Assessment of risk for road widened over unstable rock slope

L'évaluation des risques pour l'élargissement des routes sur des pentes rocheuses instables

I.J.A.BRACKLEY, Steffen, Robertson and Kirsten Inc., Johannesburg, South Africa
P.PHILOTHEOU, Scott and De Waal Inc., Cape Town, South Africa
P.J.TERBRUGGE, Steffen, Robertson and Kirsten Inc., Johannesburg, South Africa

SYNOPSIS : During the improvement of a road on the south coast of South Africa it was necessary to extend the carriageway over a rock slope with a history of instability. The road widening was achieved by the construction of half-viaducts supported on piers founded on bedrock.

The design of anchor support for the piers was carried out in two stages. First, existing failures were back-analysed, and the derived strength parameters were used in the analysis of the stability of the entire slope, with the production of a probabilistic hazard plan. Next, anchor support was designed, based on the principle that the piers with their anchors should apply a net restraining force to the slope, sufficient to ensure the stability of the individual piers, but not necessarily preventing slope failures.

1 INTRODUCTION

As part of the improvement of a National Road between the towns of George and Knysna on the coast of South Africa, a coastal section has been widened over a steep rock slope, which is known to be unstable. The widened carriageway has been supported on concrete piers and hollow cylindrical caissons. In designing these piers, it was necessary to assess the existing stability of the slope, and the effect of the additional loads on stability. Fig 1 shows a schematic section through the widened road.

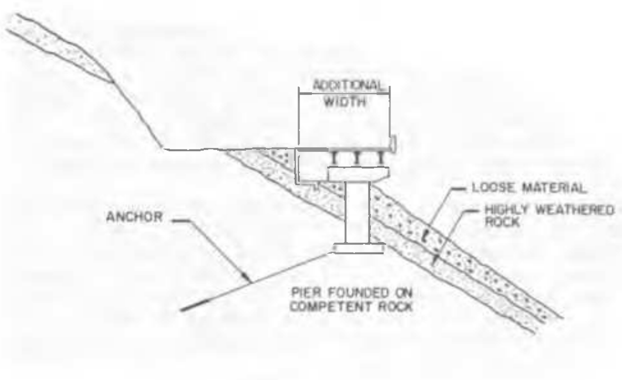


Fig 1 : Schematic section through widened road

2 THE ROAD

The problem section of the road is at the mouth of the Kaaimans River, and extends over a distance of approximately 1,5 km. The aerial photograph of Fig 2 shows the road, with a railway line running below it, near the shore-line. The bridge that can be seen is for the railway, and the road crosses the river further upstream.

The road is approximately 50 m above the shore, cut into the natural slope, which has an angle of 35 to 50° to the horizontal. It has been widened to a carriageway width of 14,8 m from 6,7 m.

Fig 2 : Aerial photograph of the road section

Several slope failures have occurred in the recent past, and an inspection of air photos has revealed additional old failures (Fig 3). It is probable that most, if not all, of these failures have been caused by the construction of the road or the railway, either because of the cutting-back of the slope or the changes in surface water drainage.

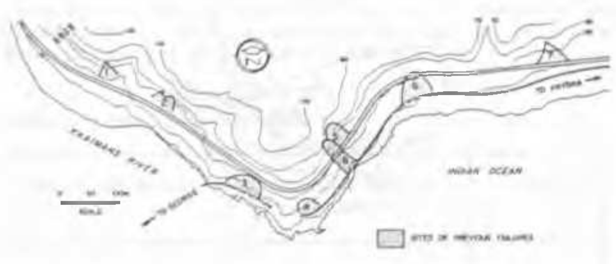


Fig 3 : Plan showing previous slope failures

3 SLOPE GEOLOGY

The local rocks are sediments of the late Precambrian age, which have been intruded by the younger Cape granites. Victoria Bay quartzites overly phyllites, which rest upon quartzites. The phyllites are the major cause of instability. (The local geology is shown on Fig 4).

