JONES, Gary A. - Van Niekerk, Kleyn & Edwards, Consulting Engineers, South Africa.
SMITH, Andrew C.S. - Reef Lefebvre (Pty) Ltd., South Africa.

DESIGN, CONSTRUCTION AND MONITORING OF A REINFORCED EARTH WALL FOR RECONSTRUCTION OF A HIGHWAY SLOPE FAILURE.

DESSIN, CONSTRUCTION ET CONTROLE D'UN MUR DE TERRE ARMEE POUR LA RECONSTRUCTION D'UN VERSANT D'AUTOROUTE DEFECTUEUX.

RESUME.

Une section d'autoroute à double voies a été endommagée par un glissement à la suite de fortes pluies en Janvier 1978. La voie supérieure fut temporairement confortée par un gunitage et des ancrages au rocher. Après des études de stabilité, différentes techniques furent envisagées pour la réparation. Elles ont montré que la solution pouvait être fournie par la réalisation d'un mur en terre armée de 115 m de long sur 16,5 m de haut. Le mur actuellement construit est en cours d'expérimentation.

INTRODUCTION

During January 1978, after exceptional rain a section of split level dual carriageway highway on Road P39-1, near Krugersdorp, South Africa collapsed. At the failure the road was on a steep gradient as it climbs an escarpment. The road was constructed partly in cut and partly on fill, on a natural side slope of about 1 in 2. The failed section was about 80 m long and the lower carriageway dropped about 6 m. The failure zone extended about 150 m down the slope, at which point the slide debris formed a tongue approximately 200 m wide.

A view of the failure is shown in Figure 1 and a cross section through the centre of the slide is given in Figure 2.

In general the failure and rehabilitation resembles that at California Highway 39, which has been extensively documented by Chang and Forsyth[1]. The instrumentation installed at both sites is also similar. This paper describes the analysis and design of the remedial measures; it is the intention that the detailed results of the performance monitoring will be available for discussion at the conference when the paper is to be presented. No apology is made for the similarity to previous projects since it is the authors' belief that it is by well documented case histories that the art of geotechnical engineering is advanced.

GEOLOGY

The escarpment up which the road climbs consists of alternating beds of shales and quartzites of the Witwatersrand Series overlying low grade schists and phyllites of the much older Swaziland Series. In the exposed cutting the bedding planes dip at about 45° to the south west which, as shown in Figure 2, is apparently favourable for the cutting and embankment stability. Both shales and quartzites are heavily jointed with the predominant set being practically at the slope of the cutting. The underlying schists and phyllites are generally very weathered. It was expected that the dip apparent in the cutting would represent that through the failed area. However, on excavation for the reinforced earth wall, it became clear that the dip of the shale in the zone through which the failure was assumed to pass, changed radically over a width of only 10 m from about 45° south west in the exposed face to about 60° north east at the outside edge of the excavation. This is shown on Figure 2.

Generally natural drainage is away from the open cutting because the ground above the escarpment slopes with the bedding, to the south west. There are a few gulleys, probably at minor fault positions, intersecting the escarpment but these carry little surface water. The escarpment itself, some 150m above the lower lying ground to the north, appears to be subjected to localized weather conditions resulting in fairly intense rainfall in short duration storms. There is little evidence of significant seepage through the scarp face.

