects Office has been commissioned to examine the geotechnical aspects of this project. A two stage study has been carried out.

The Stage I Geotechnical Studies Report (ref 64/7/1/3 of 30 July 1984) was prepared in response to the MWD district office brief of 18 June 1984. The field investigations were confined to a walk over survey, surface mapping of materials and landslides and an office study of aerial photographs. The report concluded that the current instability on the right bank was such that an adequate service life could not be assured for pipe or race conduits on a bench traversing the slope. Subsequent to that report the MWD district office engineers relaxed the water race outage requirements of the original brief and in consultation with the geotechnical engineer, prepared the Stage II Geotechnical Investigations Brief.

The objective of the Stage II Geotechnical Investigation is to define as accurately as possible the geological features of the right bank and the mechanisms which have formed it. The Stage II investigations are not aimed at option specific questions, however where possible, general comments and conclusions with regard to the current water race security and to proposed alternative aqueducts have been reported. The results of the Stage II investigations are described in this report.

2. BACKGROUND TO THE PROBLEM

2.1 Introduction

The water race supplying the Manuherkia Irrigation Scheme passes through the Ophir Gorge in a race on the true right bank. Slope instability in the gorge has resulted in general distress to the race and on a number of occasions, major slides have either blocked or destroyed substantial lengths of race.

2.2 Geological Reports

Previous geological investigations in the Ophir Gorge have been described in the following reports:

(a) NZGS Report EG362, February 1982, An Engineering Geological Assessment of the Manuherikia Irrigation Water Race, Central Otago, B R Paterson.

(b) NZGS Report EGI 84/033, August 1984, Manuherikia Irrigation Scheme Feasibility Study - Ophir Gorge Water Race, B R Paterson (prepared as part of Stage I of this investigation).

The Stage II geotechnical investigations have included seismic survey, further surface geological mapping and subsurface investigations. The results of the seismic survey are reported in:

(a) Central Laboratories Report 2-85/12, June 1985, Manuherikia Irrigation: Geophysical Survey, A J Sutherland.

The results of the Stage II geological mapping and subsurface investigations are reported in:

NZGS Report E393, 1985, Engineering Geology of the Ophir Gorge Headworks, Manuherikia Irrigation Scheme, B R Paterson.

2.3 Existing Ophir Gorge Aqueduct

There is major instability on the true right bank of the Ophir Gorge where the race carries water between the No 1 and No 2 tunnels.

The existing aqueduct traverses the oversteepened toe of landslide debris. The most serious problem in the Ophir Gorge is caused because the currently active landslides will eventually make the aqueduct inoperable in a way such that either the length of time to restore water supply (outage) is unacceptable to the farmers or the repair of the aqueduct is no longer feasible or economically viable.

Dunedin District Office, Water and Soil have indicated that the outage table (table 1) represents the "acceptable" level of risk.

TABLE 1: OUTAGE TABLE

Annual Probability of Occurrence in the Season (August-April)	Acceptable Outage (days)
1.0	5
0.1	7
0.05	20

2.4 Alternative Aqueduct Options

Because the existing aqueduct will not provide an assured water supply, several alternative aqueducts have been proposed for further study.

(a) A right bank pipeline on the existing alignment using pipe bridges and siphons to traverse the worst areas of instability.

(b) A right bank pipeline with a short right bank tunnel to avoid the worst areas of instability.

(c) A long tunnel towards Chatto Creek from the downstream end of the No 1 tunnel.

(d) A left bank tunnel with aqueduct structures at each end to carry the water across the Manuherikia River.

(e) A left bank pipeline with aqueduct structures as for (d).

(f) A low level weir with a pump to lift water directly from the river into the No 2 tunnel inlet.

The various options are described in detail in the text and diagrams of:

Manuherikia Irrigation Scheme Headworks Repairs Feasibility Report, MWD Dunedin District Office Report, 1985.

3. MATERIALS

3.1 General

The following material types appear on the geological plans and sections; schist, landslide debris, a crush zone, basal gouge, colluvium and loess. In many cases the boundary between material types is indistinct, but the material types chosen have proved to be a convenient means of classifying the material present in the Ophir Gorge. The distribution of materials is shown in figure 2.

3.2 Schist

The undisturbed "basement" rock is quartzo-feldspathic schist which occurs at varying depth throughout the area of interest. Schist outcrops and the core obtained from the subsurface investigation indicate that the schist is very blocky, with strength being controlled mainly by joints, and to a much lesser extent by schistocity. An outcrop of schist rock near the No 2 tunnel portal is shown in plate 1.

3.3 Landslide Debris

3.3.1 General

Most of the serious problems in the Ophir Gorge are caused by instability in the landslide debris material. The landslide debris is derived from mass movements in the in situ schist rock. This material grades from the very large disjointed schist blocks of the central ridge to a very chaotic landslide debris consisting of schist boulders in a silty clay matrix. Plate 2 shows a landslide debris slope. This slope has been disturbed by surface "unravelling" so the fabric is not representative of chaotic landslide debris, however the range of particle sizes is representative of chaotic landslide debris.

3.3.2 Strength of Landslide Debris

The material is generally composed of a large range of particle sizes from silt to large blocks. For this reason a laboratory assessment of the strength of the landslide debris would be extremely difficult. Salt (1983) describes a study of naturally occurring oversteepened schist derived landslide debris slopes in the Clutha and Kawarau Gorges. A Mohr-Coulomb strength envelope was derived by Salt using resistance envelope techniques (ie, back analysing slopes on the point of failure), and is reproduced in figure 1. The strength parameters derived by Salt have been used for the Ophir Gorge materials, because the mechanism of forming landslide debris in the Ophir Gorge is similar to the examples back analysed by Salt. The strength envelope was confirmed by back analysing a dry landslide in the Ophir Gorge.

Plate 1
Outcrop of Schist Rock Near the No 2 Tunnel Portal

Plate 2
Exposure of Landslide Debris on Access Track

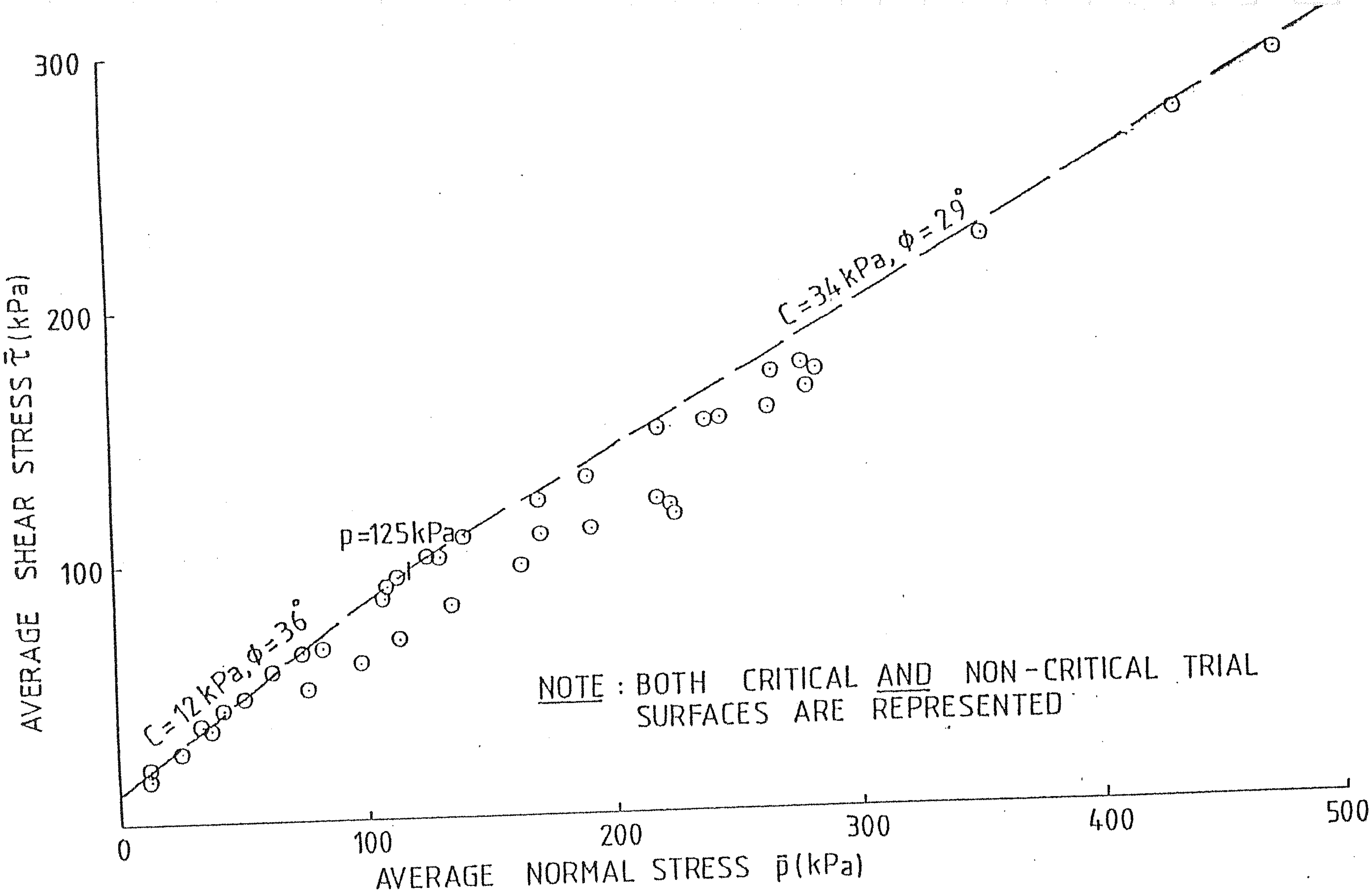


FIGURE 1

RESISTANCE ENVELOPE FOR EXISTING SLOPES IN SLOPE DEBRIS (SALT 1983)

A bilinear strength envelope was adopted for the landslide debris.

