Stabilisation of a Mudstone Derived Colluvium Slope

G. RAMSAY
Design Engineer, Ministry of Works and Development, Wellington, N.Z.

SUMMARY

The design and effectiveness of drainage measures installed to stabilise a colluvium slope are reviewed in terms of observations made during construction. The difficulties in determining complex subsurface geometry and in situ permeabilities with reasonable predesign investigations are highlighted. The benefits of reassessment of the design on the on the basis of additional information obtained during construction are illustrated.

1 INTRODUCTION

Replacement of the Poro-o-Tarao railway tunnel in the centre of the North Island of New Zealand involved establishment of a portal in an area of known slope instability.

The geology in the area includes mudstones and siltstones of the Mahoenui Group, locally known as 'papa' (Borrie and Riddolls (1980)). Slope stability problems are common in the area, some being attributed to movement of colluvium, possibly formed by rotational slumps, on underlying intact mudstone. The New Zealand Railways have had a number of stability problems in the area. Aerial photographs of the area around the proposed portal show evidence of recent slope movement and local residents have reported slope failures in the general area over the last fifty years.

Investigations and design studies reported by Parton (1974) indicated that acceptable slope stability could be achieved by installing drainage measures to lower ground water and by constructing a portal structure to minimise slope excavation.

This paper discusses additional subsurface information obtained during construction, and the revised slope model deduced from this information. The effectiveness of the drainage measures is reviewed in terms of recorded piezometer readings and drain discharges.

Modifications to the original design slope contours were made following a reanalysis of the slope stability using the revised slope model.

2 NOTATION

- C' Effective cohesion (kPa)
- ϕ' Effective angle of internal friction
- γ Soil Weight Density (tonne/m^3)
- B Pore pressure coefficient
- α Angle of inclination of sliding surface
- H Depth of sliding block
- H_w Height of water table above failure surface

3 ORIGINAL DESIGN AND INVESTIGATION

Pre-design investigations involving 15 boreholes and seven 1 m diameter shafts indicated a stiff dark grey silt/clay colluvium containing mudstone fragments overlying hard grey mudstone bedrock. The interface was characterised in some cases by a zone of highly fractured mudstone with considerable water inflows. In some shafts a thin layer of highly plastic clay was observed between the mudstone and the colluvium. Interpretation of core from the small diameter (NX) holes was difficult due to drilling disturbance in the mudstone, and a clear picture of conditions was only obtained after the shafts were sunk and logged.

Design studies were based on the inferred colluvium-mudstone interface location and utilised effective residual shear strength parameters for the colluvium. Pore pressures measured by vibrating wire piezometers installed in the exploratory shafts were used. The analysis reported by Parton (1974) gave stability factors of safety in the range .9 - 1.1 for the known marginally stable preconstruction condition. It was shown that the only feasible method of achieving adequate factors of safety after excavation of the slope for the tunnel portal was to construct a portal approach structure and to install drainage to lower the water table slope to within 2 m of the interface, (as compared with the then existing 8 m level). The drainage was intended to lower ground water levels to an acceptable level prior to final slope excavation. The analysis showed a minimum improvement over existing factors of safety of 50% for static cases and 25% for seismic loading.

To dewater the slope a curtain of .6 m diameter sand drains, spaced at 2 centres and discharging into a 300 m long drainage drive located below the interface was proposed. The drain size and spacing were determined on the basis of permeability values calculated from recharge in the exploratory shafts. In addition to the subsurface drainage the design provided for reducing surface infiltration by the draining of surface swampy areas, lining of water courses and by afforestation.

Locations of the drainage drive, sand drains and portal structure are shown on Figure 1.