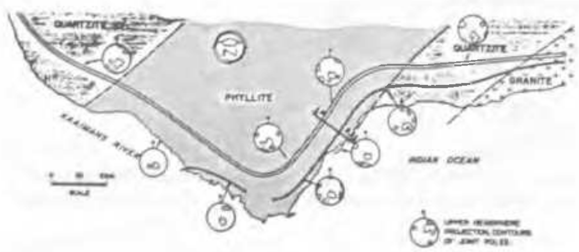
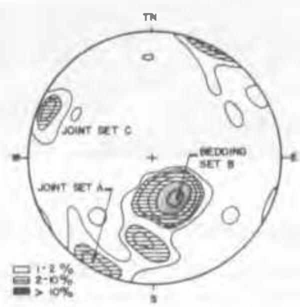


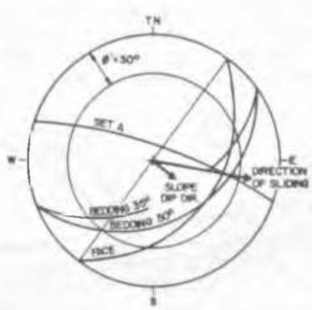
Fig 4 : Local geology

The main structural feature is folding. The sediments have been folded into an asymmetric anticline, with the north limb overturned. The south limb, which is exposed in the road and railway cuttings, dips in a southerly direction at between 25 and 45°.

There are consistent trends in the rock jointing, with bedding dipping at an average of 34° to the horizontal, in a dip direction of 151°, and two near-vertical joint sets dipping in directions 025 and 123°. (Fig 5).



(a) : Upper hemisphere projection, contours of joint poles



(b) : Lower hemisphere projection, intersection of great circles

Fig 5 : Rock joint data

The rock slope varies in geometry, but the surface is generally covered by a loose boulder layer in a soil matrix. Part of this zone is a natural scree, resting on loose material which has weathered in situ, but there is also debris from the original road cuttings. A typical section through the phyllite slope is shown in Fig 6.

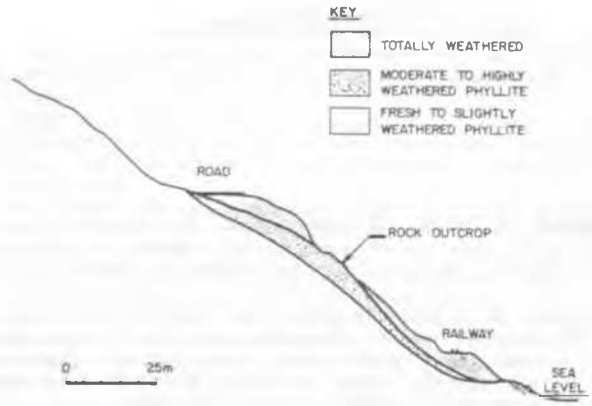


Fig 6 : Typical section through phyllite slope

4 THE SLOPE FAILURES

Of the eight failures (Fig 3), the two most significant are considered to be the recent failures 6 and 1. The most recent (6) occurred in 1981 after heavy rains, causing extensive damage to the road and the railway line. The slip took place through the weathered zone, leaving an exposed scoured surface of competent phyllite, and the road was reinstated by constructing a retaining wall directly upon this surface. Fig 7 shows a section through the failed slope.

Failure 2 was a recent, fairly large, steep-angled slip. Although there is evidence of failure through some intact material, the rock in this area is soft, micaceous and highly weathered. 200 m west is a recent, large three-plane wedge (1), also above the road. The plane along which failure has occurred is the bedding, with two major joint sets providing release surfaces.

An old and large failure surface exists between the road and the railway line (8). The surface itself is no longer exposed, but from the slope orientation and the present slope angle, it is likely to have been a bedding plane failure. Above the road, in the vicinity of this slip, a recent shallow slip on bedding is evident. A minor proportion of the failure appears to have passed through intact rock (5).

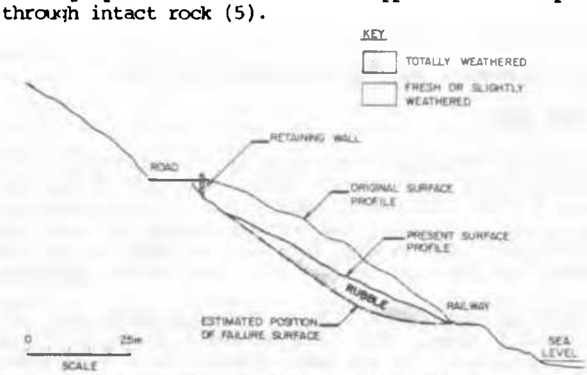


Fig 7 : Section through failure (6 on Fig 3).