$p < 125$ kPa (saturated)	$c = 0$ kPa $\phi' = 36°$
$p < 125$ kPa (dry)	$c = 12$ kPa $\phi' = 36°$
$p > 125$ kPa	$c = 34$ kPa $\phi' = 29°$

Because these strength parameters have been derived by back analysing landslide debris above the water table, the strength parameters include effects such as soil suction. The loss of soil suction upon submergence is modelled by reducing cohesion to zero (where $p < 125$ kPa).

3.4 Crush Zones

There is a crush zone between the landslide debris and the schist basement rock composed of a fragmented material varying from broken rock to a more conventional silt clay gouge. The crush zone was probably formed on some pre-existing defect.

The crush zones have developed further and extended into previously unsheared areas during landsliding. Salt (1983) has carried out laboratory and field studies of the strength of the crush zones from the Cromwell and Kawerau Gorges. The results of the testing and field studies from the Kawerau and Cromwell Gorges will be applied to the Ophir Gorge materials.

The Kawarau and Cromwell Gorge crush zone material, has a laboratory residual shear strength of the order of 23-25° (Salt 1983). Field mobilised shear strengths can be determined by back analysing currently active slides. The field mobilised shear strength $c = 0$, $\phi' = 28.5$ is significantly larger than the laboratory residual strengths. The difference in strengths is attributed to the following factors:

(a) the coarser fraction of the crush zone was removed before laboratory testing;

(b) there are large variations in the crush zone composition and in the crush zone thickness and in some places, the crush zone may be pinched out by schist blocks, giving rock to rock contact;

(c) waviness of the crush zones seams results in field strengths greater than laboratory residual values.

3.5 Colluvium

A thin layer of colluvium overlays some of the right bank slopes in the Ophir Gorge. This material generally contains schist derived gravels and boulders aligned down the slope in a matrix of fine grained slope debris. Because of the method of deposition, colluvium will exhibit anisotropic strength. The strength parallel to the slope will be less than the strength perpendicular to the slope. The strength of the colluvium is required to assess the stability of the colluvium slopes above the aqueduct and above the access roads. Experience with similar materials elsewhere suggests that the colluvium shear strength would be approximately $\phi' = 28°$ on a plane parallel to the slope.

3.6 Loess

Loess deposits have been mapped in the flatter upper slopes of the basin, eg around the borrow pit. This material is composed of a narrow range of particle sizes, predominantly silt and fine sand. Because of the location of the loess deposits, this material does not have any impact on the feasibility of any of the aqueduct options.

GRAPHIC SCALE 0 50 100 200 300 metres	ORIGINAL SCALE Mapped BRP	DSIR	FIGURE 2 DESIGN SECTORS — Engineering Geology Section	SCALE 1:2000
EXPLANATION: PLAN BASED ON NZAM P219. GEOMORPHIC BOUNDARIES ARE NOT CORRECTED FOR PHOTOGRAPHIC DISTORTION AND HENCE ARE APPROXIMATE ONLY	Dr. BRP / Tr. LVL	N.Z. GEOLOGICAL SURVEY — Ian Speden	MANUHERIKIA IRRIGATION SCHEME — GEOLOGICAL AND GEOMORPHIC PLAN - OPHIR GORGE	DRWG. NO. / FILE / Sheet of

4. A GEOTECHNICAL ASSESSMENT OF THE OPHIR GORGE AQUEDUCT

4.1 General

In order to describe and assess the geotechnical engineering implications of the geology in the Ophir Gorge, the existing aqueduct and the slopes above it are divided into five "design sectors". For the purposes of this report, "design sectors" are defined as areas of sufficiently similar geological and topographical characteristics to enable the typical section for the sectors to adequately represent the full range of conditions in the sectors. The design sector boundaries are illustrated on figure 2 and plate 3.

The existing aqueduct has been surveyed, and distances along the race are quoted in terms of this survey. The upstream end of the race is at RS 0 and the inlet portal of the No 2 tunnel is RS 1088.

4.2 Design Sector A RS 0 to RS 226

Design sector A extends from the desilter in the north to a landslide covering the race in the south. The slopes above the aqueduct are mapped as colluvium overlying in situ schist with numerous rock bluffs, figure 2 and plate 4.

The colluvium slopes are standing at 33°. Experience with similar materials in the Cromwell Gorge indicates that the material strength of colluvium is variable but that the strength is generally in excess of $\phi' = 28°$. The landslide scarps apparent on plates 3 and 4 indicate that the slopes may be over-steepened. It is postulated that the slopes are standing at 33° because of an apparent cohesion due to negative pore pressures (capillary action).

There are no serious geotechnical engineering problems in design sector A. However, the conditions currently exist for shallow slides in colluvium. These shallow slides could occur during periods of heavy rainfall.

4.3 Design Sector B RS 226 to RS 436

Design sector B comprises the area called the central ridge. This is mapped as disjointed schist/landslide debris, figure 2 and plate 5. The engineering geologist describes the surface of the central ridge as a series of bluffs of relaxed open jointed rock which appear to be piles of blocks of disjointed rock. The rock has been broken up during a landslide or rock avalanche involving limited movement, enough to break up the rock but not fragment it.