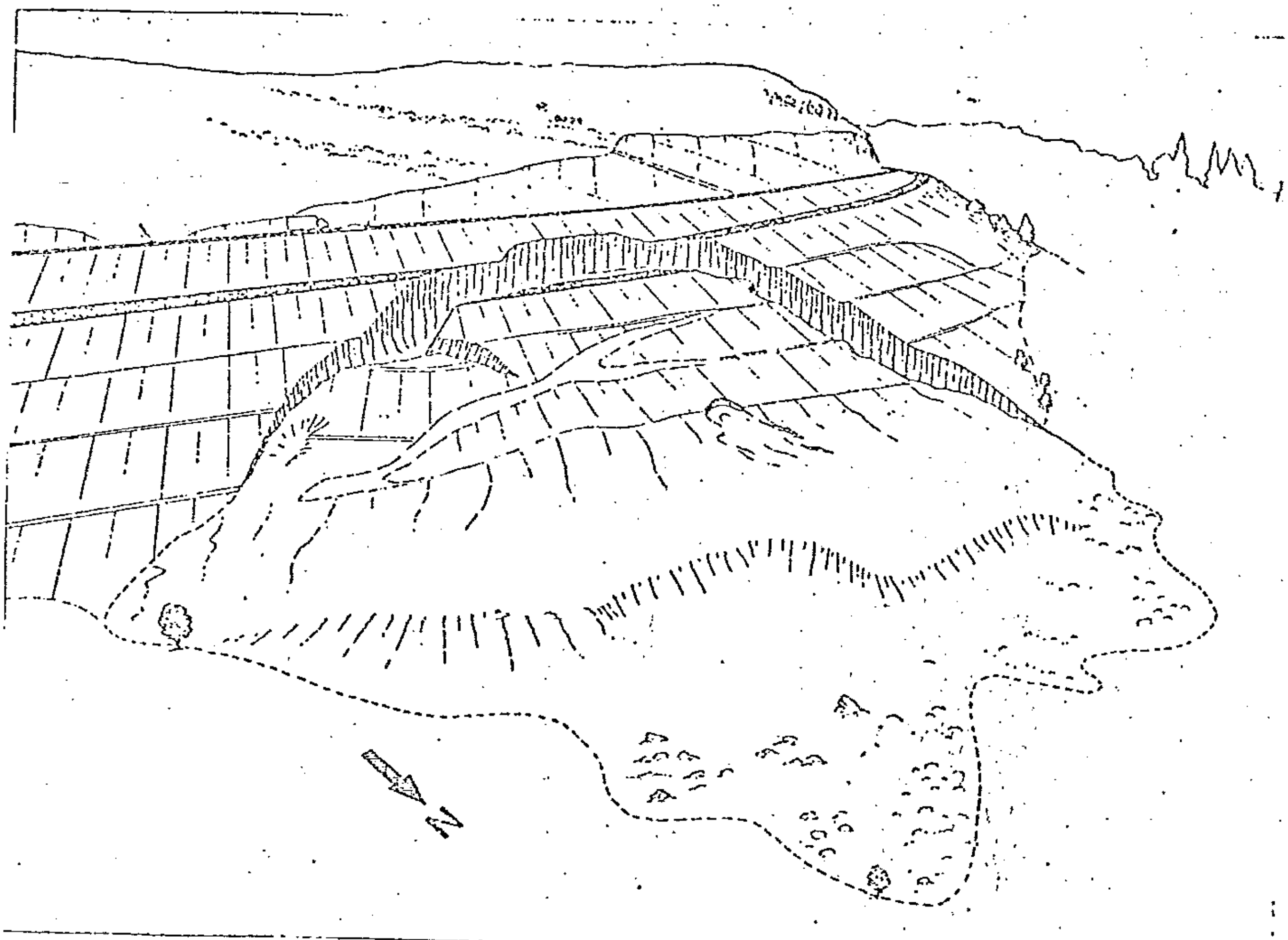

FIGURE 1. PERSPECTIVE OF SLIDE

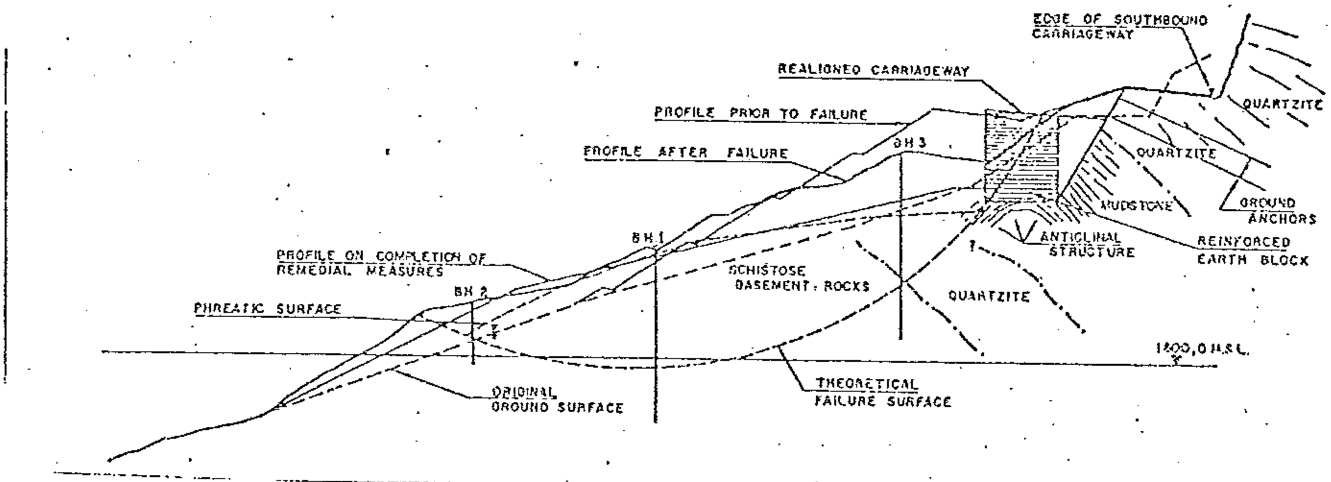

FIGURE 2. SECTION THROUGH FAILURE

STABILITY ANALYSIS

Conventional stability back analyses, using the circular arc Bishop Rigorous method, were carried out after the failure, with assumed soil and pore pressure parameters, to model the failure condition (F = 1). Fortunately it was possible to obtain a geometric model from the shape of the ground before and after failure. It was found that a deep circular arc failure surface was the only kinematically possible solution. This failure surface intersected shales, quartzites and the lower lying schistosed materials. Boreholes through the slide debris, combined with simple plastic lined deformation tell tale holes, tended to confirm the postulated failure arc position. This is shown on the cross section Figure 2. Since the geometry was known, the analysis became a straightforward variation of subsoil strength parameters with changes in assumed pore pressure so that F = 1. It was considered that measurements of actual shear strengths would be of doubtful value, since obtaining representative samples suitable for laboratory testing would be extremely difficult, particularly if the failure plane was thin.

In order to model failure it was necessary to utilize fairly high pore pressures, and although such pressures could exist in a jointed rock mass, the fact that no downslope seepage was observed, before or immediately after failure, suggests that pore pressures were probably not very high. During excavation for the remedial measures however the marked change in dip became apparent. Since failure along bedding planes in the shale then became