An old, large, shallow-seated failure between the road and the railway line (3) gives evidence of recent small failures on the edges of the old surface. The dip direction of bedding relative to the slope is oblique, and failure is thus not considered to be bedding-related. Evidence from the recent failures and the shallow nature of the original surface suggests that failure occurred through soil and highly-weathered rock only.

A fairly small planar slip (7) has taken place in moderately to highly-weathered quartzite at the eastern end of the section.

Finally, above the entrance to the railway tunnel, a small shallow failure of highly-weathered rock has resulted from a localised steepening of the bedding dip angle (4).

5 THE OVERALL STABILITY OF THE SLOPE

The first step in the design of support for the widened road was the analysis of the stability of the slope below the road in its present state. Based on the observations of the existing failures, and the stratigraphy revealed by a programme of exploratory boreholes, three types of instability were considered possible - loose rock and soil sliding over the underlying bedrock, a wedge formed by the intersection of two or three joint sets sliding out of the bedrock, or a planar movement along the bedding.

From the examination of air photos it was estimated that from ten to fifteen per cent of the road section was within failure regions, and observations during the site investigation suggested a similar figure of about fifteen per cent. Although the slides passed through loose rock and the upper layers of jointed residual phyllite, they were certainly not superficial, extending as much as 10 m below the surface of the slope.

6 MATERIAL PROPERTIES

In order to calculate the risk of failure of the slope, it was necessary to define potential failure surfaces, to determine the strength of the slope material, and to predict the water pressures which might develop within the slope. Initially, joint strengths were estimated from joint measurements, using the method of Barton and Choubey (1977). The data was collected during joint surveys carried out on rock exposures and from rock core joint observations. A basic friction angle (the strength on a planar joint) of 21° was used and a curved strength envelope derived. Direct shear tests were also carried out in the laboratory on saw-cut joints in rock core samples. The results are shown on Fig 8, which includes the envelope used in the design of anchor support. An effective friction angle of 30° was adopted, with a cohesion of zero.

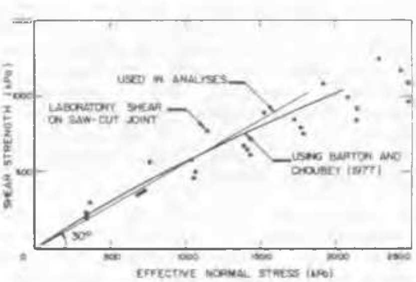


Fig 8 : Phyllite strength

Water pressures play a significant role in most slope failures, and it is probable that the failures on this slope were precipitated by an increase in pore-water pressures in the loose rock and along joints in the residual phyllite. The major failure of 1981 (6 in Fig 3) occurred after exceptionally heavy rains and, during the site investigation in August 1983 (in the dry winter season) it was noted that exposed rock cuttings were wet, with seepage on rock joints and bedding planes.

For the purpose of stability analyses, a pore pressure parameter of ru = 0,2 was used. This value was considered to be consistent with the observed conditions.

7 SLOPE STABILITY ANALYSIS

Three of the previous failures were studied (6, 2 and 3 on Fig 3). The surface topography was surveyed and the position of the failure surface estimated. The section for failure 6 is shown in Fig 7. The mode of failure was similar in all three cases, and was simulated using the Simplified Method of Janbu (1954) in a two-dimensional analysis. Strength parameters were varied until a factor of safety of unity was obtained. The pore pressure parameter was 0,2,

The strength parameters so derived were :

Location of Failure (Fig 3)	**Effective Cohesion (kPa)**	**Effective Friction angle**
6	0	35
3	15	38
2	10	38

8 PROBABILISTIC PREDICTION OF OVERALL STABILITY

Having produced estimates of strength properties from field joint mapping, laboratory tests and from back-analyses of previous failures, the next step was the prediction of the overall stability, and the preparation of a risk plan. Representative sections where no failures have occurred were selected for stability analyses. The sections were chosen after considering the site observations, the borehole logs and the geological sections drawn through the slope. From a knowledge of the local geology, likely failure surfaces on bedding were located. Strength parameters were selected, based on the geology and the back-analyses of the previous failures. Statistical distributions of these parameters were applied, taken from the variation of properties measured in the field and laboratory :

Effective cohesion	8,5 kPa	S.D = 6,2 kPa
Effective friction angle	35°	S.D = 3°

Probabilistic, two-dimensional stability analyses were carried out with the hazard defined in terms of the probability of failure as shown in Table 1, which also shows the equivalent factors of safety.