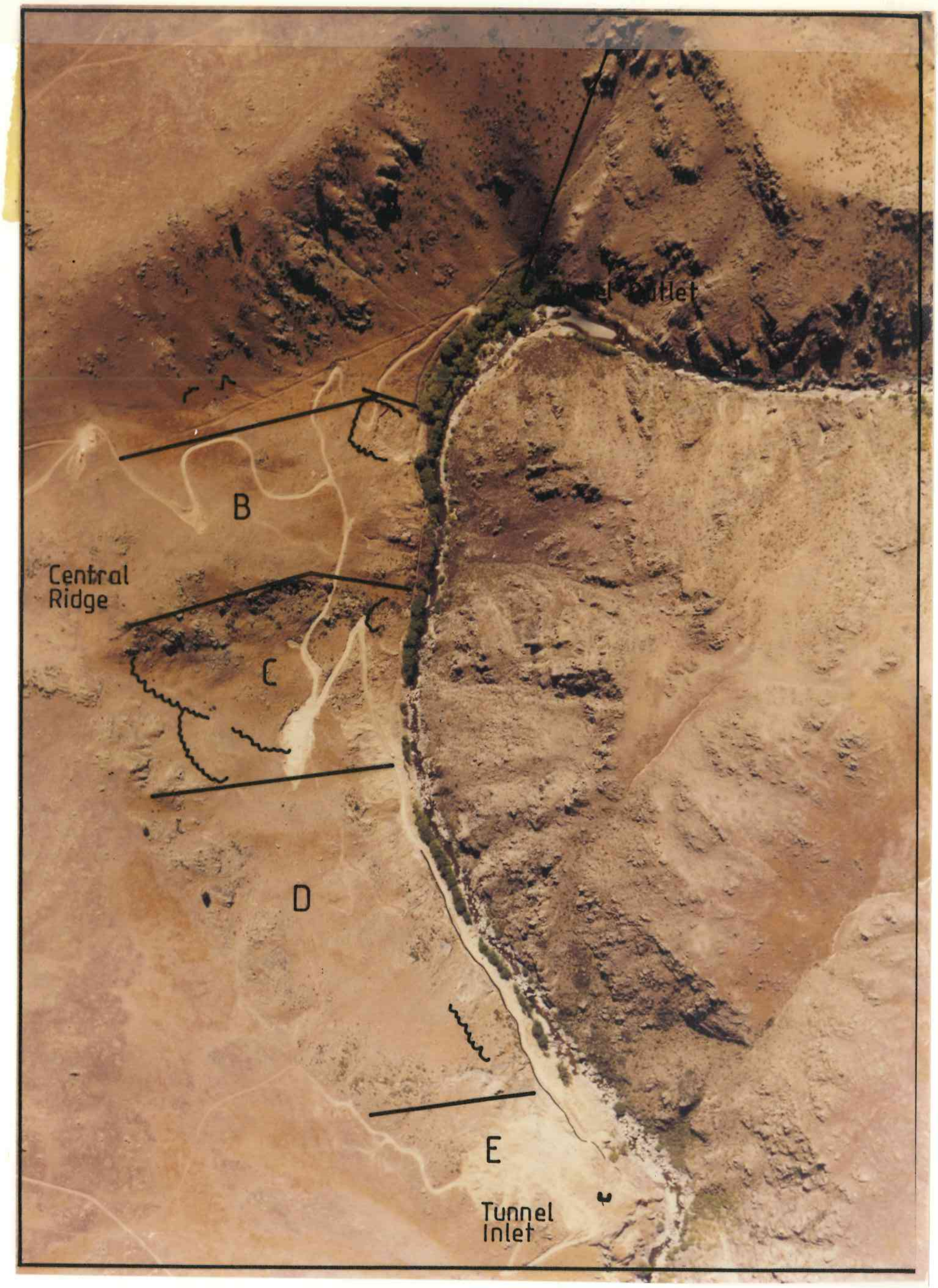


Plate 3
Aerial Photograph of the Ophir Gorge
Showing Design Sectors

Plate 4
Aerial Oblique of Colluvium Slope
Design Sector A

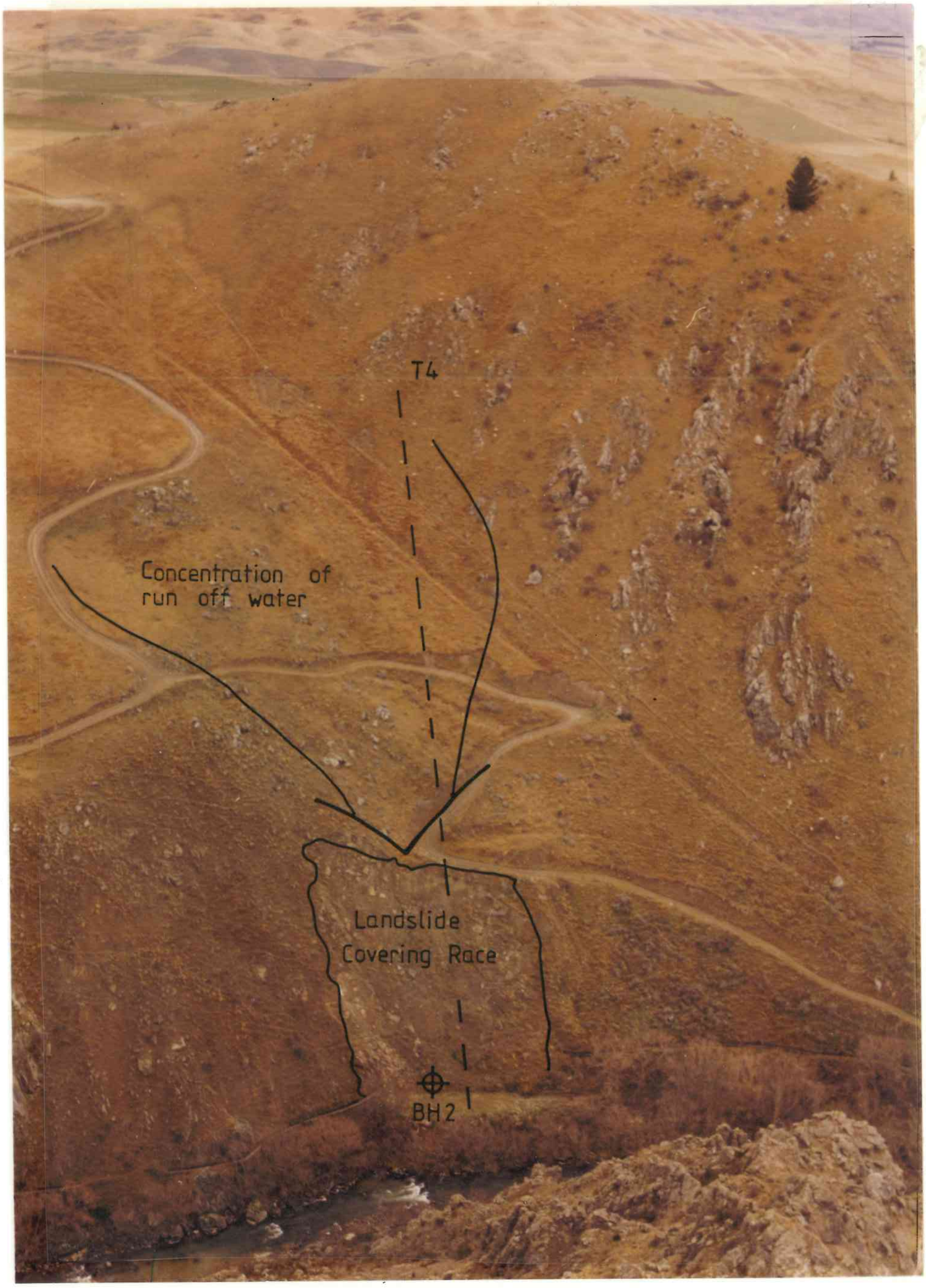


Plate 5
Aerial Oblique of the Central Ridge
Design Sector B

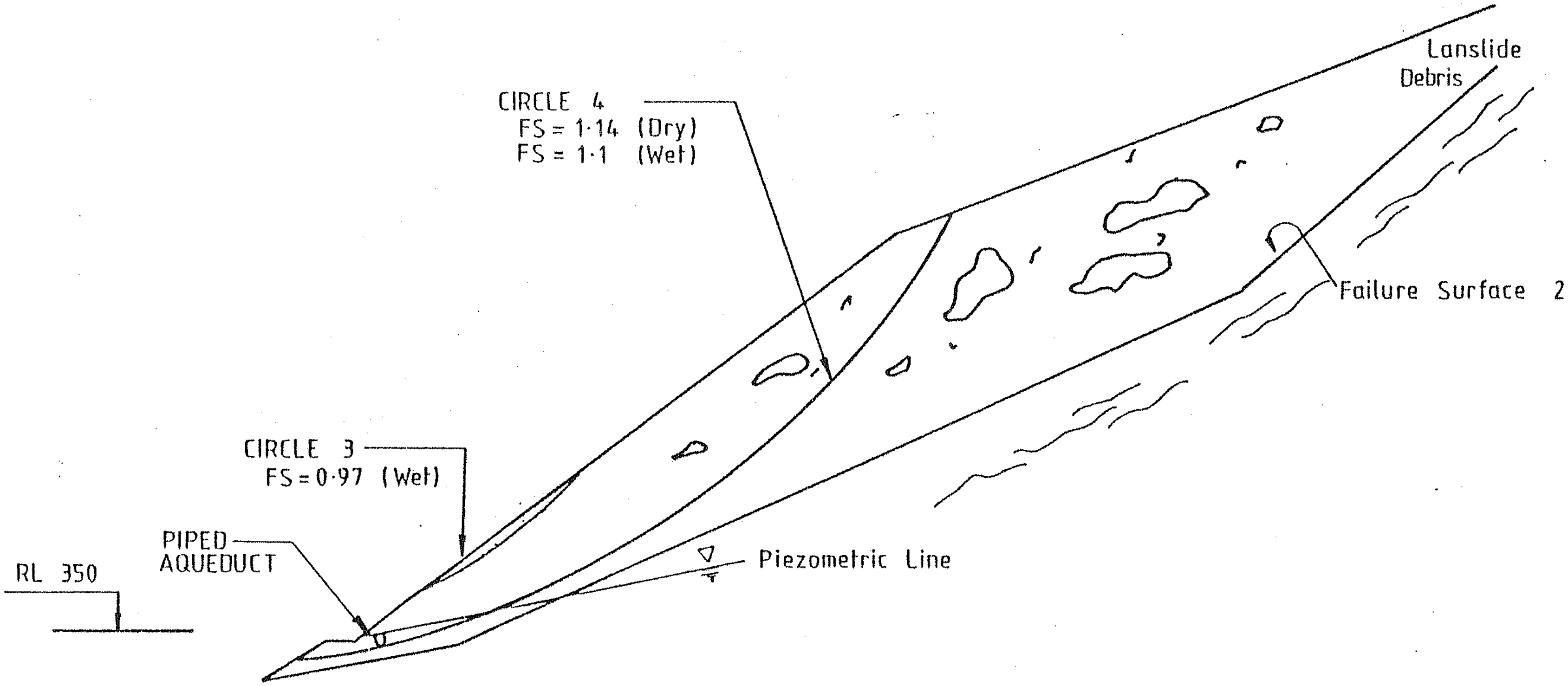


FIGURE 3 CROSS SECTION AT T2·5
(Half way between T2 & T3)
Scale 1:1000

The geotechnical assessment of design sector B will concentrate on two types of landslide. A deep seated landslide slipping on a crushed rock zone between the disturbed schist rock and the basement schist rock and shallower landslides in the oversteepened disturbed schist rock.

The results of a limiting equilibrium analysis of the deep landslide are summarised in table 2.

TABLE 2: LIMITING EQUILIBRIUM ANALYSIS DESIGN SECTOR B

Slide Surface	Strength Parameters See Section 3	Slope Condition	Factor of Safety
Failure Surface 1 Basal Landslide	$\phi = 28°$	Dry	0.89

The stability of the deep seated landslide was assessed by analysing "failure surface 1", shown in figure 4. The surface is drawn through the interface between disjointed rock and basement schist at seismic line T3. The low factor of safety (FS = 0.89) and the evidence of possible movement at race level, plates 6 and 7 is consistent with the slow, probably intermittent, creep movements that appear to be occurring on this surface. Plates 6 and 7 show two examples of the evidence of recent movement of the central ridge. Plate 6 shows a crack in handplaced rockfill. There is a horizontal displacement of the wall across the crack. Plate 7 shows a section of the race which has been pushed outwards. (Note the repairs of the crack in the outside concrete wall of the race.)

Large variations in the fabric and structure of the distrubed schist mean that shallower landslides in disturbed schist rock are not amenable to analysis. Therefore the geotechnical assessment of this type of landslide has been based on observation and the engineering geologist's recommendations.

4.4 Design Sector C RS 436 to RS 600

Design sector C extends from the central ridge in the north to where the access track intersects the aqueduct in the south, plate 8, figure 2.

There is direct evidence of three types of instability in design sector C. A limiting equilibrium analysis of each type of landslide was carried out, the results are summarised in table 3.

Plate 6
Cracks in Handplaced Rockfill
(Indicates Deep Seated Movement)

Plate 7
A section of the race pushed outwards
(Note : cracks in outside wall)