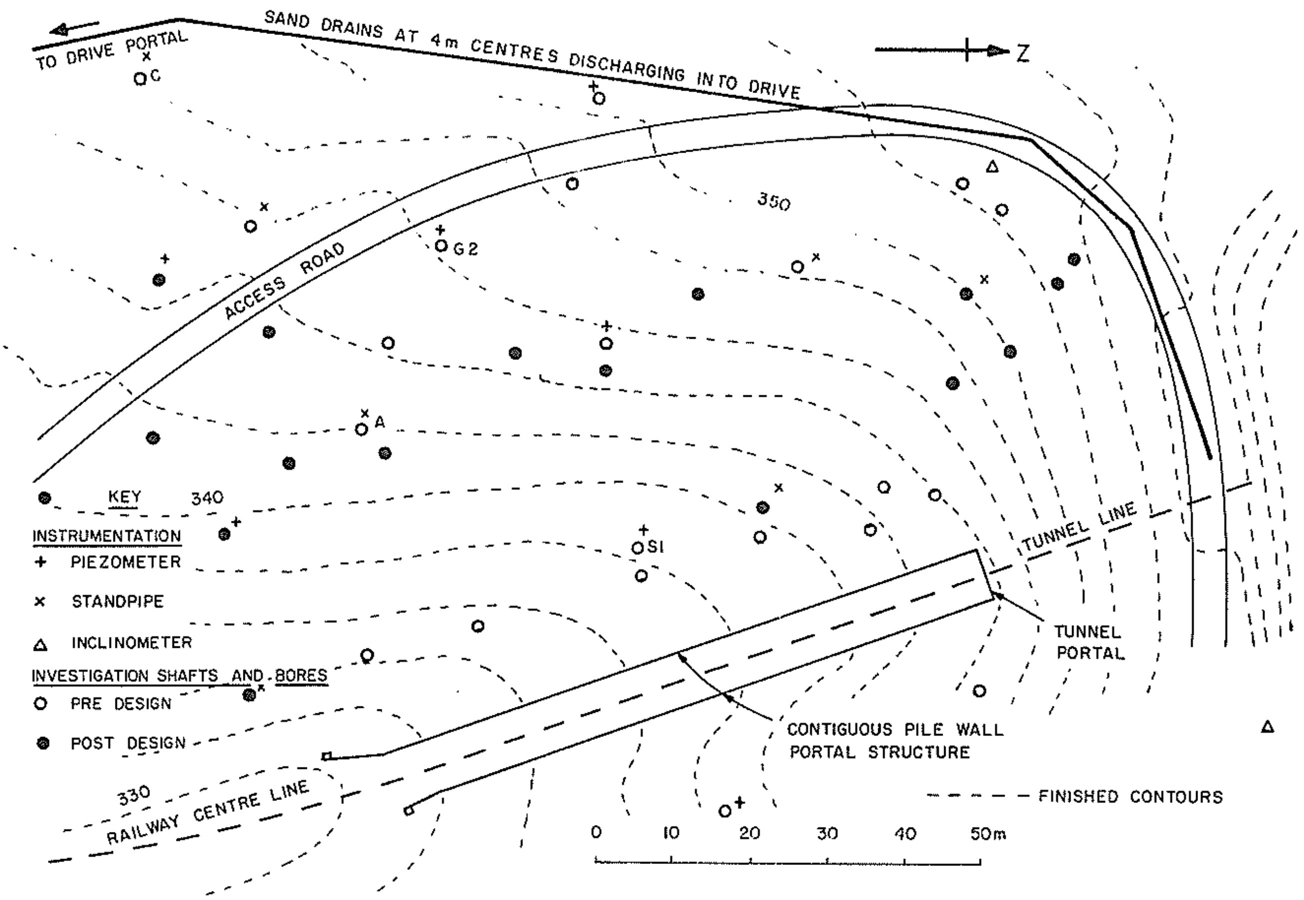

FIG. 1 LAYOUT OF SOUTH PORTAL PORO-O-TARAO TUNNEL

4 CONSTRUCTION SEQUENCE

The sequence of major construction activities is shown on Figure 2. Prior to 1976 minor works aimed at improving surface runoff were carried out. These included the diversion of streams crossing the slope into channels lined with a butyl rubber membrane overlain by concrete 'gobi blocks', and the draining and recontouring of some swampy areas.

To enable the drainage system to have the maximum effect on water levels prior to final excavation, the slope was excavated in two stages. The first stage excavation removed the minimum amount of material necessary to allow access for construction of the portal structure.

Because of delays incurred in constructing the drainage drive and sand drains, supplementary drainage measures, including inclined drains and pump shafts, were installed to accelerate slope dewatering.

The second stage excavation was delayed until the drainage measures had been installed and the analysis described in Section 7 had been made.

Minor surface slumps occurred during or shortly after both excavation stages. Remedial treatment of these slumps included removal of the slump material, local flattening of slopes and drainage by inclined under drains.

5 SITE INSTRUMENTATION

5.1 General

Instrumentation was installed in the portal area to perform three distinct functions:

(a) To provide piezometric data for back analysis of the known marginally stable preconstruction condition.

(b) To monitor the effect of the drainage measures installed.

(c) To detect any slope movements prior, during and after construction.

During the construction period additional piezometer stations were installed as the complexity of the subsurface conditions became apparent and addition stations appeared warranted.

The locations of the instrumentation stations are shown in Figure 1.