Table 1 : Hazard categories

Factor of Safety	Probability of Failure %	Hazard
1,4	0 to 1	Low
1,2 - 1,4	1 to 10	Moderate
1,2	Greater than 10	High

The results of the analyses are given in Table 2, for different assumed pore pressure parameters.

Using this data and taking cognisance of the following factors :

- slope geometry
- rock type
- joint orientation
- on-site observations (ie the frequency and magnitude of previous failures)

a plan was derived, delineating the different hazard zones below the road (Fig 9). It will be observed that the area of high risk is the phyllite slope, facing toward the south-east, in the same direction as the dip of the bedding joints.

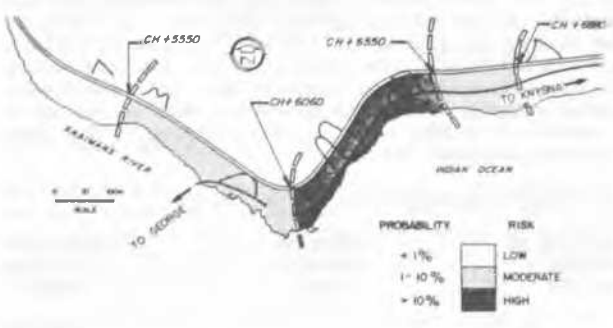


Fig 9 : Hazard plan

The five zones shown on Fig 9 are listed with the factors relevant to the determination of the hazard in Table 3.

9 STABILITY OF THE PIERS

The supporting piers and caissons were to be founded on competent rock, beneath the weathered zone. There were two inter-related questions to be answered :

Was it acceptable to widen and upgrade the road on a slope which was known to be unstable?

If the upgrading could be justified, what should be the extent of additional support measures to reduce the risk of a failure involving the road?

The first question could mainly be answered in economic terms. The problem section was a small part of the upgrading of 30 km of road, and the topography was such that there was no alternative route by which this slope could be avoided.

The second question was considered to have two possible responses :

- Support to be installed to reduce the probability of a slope failure to an acceptable value which, referring to Table 1, would be a value of less than 10 per cent.

- Support to be installed to ensure that the new loads applied by the widened road do not increase the existing probability of a slope failure, and the integrity of the individual piers is ensured.

The initial step in assessing the alternatives was an investigation of the type of failures which could occur to threaten the stability of the piers.

10 POTENTIAL SLOPE FAILURES INVOLVING THE PIERS

It has been stated that the piers would be founded on competent bedrock, below the loose rock and highly-weathered phyllite, implying that the common shallow failures would not pass beneath the base of a pier. Nevertheless, a slide or a movement of the superficial material could apply a load to the back of a pier. The force exerted by sliding loose rock was calculated assuming that it would form a wedge pressing against the back of the pier (Fig 10).

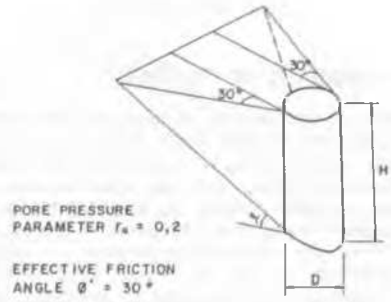


Fig 10 : Wedge of loose material against pier

A far more problematic situation would be that resulting from a deep-seated slide along joints in the competent bedrock below the base of the pier. In the phyllite on the south-west slope, the bedding joints have a mean dip direction of 151°, compared to the direction of the slope face of 215°, a difference of 64°. Therefore, a simple two-dimensional failure on bedding will not occur. Instability will be the result of sliding of rockspoil and highly-weathered phyllite over the competent bedrock, or slipping of a wedge formed by the intersection of a bedding plane with joint set A. (Fig 5).

On the south-east slope, where the dip direction of the slope is close to that of the bedding, failures through the bedrock approximating to two-dimensional failure are feasible, (which is, or course, the reason why this slope is the most unstable). The vertical joints will limit the size of the potential failure and its influence on the stability of individual piers.