Plate 8
Aerial oblique of Design Sector C

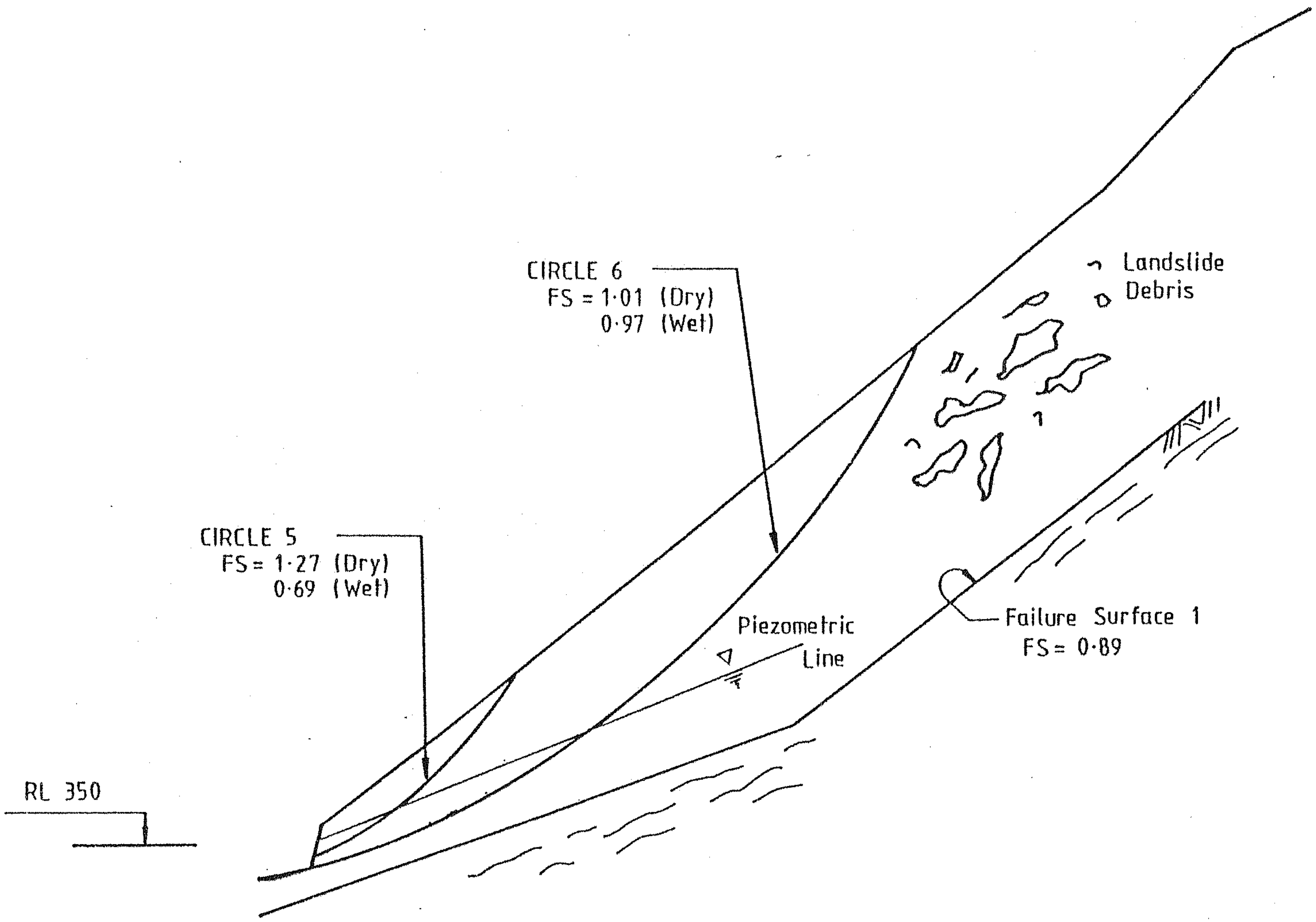


FIGURE 4 CROSS SECTION AT T3
Scale 1:1000

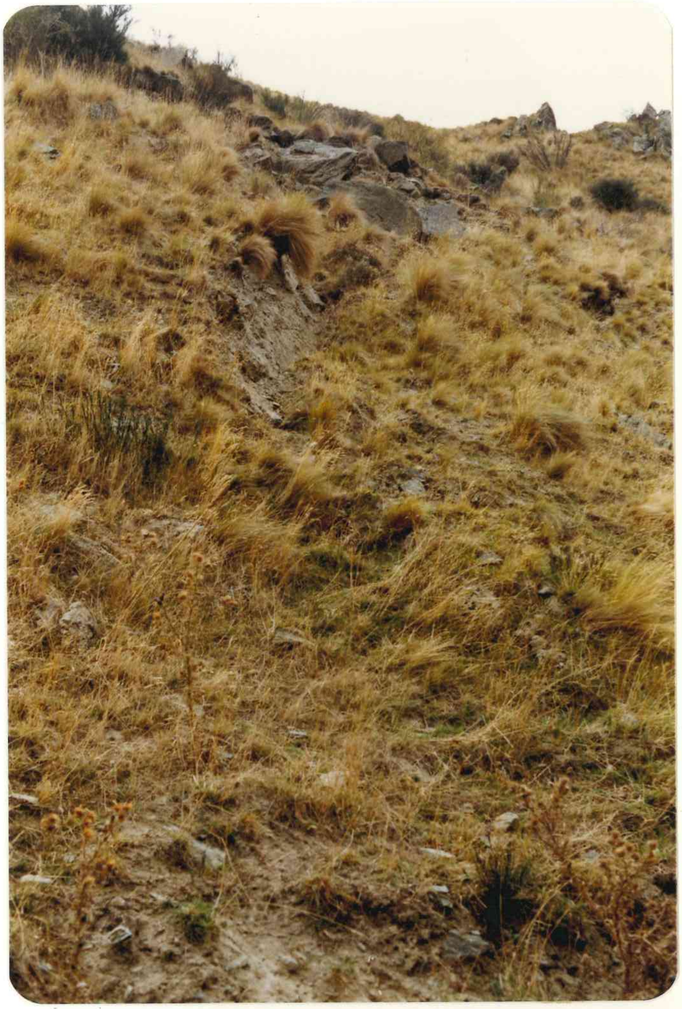
TABLE 3: LIMITING EQUILIBRIUM ANALYSIS OF DESIGN SECTOR C

Slide Surface	Strength Parameters	Slope Condition	Factor of Safety
Circle 3 (T2.5) Shallow Unravelling	c = 0, ϕ = 36°	Wet	0.97
Circle 4 (T2.5) Deep Circle	c = 34, ϕ = 29° c = 34, ϕ = 29°	Dry Wet	1.14 1.10
Circle 5 (T3) Active Slide	c = 12, ϕ = 36° c = 0, ϕ = 36°	Dry Wet	1.27 0.69
Circle 6 (T3) Deep Circle	c = 34, ϕ = 29°	Dry	1.01
Failure Surface 2	ϕ = 28°	Dry	1.03

The three types of instability observed in design sector C are: a shallow surface "unravelling"; a "circular" slide in landslide debris; and a basal landslide.

Surface unravelling is used to describe very shallow slides of the order of 1 m thick. Circle 3 models this type of instability (table 3, figure 3, plate 2). Analysis of the dry slope results in a high factor of safety. Because the grading of landslide debris indicates that it should behave as a cohesionless material, it is postulated that the low confining stress cohesion value (c = 12 kPa) is due to negative pore pressures generated by capillary action. It is further postulated that on wetting this cohesion is reduced to zero. Circle 3 is reanalysed assuming zero cohesion. The factor of safety for this circle reduced to FS = 0.97, a value consistent with the geological observations. Observations and analysis indicate that surface unravelling will continue to migrate headwards in the oversteepened schist debris face.

The landslide debris slide at RS 436 to 515 is typical of the second type of instability observed in design sector C. This landslide appears to be about 5 m deep plates 9, 10 and 11. A limiting equilibrium analysis of this landslide has been carried out (circle 5 of table 3). If a dry slope is assumed, circle 5 has a satisfactory factor of safety (FS = 1.27). A large volume of water is seeping from the slope beneath the race. It is assumed that this flow originates from open cracks in the side of the buried pipe (plate 12). In addition to these water seeps the surface of the landslide above the buried pipeline is also damp.



Plates 9, 10 & 11 Landslide at RS436-515

Plate 12 Water Seep from Buried Pipe
Landslide RS 436-515

Plate 13 Tension Cracks in the borrow pit
(The core boxes give scale)

This water is probably surface run-off from the upper slopes concentrated by the access road and fed from the corner of the road to the head of the landslide, plate 11. The observed groundwater conditions are modelled by assuming the groundwater conditions as drawn in figure 4. For wet slope conditions, the factor of safety of circle 5 reduces to FS = 0.69. This result is consistent with the observations of recent landslide movements but also implies that the water table assumed in figure 4 is pessimistic. Analysis and observation suggest that this slide will migrate headwards and will be effected by rainfall.

The third type of instability observed in design sector C is a basal landslide. The presence of this slide is inferred from the tension cracks in the upper basin, plates 3, 8 and 13, and from toe movements at the aqueduct level. Failure surface 2 has been used to model the basal landslide. A factor of safety (FS = 1.03) was calculated assuming a dry slope. A dry slope was analysed because not enough information was available to accurately infer a groundwater table for this landslide. If a groundwater table was present, then the calculated factor of safety would be lower. The dry slope factor of safety close to one is consistent with the engineering geologist's observations that the slide is currently moving.

Headwards migration of the shallow landslides described earlier in this section will result in the removal of material from the toe of the basal landslide, this will accelerate the future rate of movement of the basal landslide.

In addition to the currently active landslides, the limiting equilibrium analysis indicates that it is possible for slides 10-15 m deep to occur in landslide debris. Circles 4 and 6, described in figures 3 and 4 and table 3, model these slides. A dry slope factor of safety (FS = 1.01) was calculated for these slides. Headward migration of shallow landslides is removing material from the toe of this type of potential landslide, further reducing the stability. First time slides 10-15 m deep may occur in the future.

Stakes were installed in design sectors B and C during January 1985. The stakes were resurveyed nine months later. The short monitoring interval means the results must be interpretated with caution. The resurvey showed average downslope movements of 35 mm (a rate of 44 mm a year). The survey information provides additional confirmation of the geological evidence and the geotechnical analysis.