a realistic possibility, as opposed to failure along joints, laboratory tests were carried out on the strength properties along the bedding planes of the shale. The tests gave values of $C' = 5$ kPa and $\phi' = 25°$ which were also assumed to apply to the schistosed material. These were used in the back analysis and allowed a reduction in the postulated pore pressures for the F = 1 condition. This phreatic surface is shown on Figure 2. The combination of shear strength and pore pressure was then used to examine the stability of the various proposed reconstruction schemes.

RECONSTRUCTION METHODS

Immediately after the failure there was considerable concern that the upper carriageway would also fail, particularly since large cracks parallel to the road appeared in the median. Five inclinometer casings were placed in drilled holes along the guard rail of the upper carriageway and these were checked daily. In the meantime the traffic was confined to the inner lane and shoulder nearest the cutting face, which caused severe congestion.

In order to support the upper carriageway, a ground anchor system consisting of two rows of 300 kN anchors was installed in the median, with 59 anchors spaced on a 3 m grid. The median was trimmed back and shotcreted to a nominal 50 mm thickness before the anchor installation. On stressing the anchors, the inclinometers showed some small movements, generally about the same amount as had been observed in the opposite direction when the median had been trimmed back from its original slope of 1 in 3, to a slope of 2 in 1. The layout of the anchor system is shown in Figures 2 and 3. After stressing the anchors, the full width of the carriageway was opened to traffic.