5.2 Piezometric Survey

In situ pore water pressures have been monitored by Geonor M600 vibrating wire piezometers and by conventional standpipes fitted with a porous tip located in a sealed length of borehole or shaft. Of the nine Geonor instruments installed two have been abandoned after reading faults developed. Tests carried out on Geonor piezometers held in stock indicated that the devices are temperature

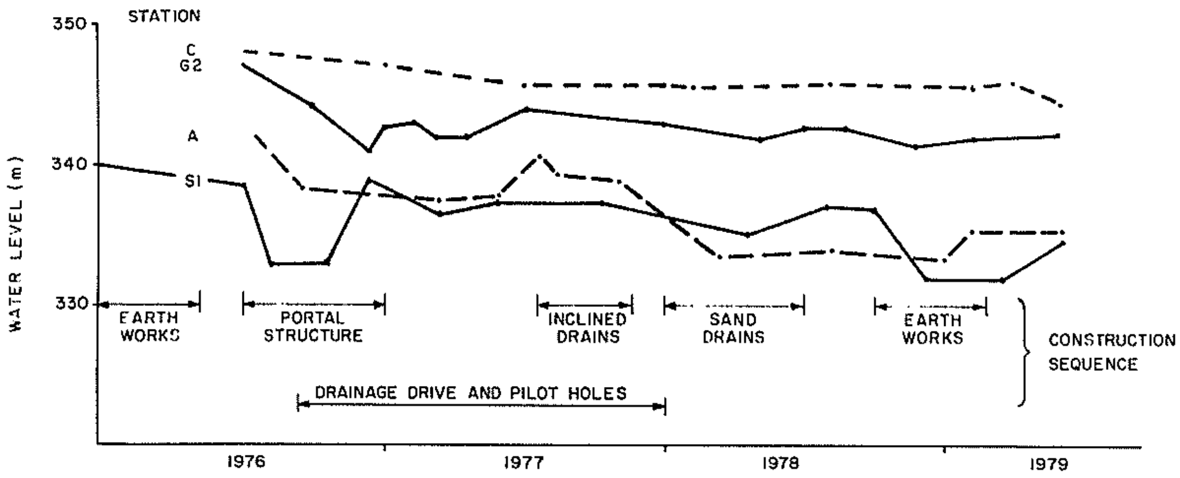

FIG. 2 SELECTED WATER LEVEL RECORDS

sensitive, the indicated pressure varying by .75 - 3 kPa per ^{0}C. The installed piezometers had been calibrated at 21^{0}C while measured groundwater temperatures ranged from 12 to 14^{0}C. As a result the accuracy of the absolute pressures indicated by the installed piezometers was questionable and conventional standpipes were installed adjacent to the piezometers to allow the latter to be field calibrated.

This emphasised the need for exhaustive pre-installation calibration of in situ measurement devices and the desirability of means of in situ recalibration. It is understood that Geonor piezometers are now supplied with temperature correction data and models permitting field calibrations are available.

5.3 Slope Movement Monitoring

Five borehole inclinometer tubes were installed in and adjacent to the area of construction activity. No significant movements have been detected from the inclinometer readings.

6 EFFECTIVENESS OF SLOPE DEWATERING

6.1 General

Daily rainfall records were kept and piezometers and drain discharges read at regular intervals. After major construction activities the frequency of readings was increased until readings had stabilised at new equilibrium values.

6.2 Piezometer Readings

The vibrating wire piezometers and standpipes tips are sealed into boreholes near the level of the colluvium/mudstone interface and record the ground water pressures at that level. Figure 2 shows the variation of recorded pressures at four stations selected from the fourteen installed.

Factors which were expected to influence the water pressures were the drainage measures installed, the unloading of the slope by excavation and seasonal rainfall variations. The time of occurrence of the major construction activities is shown in Figure 2.

Standpipe C which is located outside the area excavated has shown a steady but significant decrease in water pressure since installation. This reduction may result from the effects of surface drainage measures reducing rainfall infiltration.

Stations S1 and G2 located in an area where the interface forms a gully have shown apparent seasonal variations and major reductions related to construction activities and in particular slope excavation. The temporary drop of 6 metres in water level at station S1 in 1976 is associated with successful dewatering of the shafts for the portal structure by the use of adjacent pump shafts.

6.3 Drain Discharges

Discharges from individual included drains, vertical sand drains and pump shafts were monitored. In most cases the discharges increased significantly after periods of heavy rainfall. Figure 3 shows the variations of the total sand drain curtain discharge with time and Figure 4 gives individual sand drain discharge on two occasions.