4.5 Design Sector D RS 600 to RS 900

Design sector D extends from the point where the access road meets the aqueduct in the north to the in situ rock bluffs in the south, figure 2, plate 14. The slopes immediately above the aqueduct are composed of landslide debris. There is a break in

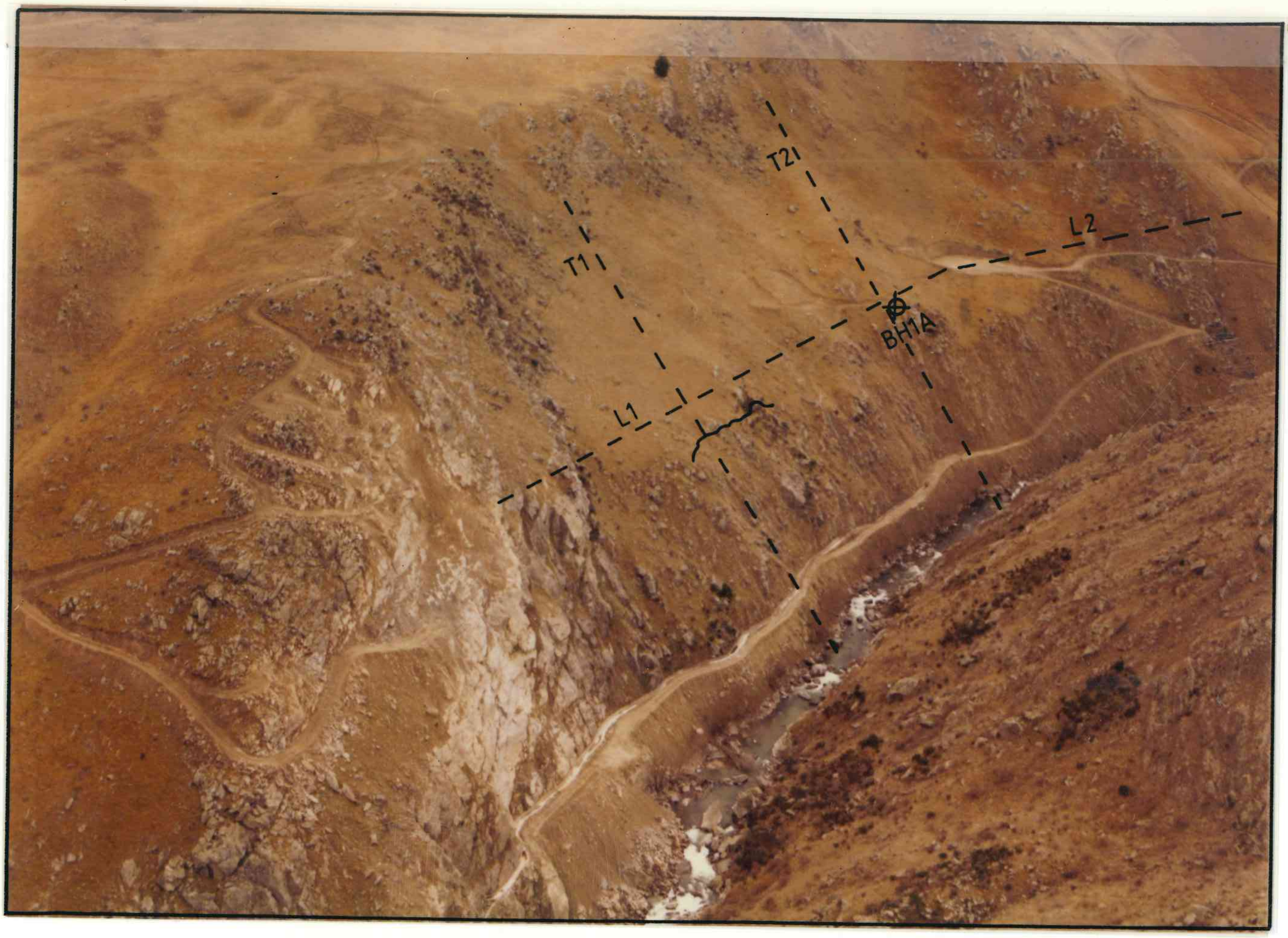


Plate 14
Aerial Oblique of Design Sector D

Plate 15
Tension Crack for Circle 8 (Figure 5)

slope approximately 50 m above the aqueduct which separates an oversteepened face (at one to one) from a steep slope (two to one). A deep open tension crack approximately 30 m long traverses along the break in slope, plates 14 and 15.

A limiting equilibrium analysis of two slip circles in landslide debris and a landslide on a crushed rock zone were analysed. The position of the crushed rock zone was inferred from the geologists subsurface interpretation of the interface between in situ rock and landslide debris at section T1. All analyses were carried out assuming a dry slope. Strength parameters of landslide debris (refer to section 3) were assumed for the two slip circles and the strength parameters for gouge were assumed for sliding on the basal failure surface. The results of the analysis are summarised in table 4.

TABLE 4: LIMITING EQUILIBRIUM ANALYSIS DESIGN SECTOR D

Slide Surface	Strength Parameters	Slope Condition	Factor of Safety
Circle 7, exits above the race	$c = 34$, $\phi' = 29°$	Dry	0.98
Circle 8, exits beneath the race, through tension cracks	$c = 34$, $\phi' = 29°$	Dry	1.04
Failure surface 3 Basal slide	$\phi' = 28°$	Dry	0.96

Circle 7 (figure 5) is the circle in the oversteepened face with the lowest calculated factor of safety. The circle starts 15 m up slope of the tension cracks, is generally about 10 m below the ground surface and exits from the slope just above the existing aqueduct.

Circle 8 (figure 5) is the circle in the oversteepened face passing through the tension crack with the lowest calculated factor of safety. The circle has a similar geometry to circle 7 but exits from the slope below the existing aqueduct.

The stability of the entire landslide debris slope slipping on a layer of crushed rock at the interface between landslide debris and in situ rock was assessed by analysing failure surface 3 (figure 5). The failure surface analysed was prepared by drawing three straight lines through the wavy surface interpreted from the seismic results. No attempt was made to model the waviness in the

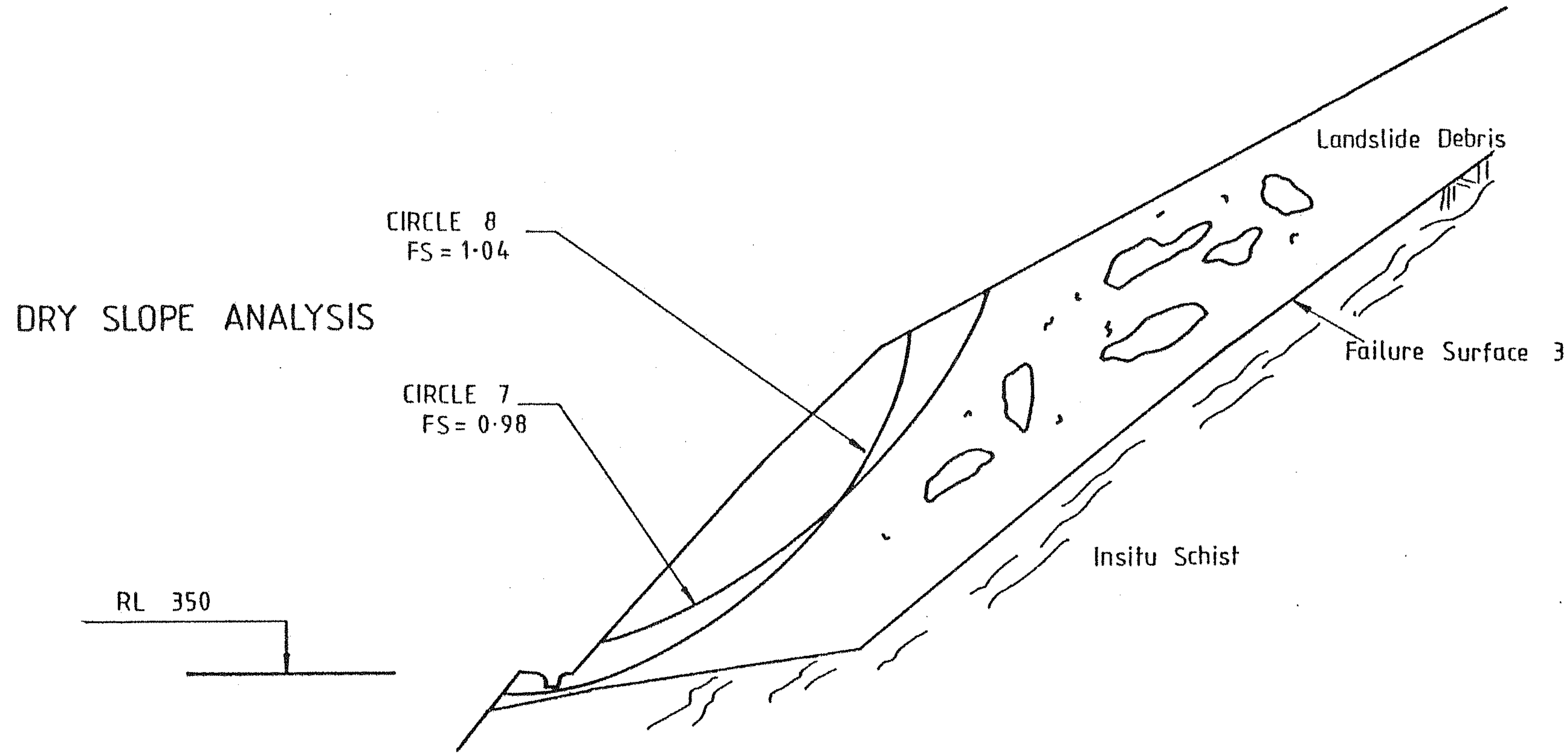


FIGURE 5 CROSS SECTION AT T4
Scale 1:1000

crushed rock surface because accurate information about the amplitude of asperities is not available. The calculated factor of safety (FS = 0.96) is less than one. There is no evidence of recent rapid large scale movements although creep movements may be occurring.

The probability of a rapid landslide movement on the crushed rock zone is low. Slow creep movements along the basal surface are probably currently occurring. However an excavation of the oversteepened schist slope at the toe of the landslide or any dramatic increase in the groundwater table could result in an increase in the rate of movement of the basal landslide.

The geotechnical assessment of the geological evidence and the stability analysis of the oversteepened landslide debris follows. Two slip circles were analysed, circle 7 has the lowest calculated factor of safety, but circle 8 passes through the tension cracks. The geometry of the two circles is very similar. There is direct evidence of incipient failure in the dry oversteepened landslide debris slope. The landslide debris is probably failing on a surface that begins at the tension cracks is generally about 10 m deep and exits either just above or just below the aqueduct. The tension crack has been developing for at least 12 months, however the rate of movement is likely to accelerate, particularly during heavy rainfall. The analysis and the geological evidence indicates that this landslide will progress headwards, ie if the landslide was to slip off the face on circle 7 or 8, it would remove support to the freshly exposed oversteepened face. Conditions would then exist for a similar landslide on the freshly exposed face. Landslides such as circle 7 or 8 remove support from the toe of the basal landslide. This may cause an acceleration in the rate of movement of the basal landslide.

4.6 Design Sector E RS 900 to RS 1088

Design sector E extends from the boundary between the in situ rock and the landslide debris in the north and the No 2 tunnel inlet portal in the south (plate 16). In this design sector, in situ schist rock bluffs rise steeply above the aqueduct.