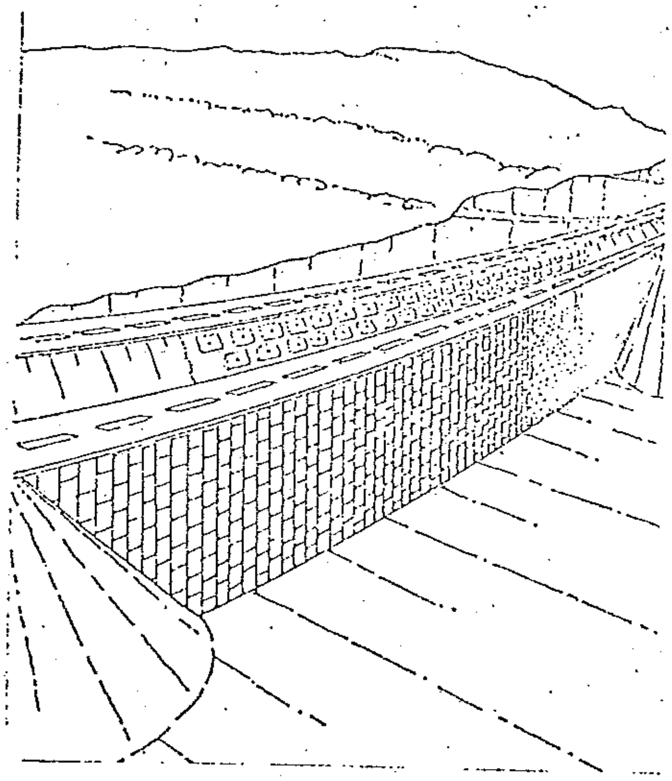

FIGURE 3. FINISHED WALL SHOWING ANCHORS

Various methods were considered for the reconstruction of the lower carriageway; these are described briefly below :

Realignment of carriageway

Due to the width of the median it was possible that the failed carriageway could be realigned; this in combination with one of the embankment replacement schemes, would considerably reduce the earthworks quantities.

Reconstruction of earthworks

Since the failure was deep seated, the removal and replacement would have entailed very large quantities, approximately 300 000 m³ costing about $2m. Also, if the original geometry was reinstated, then a deep drainage system would be necessary to ensure that pore pressures could not in the future reach the level which had caused failure. This scheme, even with the carriageway reealignment, would be expensive and rely on the proper functioning of the drainage for safety.

Retaining Wall

A conventional retaining wall would have to be about 20 m high since it would be necessary to found on the rock. Although this solution is feasible, particularly in combination with the realigned carriageway, a cost analysis showed it to be excessively expensive since the foundation costs were large. Various anchored wall solutions were considered but these also proved to be expensive.

Structural

A number of structural solutions were considered ranging from a bridge across the gap, to a row of large diameter piles tied back into the face and supporting a cantilevered deck. All these solutions were excessively expensive.

Reinforced Earth Wall

Two reinforced earth wall solutions were examined; one on the existing road line, and the other with a realigned carriageway. The latter was estimated to cost about 75% of the former; it was the cheapest of all the solutions examined and was therefore adopted. The scheme for this is shown in Figures 2 and 3. The total estimated cost of the scheme was about $700 000 of which about one third was for the reinforced earth structure with the remainder being for the realignment of the carriageway and the reshaping and landscaping of the slide debris. The cost of the reinforced earth including excavation, supply of all materials, erection and backfill was about $200/m^2.

It was decided that most of the slide debris should be allowed to remain in the failed position, since it would act as a mass on the toe of the slide. Some landscaping would be carried out and surface drainage installed. After completion, trees will be planted on the slopes.

REINFORCED EARTH DESIGN

Materials

Generally recognised criteria limit the material which is permitted within the reinforced earth fill to a non-cohesive material. It may happen that obtaining satisfactory fill could represent a substantial part of the overall costs. Fortunately for this project, a commercial source was available 6 km away which could produce sand as required, by washing and sieving a natural deposit of decomposed granite. Grading, moisture-density and shear box tests were carried out on samples of the sand with the following results. :

Sieve mm	19,0	2,0	0,425	0,075	0,015
% Passing	91	64	31	17	8

Max. density = 2145 kg/m^2
Opt. moisture = 6,2%
Angle of friction = 31° (at 1930 kg/m^3)

Shear box tests were also carried out to establish the friction between the reinforcing strips and the sand fill. These showed an angle of friction of 25°. Although it was the intention that the comparatively recently developed ribbed strip would be used, the friction tests were, for convenience, only carried out on a plain section thus obtaining a minimum value of friction.