A probabilistic analysis was carried out, for the various topographical and geological conditions along the slope, using a computer programme which creates a wedge by selecting joint inclinations from the statistical distributions of the sets, and analyses the resultant potential wedge failure. Often, the wedge is not geometrically possible, because the angle of the line of intersection of the joints is steeper than the slope angle or shallower than the inclination of the upper ground surface. However, when a possible wedge failure is created, the programme calculates the factor of safety against failure and, by analysing a large number of potential failures, the probability of a failure occurrence may be deduced.

Table 2 : Probability of Slope Failure

Section	Chainage	Slope Height m	Slope angle	Failure Plane Angle	Pore pressure parameter, ru	Probability of failure	Hazard Category
1	6390	37	27°	30°	0,0 0,1 0,2	0,3 2,0 9,5	Low Moderate Moderate
2	6246	33	40°	30°	0,0 0,1 0,2	0,5 7,5 22,8	Low Moderate High
3	6122	34	33°	33°	0,0 0,1 0,2	1,1 11,0 38,5	Moderate High High
4	6609	31	32°	35°	0,0 0,1 0,2	0,1 0,3 3,0	Low Low Moderate

Table 3 : Hazard Zones and Descriptions

Zone	Chainage (Approx)	Hazard Category for undrained slope, ie r_u = 0,2	Hazard Category for drained slope, ie r_u = 0,1	Slope Geometry	Rock Type	Joint Orientation	Observed frequency of deep seated failure
1	Pre 5550	Low risk	Low risk	F. shallow mod. slope	Quartzite	Favourable to stability	Very low
2	5550 -6060	Moderate risk	Low risk	High steep	Phyllite	Favourable	Moderate
3	6060 -6550	High risk	Moderate risk	High steep	Phyllite	Unfavourable	Very high
4	6550 -6880	Moderate risk	Low risk	F. High steep	Quartzite	Unfavourable	High
5	Post 6880	Low risk	Low risk	V. shallow mod. slope	Granite	Favourable	Very low

Notes :

1. Hazard categories defined in Table 1
2. Observed frequency of failure is related only to failures below the road

Fig 11 shows a large potential wedge failure involving two piers on the south-east slope. It was calculated that the largest wedge involving a single pier would require an anchor restraint of approximately 300 tonnes to ensure a factor of safety of 1,5 for the wedge as a whole. It was considered that the wedge would not move monolithically, instead dividing along the ubiquitous vertical joints. This implied that to be effective in preventing the development of a failure, anchor restraint would need to be applied down the slope, and not merely at the pier base itself. It also implied, on the other hand, that the piers themselves could be protected by a lesser restraint of approximately 120 tonnes applied at the position of the bases, if a failure downslope of the bases were permissible.

Various wedges involving one or two piers and two or three joint sets were analysed, with the required anchor force being in the range 100 to 200 tonnes. These forces were those required to retain the piers in position, but not necessarily to prevent slope failures. It was accepted that a failure could damage the road between piers, but that the design restraint would prevent the failure of the pier bases. This was considered to be the correct engineering solution from the economic and the ecological point of view. The application of large numbers of anchors on the face of the slope would have entailed the destruction of a large part of the indigenous vegetation, which would be unlikely to recover.

11 THE SELECTED DESIGN PROCEDURE

The piers were to be founded on competent bedrock, below the zone of loose rock and highly-weathered phyllite. The following design procedure was adopted for the pier anchors:

. The thickness of soil, loose rock, and closely-jointed, very weathered rock was determined.

. The force applied to the pier by a wedge of this superficial material was calculated.

. The new, additional disturbing force on a potential bedding failure plane was calculated by resolving the forces applied by the pier (dead and live load), and by the loose material.

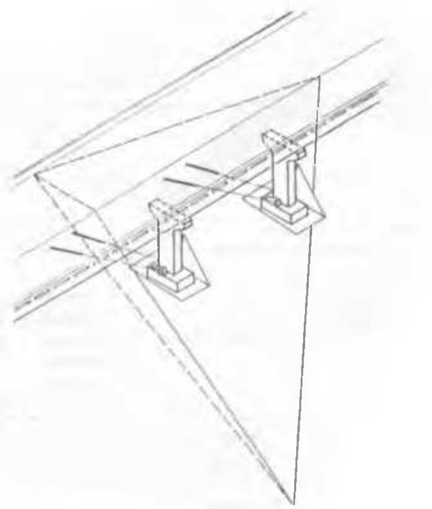


Fig 11 : Potential, large, deep-seated wedge failure, involving two piers

. Anchor forces and directions were selected to provide a **new** resisting force on the potential failure surface of 1,5 x the **new** disturbing force.

