PHILIPPINES NATIONAL RAILWAY MAIN LINE SOUTH REVITALISATION PROJECT PHASE II

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A series of railway embankment slope stability failure located on the Main Line South between Gapo and Bongalon, 8 km to 10 km north of Legaspi City on the south-eastern corner of Luzon Island in the Philippines, was identified by Ove Arup & Partners in 1996. This paper describes the mechanism and reason for failure of one of the embankments (TP9383), the results of the ground investigation and the preliminary design of stabilising measures.

1 INTRODUCTION

In 1995 the contract for Phase II of the Main Line South Revitalisation was awarded by the Philippines National Railway (PNR) to John Holland Constructions (Asia) using funding made available by AusAID. TMG International in association with Kinhill and Ove Arup & Partners were commissioned by the PNR in May 1996 to assist the client project management his duties and provide engineering advice as required. The Main Line South runs from Manila Central Terminus for 478 km to Legaspi City on the south-eastern corner of Luzon Island.

Construction of the remedial works commenced in June 1995. Ove Arup & Partner's geotechnical involvement started (August 1996) with a walk-over survey of the existing track to identify areas of concern in terms of the overall geotechnics, but concentrated on earthworks and embankment stability. Five major railway embankments failures were identified between Gapo and Bongalon, 8 km to 10 km north of Legaspi City which led to a series of geotechnical investigations being carried out in 1996 and 1997. This paper specifically discusses the failure at one of the embankments (TP9383).

2 BACKGROUND

Embankment TP9383 is situated in an area which is prone to earthquakes, volcanic eruptions, typhoons and floods. The embankment is approximately 15 m high and comprises three 5 m high slopes battered at about 1:1.5 (V:H) with 3 m wide benches. With regard to the railway alignment, the embankment is located on a pinnacle of high ground connecting a bridge which spans a meandering river. On the western side of the embankment the river is located at the toe of the slope which is therefore prone to scour and erosion. The eastern side of the embankment is located in a rice field which separates the toe from the river.

3 SITE OBSERVATIONS

The reconstructed embankment at TP9383 was inspected in August 1996 and found to have one open crack along the centre-line, 10 to 20 mm wide. The crack could be traced down the uppermost of the two batters on the embankment, but there was no indication of movement below this level at the time of the inspection. There was no sign of vertical displacement.

At the time of the site investigation (October 1996), the embankment had already slipped quite considerably. Measured displacements of up to 3 m (vertical) and 2 m (horizontal) were recorded at the back scarp of the failure surface at the top of the embankment (See Figure 1 below). Both vertical and horizontal displacements, as well as large open cracks, were observed down the batters of the failing embankment. A mound of soil was also observed at the toe of the embankment which probably represented the failed material from previous slips as well as this most recent failure. The length of the slip is approximately 22 m at the top of the embankment which then fans out to a length of approximately 47 m at the toe of the embankment.

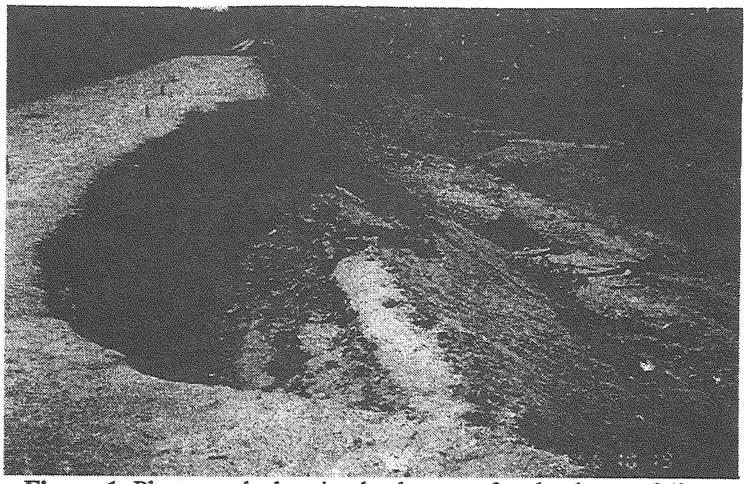

Figure 1 Photograph showing backscarp of embankment failure

During the site investigation, additional cracks opened up at the top of the embankment and down the eastern face of the embankment to the north and to the south of the slip under investigation. No signs of vertical movement were detected on these cracks.