Following a major rock fall in September 1983 MWD Alexandra Residency carried out a rock scaling operation on these bluffs. This operation was successful in improving the short term stability of the face. The joints in schist rock are open (see plate 1) and weathering and erosion of the joint infill material will result in further relaxation of the face.

Conditions for a significant rock fall currently exist, but a major triggering event such as heavy rainfall, a severe freezing thawing cycle or an earthquake, would be required. As the weathering and erosion of joint infill material proceeds, the risk of a significant rock fall will increase.

Plate 16. Aerial Oblique of Design Sector E

4.7 Summary of Stability Problems in the Gorge

The stability problems affecting the existing race are summarised in table 6. The annual probability of occurrence during the irrigation season is based on a subjective assessment of the calculated factor of safety of the slip circles and the basal slides.

The assessed outage is based on the following:

- a subjective assessment of the damage to the race;
- the time required to establish plant and manpower for major repairs;
- an allowance for access in poor weather;
- an assumption that the access road is to be upgraded to recommended option 1a described in appendix A of this report;
- an assumption that materials for repairs are stockpiled on the site;
- an assumption that plant and manpower is immediately available; and
- an assumption that temporary repairs are acceptable after a major slide.

The table was prepared to help the civil designers to compare reliability of various options. It is not intended as an accurate prediction of future rates of movement. Rates of movement may slow down during dry periods and may accelerate during wet periods or periods of seismic activity.

Active landslides can be observed in the Ophir Gorge. Observation and analysis indicate that rates of movement are likely to accelerate in the future. Shallow sliding from the oversteepened face increases the risk of future deep seated movement. Construction activities in the gorge will probably result in an increase in the rate of landsliding.

TABLE 5: SUMMARY OF STABILITY PROBLEMS EFFECTING THE EXISTING AQUEDUCT

Design Sector/Event	Implications	Assessed Outage	Annual Probability of Occurrence in Season
Design Sector A 1 Shallow colluvium slides	Filling or covering the race.	2 days	0.05
Design Sector B Slides in landslide debris: 1 Exiting above the race 2 Exiting below the race 3 Mass movement	 Filling or covering the aqueduct. Breaking the aqueduct. Slow movements, off-season repairs will prevent outage.	 3 days 7 days -	 0.10 0.05 -
Design Sector C debris: 1 Surface "unravelling" 2 Slide exiting above race 3 Slide exiting beneath the race 4 Mass movement	 Nil Small. Breaking aqueduct, repair difficult. Breaking aqueduct, repair difficult.	 - - 20 days (temp. repair) 20 days	 - - 0.3 0.07
Design Sector D Slides in landslide debris: 1 Exiting above the race 2 Exiting below the race 3 Mass movement	 Filling or covering the aqueduct. Breaking the aqueduct. Breaking the aqueduct.	 5 days 14 days 20 days	 0.2 0.2 0.02
Design Sector E 1 Rockfall	Fill the race.	7 days	0.05

5. GEOTECHNICAL ASSESSMENT OF THE VARIOUS OPHIR GORGE AQUEDUCTS UPGRADING OPTIONS

5.1 General

The Stage II geotechnical studies were not aimed primarily at option specific questions. However, in this section general comments about the various aqueduct options are made. A detailed description of the various options is contained in the district office feasibility report and the geotechnical report concentrates on the geotechnical aspects of the options.

Six options were discussed in the interim feasibility report; two pipeline options, three tunnel options and a weir and pump lift option. Within each option there are a number of sub-options, eg, if the weir were to be placed further upstream, pumping costs would be reduced, but this saving would be offset by increased pipe costs and increased risk. A detailed discussion of sub-options is not warranted at this stage because investigations were not option specific and because estimates are to be prepared to ROC standard only.

5.2 Option 1: Right Bank Pipeline on Existing Alignment

5.2.1 General

At the end of the Stage I studies it was concluded that option 1, upgrading the existing pipeline, was the cheapest option costed but that this option did not provide an assured water supply. In this section, option 1 is reviewed in light of the revised outage table (table 1) and Stage II geological and geotechnical studies.

The alignment of the pipeline follows the alignment of the existing aqueduct between the No 1 and No 2 tunnels. Where possible, it is proposed to carry the water in a closed concrete pipe placed in the existing open race. Through the section of the gorge where slope stability problems are serious, it is proposed to carry the water in a steel pipe supported on a pipe bridge or in an inverted siphon. The geotechnical assessment of this option will be based on the stability problems described in section 4 and summarised in table 5.

5.2.2 Design Sector A

Within design sector A there is a low probability of shallow colluvium slides filling the existing open race. A concrete pipe placed in the existing race would provide an assured water supply through design sector A.

5.2.3 Design Sector B

In design sector B it is proposed to place the pipe in the existing water race, except that from RS 215 to RS 360 it is proposed to construct an inverted siphon. The siphon is required for the following reasons:

- the existing race is buried between RS 226 to RS 274 and from RS 330 to RS 350;
- there is direct evidence of distress in the rock fill supporting the race, eg, plates 6 and 7;
- the tight radii in the existing race, eg, at RS 315, plate 7, would make pipe laying difficult.

The sections of the pipe placed in the open race are at risk from three types of landslide.

Slow mass movements of the central ridge are unlikely to result in outages during the irrigation season, if off-season maintenance is carried out to accommodate the movements at the pipe joints. This maintenance would be extremely difficult if the pipeline was buried, therefore in design sector B the pipeline should be placed uncovered in the open race.

Two classes of slide in the debris slope are possible, a shallow slide exiting above the existing race and a slide exiting below the race line. Slides exiting above the race line will not cause outages during the irrigation season but material covering the pipeline would need to be regularly removed. Slides exiting below the race will break the pipeline and cause a significant outage. The analysis indicates that there is a risk of slides up to 10 m deep exiting below the race. These slides could occur anywhere in design sector B.

The siphon can probably be placed on in situ schist rock, therefore debris slides and basal slides are unlikely to break the siphon. If option 1 is pursued, the position of basement rock along the siphon foundation should be determined during the Stage III investigations.

5.2.4 Design Sector C

It is proposed to carry the pipe on a pipe bridge from RS 425 to RS 650. The geological and geotechnical studies indicate three types of instability in design sector C: surface unravelling; slides in landslide debris; and mass movement landslides on the interface between landslide debris and in situ rock. Surface unravelling would not affect the integrity of the pipe bridge.

The pipe bridge should be designed to survive shallow slides in landslide debris without significant race outages. However, it is probably not economic to design the pipe bridge to survive, without damage, deep (greater than 10 m) slides in landslide debris.

The mass movement (basal) landslide would eventually result in significant outages. However, if the rate of movement continues to be a slow creep type movement, then the pipeline could be adjusted in the off-season to accommodate the movement. If the rate of movement accelerates then movements may cause serious and extensive damage to the pipeline and pipe bridge.

A ductile pipe bridge founded in the landslide debris would be the most suitable structural form for the pipe bridge. The pipeline should be designed to accommodate; slow, down slope, creep movements of the order of 100 mm per year. (This figure cannot be verified until adequate survey data is available.) The capital cost of this option should include an allowance for "spare parts" stockpiled on site to enable speedy repairs to the pipeline. The cost of maintaining the pipe bridge should be added to the capital cost when comparing option 1 to the alternatives.

5.2.5 Design Sector D

It is proposed to lay the pipe in the existing race from RS 650 to the No 2 tunnel portal. Because there is a high risk of rock falls damaging the pipe it is proposed to bury the pipe throughout design sector D.

Slides in landslide debris exiting above the pipeline will not affect the integrity of the pipeline. However, slides in landslide debris exiting below the race will break the pipeline. The geotechnical assessment of the various slides and a subjective assessment of the annual probability of occurrence is discussed in section 4.

The assessed outages for design sector D in table 6 include allowances for removing slope debris from above the pipeline. This operation would be necessary before digging up the pipe for repair.

Slow movements on the interface between landslide debris and in situ rock will cause distress and eventually break a buried pipeline.

5.2.6 Design Sector E

The steep slopes above the aqueduct in this design sector are mapped as in situ rock. The buried pipeline should survive rock falls without damage. The foundations of the existing aqueduct were recently covered by the fill material used to construct access to the rock slopes. As a result the geotechnical engineer has not seen the condition of the buried foundations of the existing race line in this design sector. However, "eye witness" accounts and the geological evidence indicate that the race foundations are likely to be satisfactory.

5.2.7 Summary of the Stability Problems Effecting Option 1

The stability problems, the assessed outages, and a subjective assessment of the probability of occurrence are summarised in table 7. In addition to the assumptions made in preparing table 6 (section 4.7) it was further assumed that it will be acceptable to make temporary repairs to the pipe bridge in the event of major damage.

The reliability of option 1 was compared to the reliability of the existing aqueduct and to the acceptable outage described in the brief. A graph (figure 6) of the "annual probability of occurrence" versus assessed outage has been prepared by "combining" the individual events. To combine probabilities, it was assumed that dry slope failures are independent but that landslides triggered by heavy rainfall are not independent. Figure 6 shows that the pipe bridge has a higher reliability than the existing race but that it does not meet the acceptable outage standards described in the brief.

The cost of repairing sections of the buried pipeline or pipe bridge damaged by landslide will be much greater than the current cost of maintaining the existing race. When determining the cost of maintaining the pipeline, civil designers should be aware that construction of the pipeline and the pipe bridge will cause an acceleration in the current rates of landsliding in the gorge.

Average assessments of the annual probability of occurrence for the various landslide events have been prepared. These deterministic assessments (table 6) were prepared to enable the designers to compare alternative options. A risk/economic analysis of various options should make allowance for the uncertainty in the deterministic assessment of annual probability of occurrence.

5.3 The Tunnel Options

5.3.1 General

Three tunnel options have been proposed: a short right bank tunnel avoiding the worst areas of instability (option 2); a long tunnel towards Chatto Creek valley avoiding the Ophir Gorge instability and access problems (option 3); and a left bank tunnel (option 5).