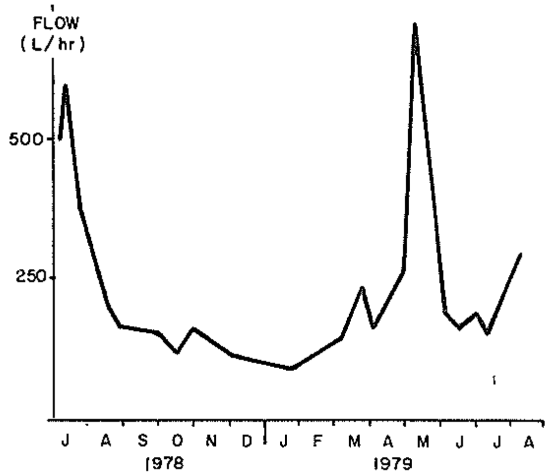

FIG.3 TOTAL DISCHARGE FROM SAND DRAINS

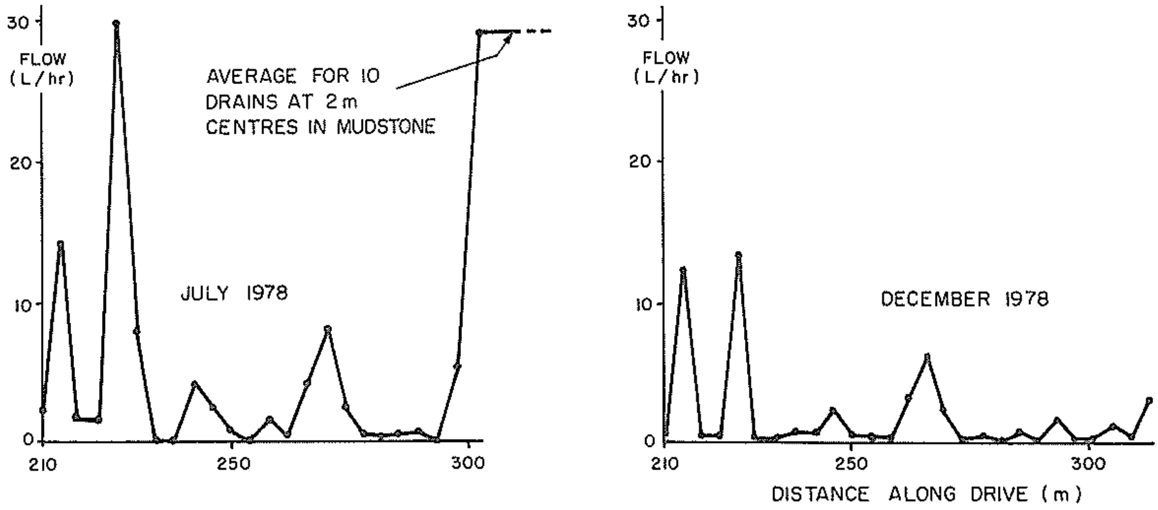

FIG. 4 INDIVIDUAL SAND DRAIN DISCHARGES

Discharges from inclined drains showed similar variations in discharge between closely spaced drains and similar peaks associated with periods of rain. Five pump shafts installed to accelerate slope dewatering were unsuccessful and required only periodic baling, being essentially dry holes.

6.4 Effectiveness of Drainage Measures

The spacing and size of the sand drains were determined using a coefficient of permeability obtained from analysis of recharge rates in exploratory shafts. The value obtained (4×10^{-6} m/sec) was considered as an average between the high "permeability" of the blocky zones observed in the mudstone and the low permeability of the colluvium (2.5×10^{-9} m/sec) as inferred from consolidometer tests and empirical grading/permeability relationships.

The variation in the sand drain discharges suggests that the blocky zones may be localised and that in most of the drains a considerably lower coefficient of permeability would be appropriate.

Observations suggest that the sand drains exhibiting the high flows providing the peaks in Figure 3 have very variable discharge. It is possible that these drains intersect blocky zones and are intercepting recharge water entering the slope through the high permeability blocky zones.

The base discharge from the drainage drive of approximately 100 litres/hour is possibly an indication of the water being removed by the sand drains acting in their designed function as draw down wells. The recorded base discharge is approximately one fifth of the design estimated discharge for the pre-construction water table.

Despite the fact that drainage measures are removing significant volumes of subsurface water, reductions in water pressure due to the major drainage curtain system are not obvious. That drainage measures can reduce in situ water pressures is evidenced by the temporary lowering of water levels at station S1. Similarly at standpipe A a permanent water level lowering of 2.2 metres occurred after an inclined drain passing close to the station discharged 62,000 litres in 3 months.