. The calculated anchor forces were compared to the anchor forces required to restrain the potential deep-seated wedges, to ensure that they were greater.

The proposed procedure is shown diagrammatically in Fig 12. From the examination of borehole logs, from a consideration of the geometry of potential wedge failures, and from the investigation of previous failures, a minimum required free length of 15 m was estimated for the anchors, to be confirmed by penetration rates. (Similarly, the founding depth for the piers was determined during excavation. At a number of pier positions, the anchor lengths and excavation depths were significantly greater than predicted).

12 DRAINAGE

It has been mentioned that increased pore water pressures probably played a part in the previous failures, and the effects of an increasing water pressure were noted in Table 2. Fig 13 shows graphically the relationship between the pore pressure parameter and the probability of slope failure. Using the admittedly arbitrary assertion that the installation of adequate drains in the slope will reduce the maximum pore pressure parameter from 0,2 to 0,1, it follows that a high

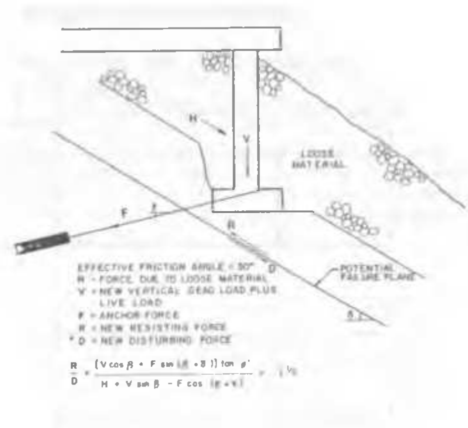


Fig 12 : Section showing anchor design procedure

probability of failure would reduce to an acceptable value of below 10%. However debatable the assertion, it is clear that effective surface and subsoil drainage will considerably reduce the risk failure, and design recommendations to this effect were made.

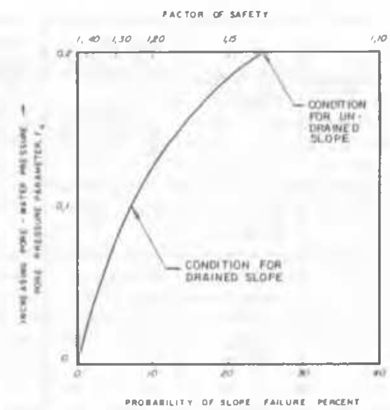


Fig 13 : Effect of water pressure within the slope on the probability of failure

13 CONCLUSIONS

The instability of the slope, indicated by known previous failures, was confirmed by the identification on airphotos of additional old failures. It is considered that the failures which have occurred since the construction of the road and railway were precipitated by the excavations for them, and by the changes in surface drainage. The majority of failures had occurred on steeply dipping bedding in phyllite, on a south-east facing slope.

The design criterion for the pier support on the unstable slope was that the new loads alone should apply a factor of safety of 1,5. In other words, when the **new additional** forces applied by the pier and its anchor restraint are resolved along a potential bedding failure plane, the **additional** restraining force should be 1,5 x the **additional** disturbing force. Further, the applied anchor force should be greater than that necessary to preserve the pier from destruction by a deep-seated wedge failure.

The implication of the adopted design method is that, although individual piers have been stabilised, there is still a moderate probability that slope failures will occur. These failures would not disturb the piers and caissons, but could damage the road between the supports, necessitating remedial works. The regular survey monitoring of the road will provide warning of instability, and allow timely action to be taken. This approach is considered to be preferable to an attempt to stabilise the entire slope.

14 REFERENCES

BARTON, N. and CHOUBEY, V (1977). The shear strength of rock joints in theory and practice. Rock Mechanics (Springer-Verlag) 10, p 54.

JANBU, N (1954) Application of composite slip surfaces for stability analyses. Proc. Eng. Conf. Stability of Earth Slopes, Stockholm, Vol 3, pp 43-9.

15 ACKNOWLEDGEMENTS

This paper is published by permission of the South African Department of Transport, to whom we offer our thanks.

A number of engineers were involved in the investigation. We wish to acknowledge the contributions of Dr O Steffen, Mr D Warwick, Mr T Rumpelt and Mr G O'Connell-Jones.

The initial drilling investigation was carried out by Messrs Mountain and Partners.