In order to compare the tunnel options to other alternative options, estimates of tunnelling costs in schist and schist derived materials were prepared. Excavation and temporary support unit rates are based on contract rates and own forces construction of investigation drives in schist rock and landslide debris. Lining costs were derived from water supply tunnels of similar size to those proposed for the Ophir Gorge.

Three classes of rock are defined for the current assessment of tunnel costs. For each class of rock, establishment and overhead costs have been determined from historical data for tunnels of similar length.

<u>Class I Rock</u>: Rock quality along the tunnel line is likely to be very favourable for hard rock tunnelling. The schist is blocky

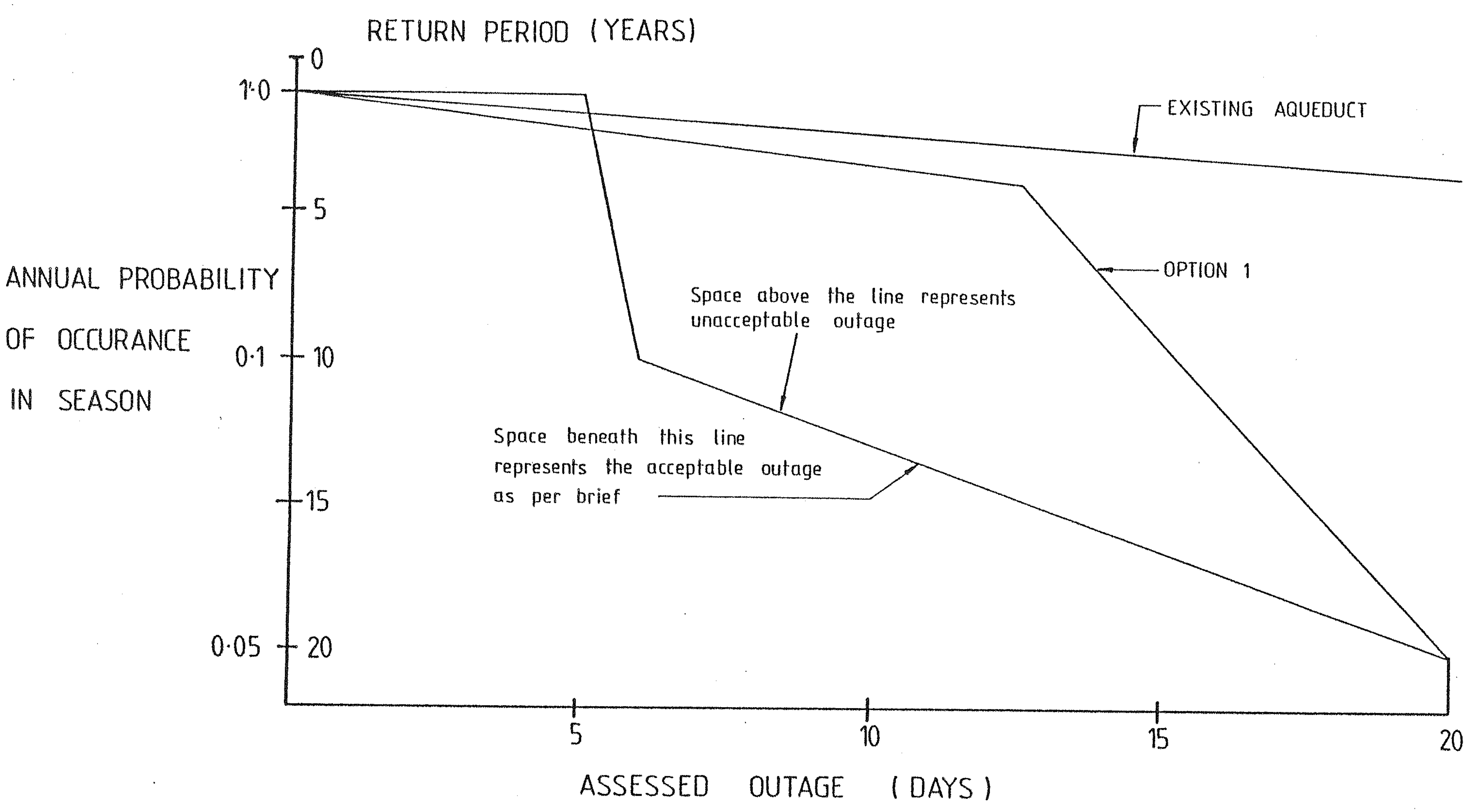


FIGURE 6 ANNUAL PROBABILITY OF OCCURANCE
ASSESSED OUTAGE RELATIONSHIP

TABLE 6: SUMMARY OF STABILITY PROBLEMS EFFECTING OPTION 1

Design Sector/Event	Aqueduct Description Implications	Assessed Outage	Annual Probability of Occurrence in Season
Design Sector A	Pipe in race line.		
1 Shallow colluvium slides	May cover race.	-	-
Design Sector B	Inverted siphon RS 215-360, otherwise pipe in race.		
Slides in landslide debris:			
1 Exiting above the race	High maintenance cost removing material from the race.	-	-
2 Exiting below the race	Breaking pipeline.	5 days	0.03
3 Movement of central ridge	Slow movements can be accommodated during off-season maintenance.	-	- Small
Design Sector C	Pipe carried on pipe bridge.		
1 Surface "unravelling"	No implications.	-	-
Slide on landslide debris:			
2 Less than 10 m deep	Minor local damage to pipe bridge.	3 days	0.15
3 Greater than 10 m deep	Extensive local damage to the pipe bridge. Reconstruction needed.	12 days*	0.15
4 Mass movement:			
(a) Slow	Readjust of pipeline.	-	-
(b) Fast	Reconstr of pipe bridge.	20 days	0.05
Design Sector D	Buried pipe in existing race.		
Slide in landslide debris:			
1 Exiting above the race	Nil.	-	-
2 Exiting below the race	Breaking the pipeline.	12 days+	0.2
3 Mass movement	Breaking the pipeline.	18 days+	0.02
Design Sector E	Buried pipe in existing race.		
Rockfalls	Nil.	-	-

* Temporary repair only. + Slope stabilising necessary.

with breakage being controlled mainly by joints, in general it is likely to perform at least as good as the best quality quartz-feldsphathic schist in the Paerau tunnel and as such should be mainly self-supporting.

ROC Unit Rates (CCI = 2300) for tunnelling in Class I rock are given below.

Excavation; including spot rock bolting, light support	$670/m
Overheads (including contingency, establishment, investigation, supervision, design and profit)	$670/m
UNIT RATE	$1,340/m

Class II Rock: Rock quality similar to class I except that rock is more relaxed, joints are open. This class of rock is expected near the portals. Excavation will require light support, but a permanent concrete lining will be required for a tunnel carrying water.

ROC Unit Rates (CCI = 2300)

Excavation (including support)	$670/m
Lining	$900/m
Overheads (including contingency, establishment, investigation, supervision, design and profit)	$1,570/m
UNIT RATE	$3,140/m

Class III Rock: This class of rock is used to classify landslide debris. Heavy temporary support will be required. Substantial overbreak is expected behind the tunnel design profile. Tunnelling conditions will be difficult, with possible water inflow, mixed face conditions and forepoling in places.

ROC Unit Rates (CCI = 2300)

Excavation (including primary support)	$1,000/m
Lining (including overbreak)	$1,800/m
Overheads (including contingency, establishment, investigation, supervision, design and profit)	$2,200/m
UNIT RATE	$5,000/m

Establishing a portal in chaotic debris will be both difficult and expensive. The ROC of a tunnel portal is presented as a separate cost.

ROC Tunnel Portal Landslide Debris:

Portal	$120,000
Overheads, etc.	100,000
	$220,000

5.3.2 Short Right Bank Tunnel: Option 2

The civil engineering designers have considered a short right bank tunnel avoiding the "worst" of the stability problems with an inlet portal at about RS 380 and an outlet portal at RS 680.

The geological and geotechnical studies show that this tunnel is not feasible. Deep seated slides in landslide debris and basal landslides slipping on the interface between in situ schist and landslide debris would regularly break the tunnel, causing long term outages.

It would be geotechnically feasible to drive a right bank tunnel from an inlet portal at or around RS 100 with an outlet portal at or around RS 950. The tunnel line would need to be 130 m into the slope face to completely avoid landslide debris and the weathering zone of the insitu schist. The resulting tunnel is 1 km long. Based on the current understanding of the geology, the rate is expected to be 60% in class I ground and 40% in class II ground.

5.3.3 Long Right Bank Tunnel: Option 3

A 1.8 km long tunnel driven from the Chatto Creek valley into the Ophir Gorge south of the desilter has been considered. Based on the currently available geological information, the rock quality along the tunnel line is likely to be very favourable for hard rock tunnelling. 85% class I rock and 15% class II rock is expected.

This option would provide an assured water supply and would result in a low annual maintenance cost. This option would obviate the need for upgraded access to the Ophir Gorge and would minimise the risk of construction in the gorge accelerating sliding in the Ophir Gorge.

5.3.4 Left Bank Tunnel: Option 5

A 750 m long tunnel has been considered for the left bank. The tunnel would start from a river crossing at approximately RS 100 and return to the right bank at approximately RS 950.

Based on the current understanding of the geology, the rock quality along this tunnel line is likely to be very favourable for hard rock tunnelling. 85% class I rock and 15% class II rock is expected.

The construction of a left bank aqueduct would require the construction of two river crossings. These structures would be founded on insitu schist rock. Preliminary geological investigations indicate, that at all possible crossing sites, insitu schist rock will be close to the ground surface.

The construction works may result in an acceleration of the rate of landsliding in the Ophir Gorge, because of the disturbance caused by construction traffic.

The left bank tunnel would provide an assured water supply and would result in a lower annual maintenance cost.

5.4 Left Bank Pipeline: Option 4

A left bank pipeline was investigated in the interim feasibility study. There is a large scree slope on the left bank. Construction of the pipeline across this scree slope would be difficult. Preliminary costing and geological studies indicate that this option is not feasible. No further investigation is recommended into this option.