6.5 Conclusions

The dilemma faced in assessing the causes and permanence of recorded reductions in the water pressures revolves around whether the reductions result from the removal of water by the drains or from reduction in porewater pressure due to the slope excavation. It is likely that the time for the drainage measures to become effective and for unloading induced pore pressure reductions to dissipate will be similarly affected by variations in permeability.

It is possible that the localised drainage of blocky zones in the mudstone beneath and near the inferred failure surface has a significant though non quantifiable affect on water pressures at the interface. Thus the sand drains may be effective in reducing water pressures at the interface even though they may be relatively ineffective in their design function as draw down wells to dewater the colluvium.

The variation of permeability within the slope suggests that the use of published methods for computing the effects of inclined drains (Kenney et al (1977)) and other drainage measures (Mansur and Kaufmann (1962)) may be unsuitable. Such methods may lead to incorrect and possibly non conservative estimates of the time required for drainage measures to lower a water table.

7 REANALYSIS OF SLOPE

7.1 General

As construction proceeded a considerable amount of additional subsurface data was accumulated and, as intended by the designer, the design was reviewed in the light of this information.

7.2 Mudstone Colluvium Interface Profile

Excavation of the bored piles for the portal structure and drilling of the sand drains provided accurate interface levels at the top and bottom of the slope excavation area. In addition to the predesign investigation bores and shafts 24 small diameter bores and six 1 m shafts were drilled and logged during construction. The interface was also