The strips, of maximum length 11,5 m, were galvanised steel with cross sections of either 40 mm x 5 mm, or 60 mm x 5 mm, with projecting ribs. It was assumed that 1 mm of the overall strip thickness would be lost by corrosion over a 70 year period.

Precast concrete panels of cruciform shape and 180 mm thickness were used for the facing. The overall panel size was 1,5 m square with each of the four cut out corners being approximately 375 mm high by 190 mm wide. The mass of a panel was about 1 tonne.

Design

The reinforced earth mass has been dimensioned to ensure its overall internal stability. The width of the structure was about 0,7 times its total height at any section, except at low sections where the minimum width was 4,5 m. The vertical stresses at various levels of the reinforced earth mass have been analysed. This has been done by considering the vertical loads (V) on a particular section and allowing for overturning moments (M) due to earth pressures on the back of the structure. A Meyerhoff stress distribution has been assumed :-

$$\sigma_v = \frac{V}{B-2e} \quad ;$$

σ_v is the vertical stress at each level,
B is the width of the structure,
e is the resultant eccentricity $e = \frac{M}{V}$

The mass is assumed to be in an active state of equilibrium and the horizontal stress (σ_h) at any level is assumed to be equal to the vertical stress (σ_v) at that level multiplied by the coefficient of active earth pressure (K_a).

$$\sigma_h = K_a \cdot \sigma_v$$

Sufficient density of steel has been placed at every level to take the full horizontal stress, i.e. the assumption has been made that the full horizontal stress in the earth has been transferred through friction to the steel reinforcing strips. The condition of adherence between the reinforcing strips and the backfill has been checked. The reinforcing strip must present sufficient surface area to the earth at any level to prevent failure due to sliding of the earth over the steel;

$$L \text{ minimum} = \frac{\sigma_h}{2bN\mu\sigma_v}$$

L is the strip length,
b is the strip width,
N is number of strips per m^2,
μ is coefficient of friction steel to earth.

Similar equations have been recommended by Chang and Forsyth[1], Schlosser and Nguyen-Thanh Long[2], Al-Hussaini and Perry[3], Lee et al[4], and Vidal[5].

PERFORMANCE MONITORING

It was convenient to consider the monitoring under two aspects; internal, referring to conditions within the reinforced earth, and overall, referring to the performance of the structure as a whole. The structure was instrumented at a section which is 16 m high. Figure 4 shows the positions of the various monitoring devices; viz. earth pressure cells, strain gauges, settlement gauges, extensometers and inclinometers. To avoid congestion the pressure cells and strain gauge instrumented strips are offset by one panel from the extensometers and settlement devices.

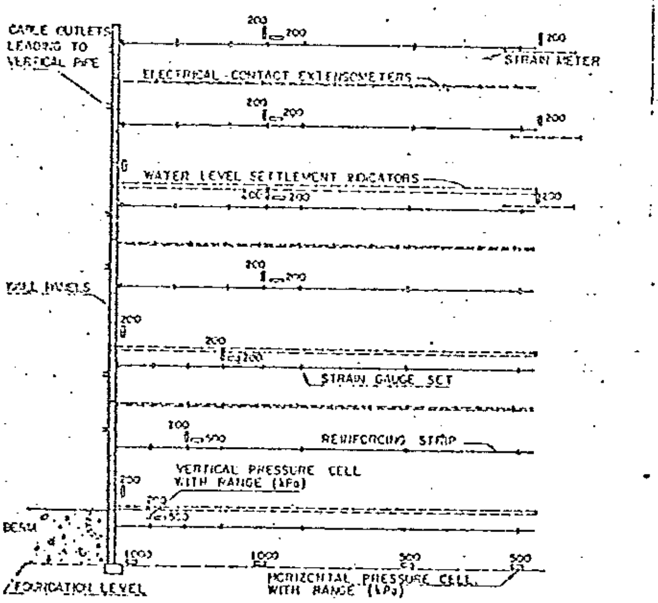

FIG. 4 SECTION SHOWING INSTRUMENTATION

Internal

The instruments have been placed to give information on internal stresses and deformations and also to check the accuracy of the following design assumptions :