5.5 Weir and Pump Lift: Option 6

To provide an assured water supply, the weir should be located at or about RS 950. Preliminary studies indicate a concrete weir is not economic but that a rock fill weir placed in the river is worthy of further study. This option would provide an assured water supply.

Preliminary geological assessments, indicate suitable foundations exist for a low level weir. In the Manuherikia river bed, insitu schist rock is overlaid by a thin (approximately 2 m thick) layer of river boulders. Water leakage through the foundation and abutment will not be a major problem for the type of structure proposed.

6. STAGE III SITE INVESTIGATIONS

6.1 General

The stage I and II site investigations have been aimed at establishing an understanding of the site geology and were not option specific, although the investigations have provided sufficient geotechnical background for the selection of a preferred option. After the preferred option has been selected further (Stage III) investigations would be required to determine the parameters needed for preparing the final plans, specifications and estimates. After the option to be carried through to the design stage has been chosen, a detailed investigation programme, brief and estimate can be prepared.

This section of the report makes general comments about the probable stage III investigation requirements for three of the options; the right bank pipeline; a tunnel option; and the weir and pump lift option.

The stage III investigations for the access roading have been discussed in appendix A.

6.2 Investigations for the Right Bank Pipeline

The general right bank geology has been determined during the stage I and II investigations. The aim of the stage III investigations will be to determine in detail the foundation conditions along the pipeline route. Two materials were identified, beneath the pipeline route, landslide debris and insitu schist rock. The strength parameters for these materials are described in section 3. Stage III investigations should aim to identify whether there is landslide debris or rock beneath the pipeline and where landslide debris is present, the investigation should determine the depth to insitu rock.

The stage III investigations for this option would probably include test pits and/or auger holes supplemented by seismic survey.

6.3 Investigations for Tunnel Options

This section contains general comments concerning stage III investigations for tunnel options through insitu schist rock in the Ophir Gorge. The aim of these investigations would be to determine the geological conditions at the tunnel portals and along the tunnel line. The information would be required;

(i) to choose a tunnel alignment,

(ii) to predict tunnelling conditions,

(iii) to design the tunnel profile and the tunnel lining,

(iv) to locate and design the tunnel portal, and

(v) to determine the cost of these operations.

The stage III investigations would be planned to study in detail the following features.

(i) A suitable tunnel portal location, chosen where the risk of soil or rock slides was minimised.

(ii) Stage III investigations should include an assessment of the depth and condition of the weathered and relaxed rock close to the ground surface.

(iii) A lineament has been mapped in the Ophir Gorge and crush and gouge zones were logged in the drill core from the drill holes of the right bank. The extent and implications of these features should be investigated in detail.

(iv) The rainfall in the Ophir Gorge is low but small aquifers perched on a crush or gouge zones could be expected. Groundwater conditions should be assessed during the stage III investigations.

(v) Tunnelling conditions should be assessed to determine a tunnel driving method, tunnel support requirements and hence a tunnel excavation cost.

Geological conditions dominate the design of tunnels. The feasibility stage assessment of tunnel conditions was based on an examination of the existing No 1 and No 2 Manuherikia Irrigation Scheme tunnels, air photo interpretation and an examination of rock exposures.

The Stage III investigations would include a more detailed investigation and recording of rock exposures on the ground surface and inside the existing tunnels and a more detailed examination of existing air photos and geological maps. In addition to these investigations subsurface investigations, drilling and seismic would be carried out along the tunnel line.

River Crossings

The left bank tunnel option requires two river crossings. The aim of the investigations for these structures would be to determine the foundation conditions. Preliminary investigations indicate that insitu rock would be close to the ground surface. Stage III investigations for the river crossings may include cored drillholes or shafts and perhaps seismic survey to determine the depth to insitu rock beneath the foundations of the river crossings.

6.4 Investigations for Pump and Weir Options

Preliminary study and costing has indicated that a rock fill weir would probably be appropriate for this site. Stage III investigations would aim to;

(i) determine the depth to suitable foundations for the weir and ancillary structures,

(ii) determine the permeability of the foundations and abutments, and

(iii) determine the availability of suitable construction materials.

To achieve these aims, the stage III investigations should include;

(a) backhoe trenches and perhaps seismic survey to investigate the depth to insitu rock at the abutments and foundations for the weir,

(b) detailed geological mapping of discontinuities and perhaps packer testing in boreholes to determine the permeability of the foundations and abutments, and

(c) test pits at the borrow areas to determine the availability of suitable construction materials.

The transmission line would need to be installed to supply electricity for the pumps. The stage III investigation should include a walk over survey by the geologist of the various transmission line options.

7. SUMMARY AND RECOMMENDATIONS

7.1 The Stage II Geotechnical Studies were aimed at establishing the right bank geology and building up an understanding of the mechanisms and extent of the slope instability on the right bank.

7.2 Field investigations included surface mapping, five logged boreholes and a seismic refraction survey.

7.3 The field investigations have indicated that the interface between landslide debris and insitu schist generally strikes parallel to the river and dips at 35^{o} to the horizontal. The subsurface investigations have indicated that the geology is more complex than anticipated. The sliding surfaces appear to be spoon shaped rather than planar and the landslide appears to be sliding on defect planes rather than foliation. The landslide debris is up to 40 m thick.

7.4 Strength properties determined for schist derived landslide debris from the Cromwell and Kawarau Gorges have been assumed for the schist derived landslide debris in the Ophir Gorge. These values have been used in the limiting equilibrium analysis of the Ophir Gorge slopes.

7.5 The results of the stability analysis are consistent with the surface and subsurface geological interpretation. The analyses indicate that there is a high risk of; shallow 'unravelling' failures, shallow landslides (approximately 5 m deep), deep landslides (approximately 20 m deep), and mass movement (basal) landslides. The analyses also indicate that shallow landslides reduce the factor of safety of deep landslides, this means that the future instability in the Ophir Gorge will be more serious.

7.6 The analysis shows that most landslide debris slopes on the right bank have a factor of safety close to one. Major earthworks or heavy traffic on the right bank is likely to trigger landslides.

7.7 The geological and geotechnical evidence suggests that there is a very high risk that the existing aqueduct will be seriously damaged some time within the next five years. Alternative aqueduct options must be considered.

7.8 A secondary objective of the Geotechnical Feasibility Study was to examine the alternative options desicrbied in the 'Manuherikia Irrigation Scheme Headworks Repairs Interim Feasibility Report'. Six options were discussed in the report. Three of these options warrant serious consideration and three schemes appear to be unsuitable for economic and/or geotechnical reasons.

(i) The right bank pipeline and short tunnel proposal is not a geotechnically feasible option; in addition the cost of constructing a tunnel on this difficult line is likely to be prohibitive. No further work is recommended on this option.

(ii) Further study of the left bank pipeline is not warranted. There is a very large scree slope on the left bank. Construction of a pipeline across this slope is not considered geotechnically or economically feasible.

(iii) A long tunnel towards the Chatto Creek valley has been proposed. This option is geotechnically feasible, but may be expensive to construct. Further investigations into this option will be warranted only if the estimated cost is comparable to the alternative options.

(iv) The cheapest option studied in the interim feasibility report was a right bank aqueduct on the existing alignment. The aqueduct will traverse the worst areas of instability either on a pipe bridge or in a siphon. The reliability of this option was compared to the acceptable outage table included in the brief. It was concluded that this option does not meet the reliability requirements of the acceptable outage table. It is recommended that the acceptable outage table be reassessed. It is recommended that before further work is carried out on this option the acceptability of lower scheme reliability is discussed with the client.

(v) Limited geological information is available for the left bank, however, no major geotechnical problems are expected with the proposed left bank tunnel. This option results in a reliable water supply and preliminary estimates indicate it may be economically viable. If this option is economically viable then further geotechnical investigations will be warranted.

(vi) A weir and pump lift has been proposed. This option is geotechnically feasible and results in a reliable water supply.

7.9 In section 6 of this report, general comments about the Stage III investigations were made. Detailed planning of the Stage III investigations may be carried out after the district office civil engineers have estimated the costs of each option, prepared a ranking of the options and indicated which options should be investigated during the Stage III investigation.

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APPENDIX A

APPENDIX A

ACCESS ROUTES TO OPHIR GORGE

A1. INTRODUCTION

The present access road to the Ophir Gorge has grades as steep as 1 in 3 in places and several tight corners. Access is therefore restricted to tracked vehicles and four wheel drive vehicles in good weather. The road is inadequate for any major construction work. An all weather access road to the gorge is needed for regular maintenance of the water race, to enable quick access in the event of disruption to water supplies, and would be essential for scheme refurbishment work.

Access to the gorge from downstream is physically restricted to the right bank, downstream of the end of tunnel 1. Three new access routes have been proposed and mapped out by Dunedin District Office (figure A1). Walk over surveys of the routes have been carried out by Head Office and District Office personnel and an engineering geologist from New Zealand Geological Survey. At this stage of investigations it has been assumed that access is needed throughout the Ophir Gorge from the desilter to the entrance of tunnel 2. All three proposed access routes therefore end at the presently existing platform just upstream of the entrance to tunnel 2 (figure A3). If the pumped weir option of scheme refurbishment is adopted, the destination of the access route may be different.

Access option 1 follows the old disused raceline from Chinky Gully along the river bank to the end of the present access track (figure A1). Option 2 uses the present access road to climb the hills, west of the gorge, then winds down along the side of the hills to meet the disused race about 100 m downstream of the entrance to tunnel 2. From here it follows the same route as Option 1. Option 3 uses the existing road to the top of the scarp above design sector D (see section 4 of this report). A cut is made through the scarp and the road traverses the top of the basal failure to the borrow pit (figure A6, plate A4). From here District Office originally proposed a cut section across the toe of the slip to the end of the present road. For geotechnical reasons an alternative to Option 3 was agreed upon after discussions between District Office and Head Office. This alternative, called Option 3A, is the same as option 3 down to the borrow pit and then uses the same route as the present access road. These options will be discussed in more detail in following sections.

The final choice of access route option will be based on the choice of option for upgrading the aqueduct, as well as on geotechnical and economic assessment.