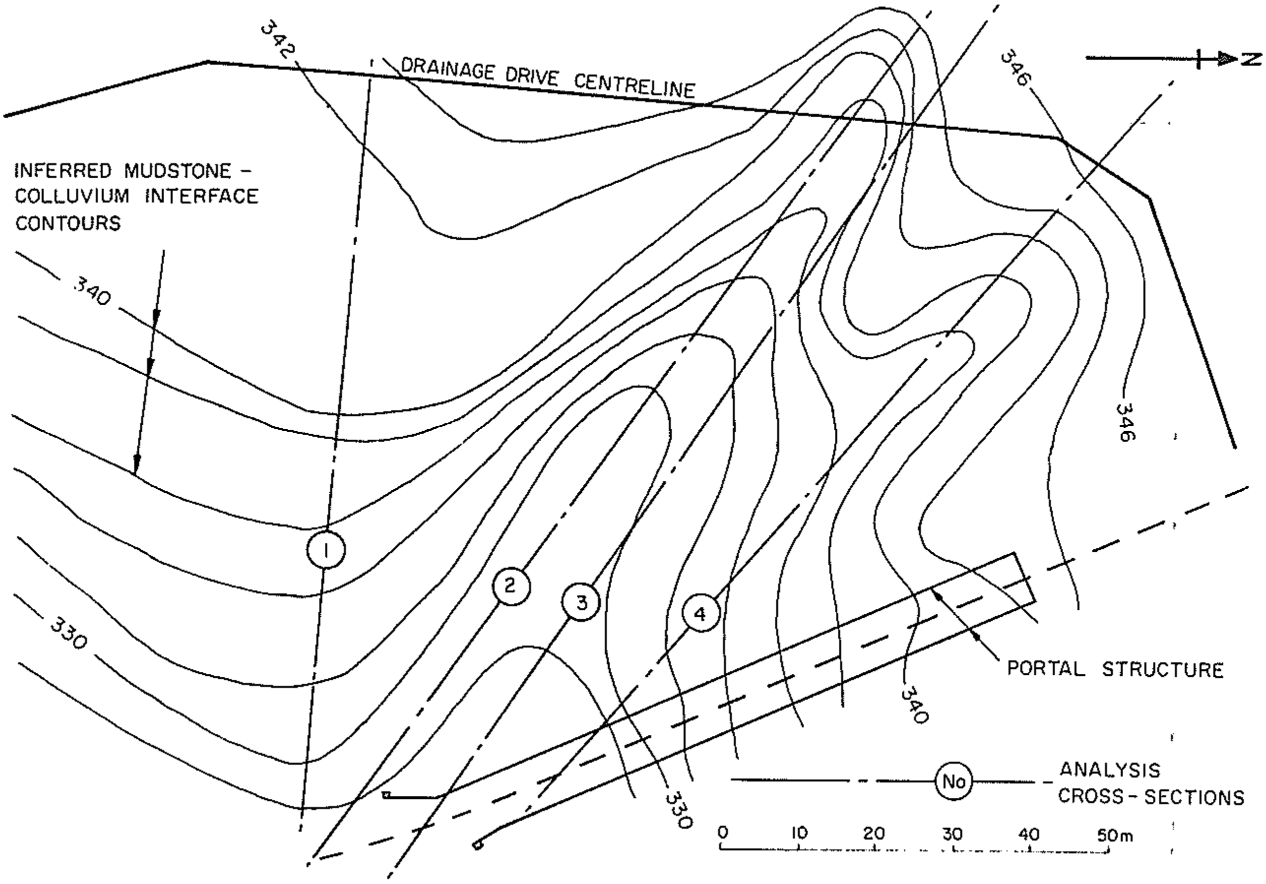

FIG. 5 CONTOURS ON MUDSTONE COLLUVIUM INTERFACE

surface exposed north of the portal structure during the first stage excavation. Figure 5 shows an inferred contour plan of the interface produced from data available at a late stage in construction. A total of 90 spot levels were available and of these 12 are not in agreement with the contours as drawn. It is possible that some of these may indicate further gullies in the interface which could only be confirmed by further extensive drilling. In other cases the discrepancies may result from difficulty in detecting the interface using disturbed NX core.

It is apparent that the interface is complex, gullied, and could not be realistically defined with reasonable pre-design investigations. In this case, the designer had 22 spot levels available of which 4 are possibly inaccurate and misleading for the reasons given above.

Further, design cross-sections derived on the assumption of a reasonably planar interface would provide an inaccurate model. The cross-sections analysed in the original design incorporated soil profiles from boreholes off set up to 30 m from the cross-sections. Some of the cross-sections did not show gullies in the interface apparent in Figure 5.

7.3 Piezometric Data

Even with piezometric data available at fourteen locations in the area it was difficult to obtain a clear model of the water table shape. It appears that the water table may be lower in areas above gullies in the interface, although the water pressure at the interface was higher in these areas due to the greater depth of the interface.

Just as definition of the complex interface geometry is not possible without a large number of spot levels, it is also likely that a considerable (and excessive) number of piezometer stations would be required to determine the distribution of water pressures in the slope.

7.4 Reasons for Full Reanalysis

The colluvium interface geometry discussed in sub section 6.2 was significantly different from that assumed at the time of design. It therefore did not appear appropriate to simply repeat the analysis of the design cross-sections with the as measured water pressures.

To assess the likely effects of the changed interface model a crude sensitivity analysis was made using an infinite slope model.

The effects of varying the inclination of the sliding surface and the ratio of the height of the water table and the height of soil above the sliding surface were examined. The results are presented in Figure 6.

The revised interface model had a locally steeper interface and locally reduced colluvium thickness compared with the original design model. Examination

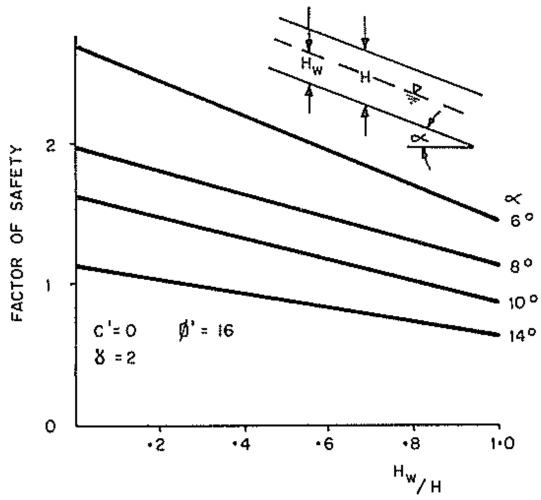

FIG.6 STABILITY OF INFINITE SLOPE

of Figure 6 suggested that if the original design slope profile was retained the stability factors of safety might be unacceptably low. Accordingly a full reanalysis was undertaken.

7.5 Reanalysis

Seven cross-sections were examined for short and long term stability. Non circular failure surfaces following the interface were examined using the method of Janbu (1973) and a computer program with a search routine was used to determine critical circular failure surfaces within the colluvium. The locations of four typical cross-sections are shown on Figure 5.

For the short term case, the measured piezometric pressures were used, with residual shear strength parameters ($\phi' = 16^0$) being assumed within 1 m of the presheared colluvium interface and peak strength parameters ($\phi' = 33^0$) elsewhere. For the long term case a water table 2 m above the interface was assumed with residual strength parameters throughout the colluvium.

The results of the analysis indicated that the final factors of safety for the as designed slope would be of the order of 1.2 - 1.3 on critical sections compared with 1.5 as calculated on the basis of the information available to the original designer.