(a) The horizontal stress in the earth mass can be calculated on the assumption that the earth is in a state of rest at the top of the structure and in a state of active equilibrium at the lower levels of the structure. As noted earlier, the vertical pressures have been assumed to conform with the Meyerhoff distribution. Both vertical and horizontal pressure cells have been installed at various levels and positions to check these assumptions. The cells were Kyowa double diaphragm mercury filled resistance strain gauge pressure transducers, with one active face approximately 70 mm diameter. The grain size of the fill is small, i.e. 64% smaller than 2mm, and it was considered that the relative size of the diaphragm and fill material was satisfactory. A total of 24 cells were installed, 21 of these within the reinforced earth mass and 3 behind the mass to check possible decrease in pressures leading to tension cracks at the interface of the reinforced earth and wedge of ordinary fill.

(b) The maximum stress in the reinforcing strips is equal to the horizontal stress in the earth at that point, i.e. the full horizontal stress of the earth is transferred to the reinforced strips by means of friction. Strain gauges placed on the reinforcing strips will give the actual stress in the strips at various levels. Seven strips were instrumented at a total of 48 positions as shown in Figure 4. Each position has 4 gauges, each 20 mm long, 2 on the top and 2 on the bottom of the strip, arranged in a bridge to provide compensation for temperature and bending. The gauges were protected by cover plates. For convenience the reinforcing strips, 11,5 m long, were divided into 2 sections, 5,5 m and 6 m long. Calibration of each strain gauge bridge was carried out by incrementally loading the strips in the laboratory.

(c) The locus of maximum stress in the reinforcing strips is assumed to run from a point near the toe of the structure at an angle of 45 + Ø/2 to the horizontal. It then becomes vertical once it reaches a distance about 0,3 times the height of the structure from the facing. This position is important for determining the area of reinforcing strip available for mobilising the frictional forces. The strain gauge readings should indicate this locus.

(d) The earth pressure on the facing is a local effect and the facing elements only take secondary loads. The design assumes no reduction of reinforcing strip strength due to reduction of the available cross-sectional area at the bolted reinforcing strip/facing connection for this reason. The pressure cells will measure the earth pressure at the facing.

(e) The "apparent coefficient of friction" between reinforcing strip and earth has been assumed to be 1,0 at the top of the structure and tan Ø below a height of 6 m from the top of the structure. Pull out strips have been placed at various heights beneath the top of the structure to verify this assumption. Two pull out strips have been provided at each position so that short term and long term tests can be carried out.

Overall

The reinforced earth mass, as shown in Figure 2, was generally founded on soft rock, other than the wall face itself, which was on a couple of metres of backfill. Settlements under the mass were therefore expected to be negligible. However, some settlements within the mass, and some horizontal movements were expected and it was possible, particularly since it was urgent that the carriageway should be reinstated, that these deformations could have lead to subsequent cracking within the carriageway. If such cracks should have appeared, it was essential to be able to assign a rational cause, otherwise there could have been unjustified fear that the overall stability problem was continuing. A precise survey system was therefore installed so that deformations of the wall could be accurately measured. In order to measure deformations within and immediately behind the reinforced mass, inclinometers and water level settlement tubes were placed in the fill together with multiple point horizontal extensometers, as shown in Figure 4. Since the stability of the total mass depended on excessive pore pressures not developing in the future, piezometers were installed to carry out long term pore pressure readings.

SUMMARY

A failure of a highway on a high embankment is described. The subsequent stability analyses and remedial measures are discussed. At this site a reinforced earth structure provided the most economical and rapid solution. Since the wall was to be about 16 m high and 115 m long, a particularly valuable opportunity presented itself for monitoring the performance of the wall. Stresses and deformations within the earth mass, as well as overall deformations of the structure, were measured. The detailed results and interpretation of these are presented separately.

The vital assistance of Arthur Taute and Paul Finlow-Bates for this project is much appreciated, as is that of David Weston who was responsible for much of the instrumentation design, installation and interpretation. The work was carried out on behalf of the Director of Roads of the Transvaal Provincial Administration whose permission to publish is gratefully acknowledged.

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