LINE	CONTOURS	FAILURE MODE	WATER TABLE	F.S
1	As designed	Circular	Existing	1.1
2	As designed	Circular	Final	1.3
3	Revised	Circular	Existing	2.3
4	Revised	Circular	Final	1.5
5	As designed	Non circular	Existing	.95
6	As designed	Non circular	Final	1.3
7	Revised	Non circular	Existing	1.5
8	Revised	Non circular	Final	1.6

Table 1 : Factors of Safety for Cross-section 3

Analysis showed that extending the portal structure by 15 m and thereby reducing the slope excavation above the portal structure resulted in more acceptable factors of safety. Results for analysis on cross-section 3 (shown on Figure 3) are presented in Table 1, the critical failure surfaces for the as designed and proposed modified surface contours being shown on Figure 7.

Table 1 indicates that with the as measured water table a slope cut to the as designed contours would fail. However in practice the pore water pressures in the slope would be reduced as a result of the unloading associated with slope excavation. For a Skempton (1954) pore pressure coefficient B of 1.0 the immediate post excavation pore pressures would be approximately equivalent to the assumed long term lowered water table and thus the immediate stability would approximate the long term stability.

The circular failure surfaces were analysed to confirm that the non circular failure surface following the interface was the critical case. For the short term (existing water table) cases the circular surfaces are non critical. However for the long term (final water table) cases the circular surfaces are shown as being critical. Because these circular surfaces do not follow a pre-existing failure surface the use of residual strength parameters is unwarranted (Morgenstern (1977)) and the values in Table 1 are overconservative.

As shown in Figure 7 the critical circular failure surfaces are different for the short and long term cases.

7.6 Three Dimensional Stability Analysis

The use of two dimensional (2D) analysis for an apparent three dimensional (3D) situation appeared questionable. While some solutions for 3D failure surfaces have been published these did not suit the geometry or allow for the effects of water pressure. Therefore a solution for a trough shaped section failing in an infinite slope was derived. Normal forces on the side surfaces were derived by resolving vertical and at rest horizontal (Ko) stresses acting in the soil. The solution was expressed as a ratio of the factor of safety for the wedge (F_3) and the factor of safety for a 2D slice through the centre of the wedge (F_2).

Typical results for a cohesionless material are presented in Figure 8. For the case under consideration the variation between the 2D and 3D factor of safety is not significant.

These results vary from those of Hovland (1977) who considered wedge failures without considering the effects of at rest horizontal stresses and obtained considerably lower factors of safety for the 3D case. These may be appropriate for failure of a jointed rock mass but are in the author's opinion overconservative for an intact soil mass. Considerable increases in the factor of safety for the 3D wedge case are obtained for purely cohesive materials (Baligh and Azzouz (1975)).

7.7 Design Modifications

As a result of the reanalysis the portal structure was extended by 15 metres and the extent of the second stage excavation was reduced.

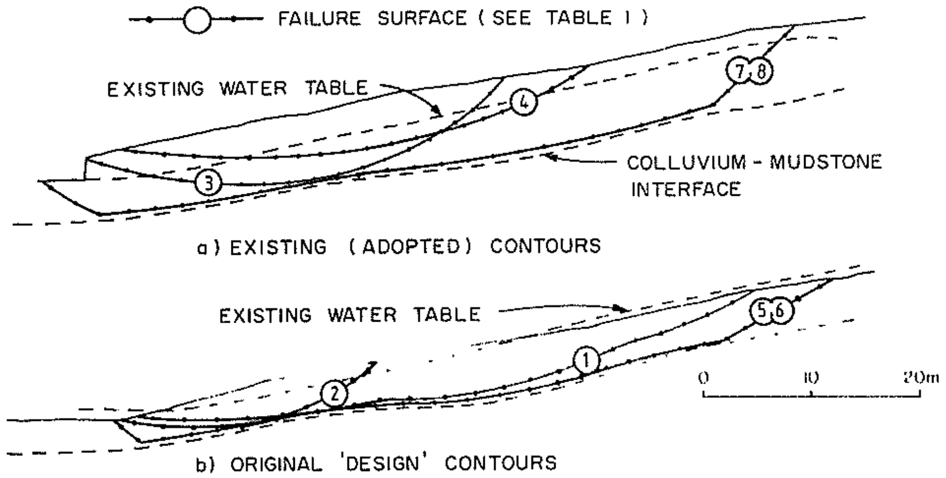

FIG. 7 CRITICAL SURFACES ON SECTION 3

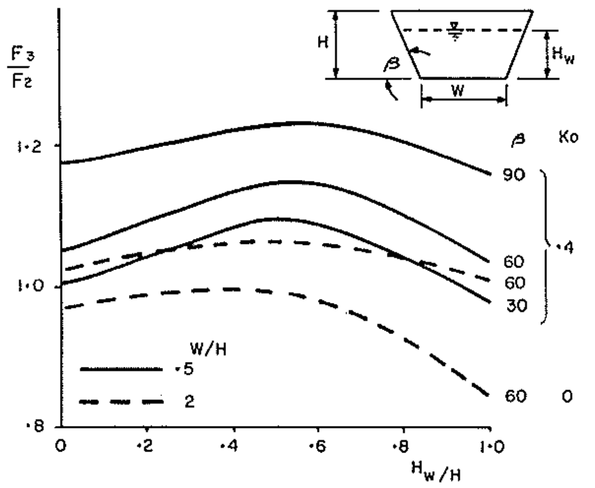

FIG. 8 RATIO OF F.S. FOR INFINITE SLOPE (F_2) AND F.S. FOR INFINITE WEDGE (F_3)

In addition a pattern of inclined drains are to be installed from within the portal structure. It is hoped that some of these will intersect blocky zones near the interface and in particular in interface gully areas where water pressures still appear to be high.

8 CONCLUSIONS

The case history reported illustrates the need in slope stabilisation exercises for the designer to review and where necessary modify the original design in the light of additional subsurface and instrumentation data gathered during construction.

Information gained from extensive drilling and shaft excavation carried out during construction suggests a complex gullied mudstone/colluvium interface which could not have been defined with reasonable pre-design investigations. Positive identification of the interface in small diameter (NX) core was difficult due to drilling disturbance in the mudstone. Down hole logging of 1 m shafts provided the only accurate method for locating the interface.

Piezometer readings suggest a complex ground water regime with depressions in the water table surface corresponding to gullies in the interface. While significant reductions in water levels occurred during construction, it was not possible to accurately assess to what extent these resulted from the influence of the drainage measures or from reductions in pore water pressure due to the unloading associated with slope excavation.

Discharges from closely spaced subsurface drains varied considerably and it is suggested that the drain discharge depends largely on whether or not a zone of blocky mudstone below the interface is intersected. It does not appear possible to predict or pre-determine the location of such zones. The colluvium, intact mudstone and blocky mudstone have significantly different permeabilities and the applicability of available solutions for computing drain discharge is doubtful due to the difficulty in assigning a suitable permeability value.

Reanalysis of the slope using the additional information collected during construction indicated a need for some modifications to details in the original design.

While the drainage measures adopted are removing significant quantities of water from the slope it may be some years before the stability of the slope becomes dependent on the effectiveness of the drainage measures. Long term maintenance and monitoring of the drainage measures is therefore essential.

9 ACKNOWLEDGEMENTS

The author acknowledges the permission of the Commissioner of Works to publish this paper, and wishes to thank I.M. Parton and G. Borrie for their useful comments.

10 REFERENCES

BALIGH, M.M. and AZZOUZ, A.S. (1975). End Effects on Stability of Cohesive Slopes, Proc A S C E, Vol. 101, No. GT11, pp 1105-1117.

BORRIE, G.W. and RIDDOLLS, B.W. (1980). Engineering Geological Investigations in Soft Rock Terrain, Poro-o-Tarao Tunnel, New Zealand. 3rd Aust. - NZ Geomechanics Conference, Wellington, New Zealand.

HOVLAND, H.J. (1977). Three Dimensional Slope Stability Analysis Method, Proc A S C E, Vol. 103, No. GT9, pp 971-986.

JANBU, N. (1973). Slope Stability Computations, Embankment Dam Engineering, Casagrande Volume, ed. by R.C. Hirschfield and S.J. Poulos.

DENNEY, T.C., PARZIN, M. and CHOI, W.S. (1977). Design of Horizontal Drains for Soil Slopes. Proc A S C E, Vol. 103, No. GT11, pp 1311-1323.

MANSUR, C.I. and KAUFMAN, R.I. (1962). Dewatering Chap. 3 of Foundation Engineering, ed. by Leonards. McGraw-Hill, publishers.

MORGENSTERN, N. (1977). Slopes and Excavations in Heavily Over Consolidated Clays. Proc 9 Int. Conf. Soil Mech. and Found. Eng., Tokyo, pp 567-581.

PARTON, I.M. (1974). Assessment of Slope Stability at Poro-o-Tarao Tunnel South Portal. Proc Symp. on Stability of Slopes in Natural Ground, NZ Geomechanics Soc., Nelson.

SKEMPTON, A.W. (1954). The Pore Pressure Coefficients A and B, Geotechnique, Vol. 4, pp 143-147.