Photographs of the original embankment prior to reconstruction indicate that there were three distinct slips which had occurred on the eastern side of the embankment in the past. The locations of these old slips coincide quite well with the locations of the current slips. This would tend to suggest that the newest slips were simply a reactivation of the old ones. It is understood that the remedial works involved excavation and reconstruction of the upper parts of the embankment, but did not extend to the location, excavation and replacement of the slip plane itself

4 SITE INVESTIGATION

A total of 3 boreholes were drilled down the face of the failing section of the embankment in September 1996 using wash boring techniques. SPT's were carried out in all boreholes with undisturbed 75 mm diameter thin wall tube sampling in the clays. Pocket penetrometer readings and shear vane tests were carried out in the tube samples to determine the undrained shear strength. A single standpipe piezometer was installed in each borehole to monitor the groundwater pressures in the bedrock. During the course of the investigation 4 survey monitoring points were installed down the face of the embankment to monitor surface movements. Monitoring was carried out during the investigation and showed that the embankment continued to move during this period which resulted in the loss of one length of drill casing.

Laboratory testing comprised indicator tests including natural moisture content (6 No.) and Atterberg limits (6 No.) which were carried out by an independent laboratory at the Philippines National University in Manila. Sophisticated triaxial testing was not undertaken as a reliable laboratory in the Philippines with the capability of carrying out such tests could not be identified at the time of the investigation.

5 RESULTS OF INVESTIGATION

5.1 GEOLOGICAL PROFILE

The subsurface profile adopted for analysis is shown in Figure 2. The profile generally comprised up to 9 m of fill overlying 3 m to 4m of soft colluvium (medium-high plasticity silty CLAY) which in turn overlies 2.5 m to 4 m of soft flood plain alluvium (medium-high plasticity silty CLAY) near the toe of the embankment.

The colluvium and alluvium are located above a layer of residual soil, relatively consistent in depth, overlying a fine grained igneous bedrock (Basalt & Dacite). The bedrock appeared to rise toward the top of the embankment, suggesting the embankment was originally constructed on top of a rock outcrop carved by the action of a meandering river.

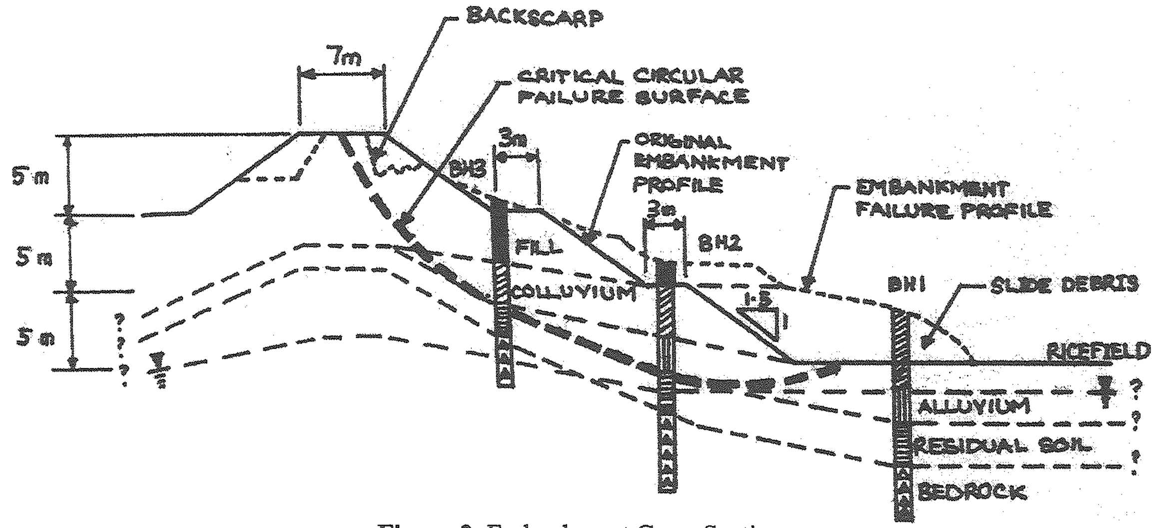

Figure 2 Embankment Cross Section

5.2 GROUNDWATER CONDITIONS

The maximum recorded water levels were used to back-analyse the slope failure. It should be noted that embankment TP9383 was reconstructed during the "wet season" and failure of the embankment occurred during the same period.

5.3 LABORATORY RESULTS

In BH1 the alluvium encountered at the toe has a natural moisture content (84%) in excess of the liquid limit (55%) indicating that the material is very soft to soft which is relatively consistent with the SPT (N=2) and pocket penetrometer (PP=50 to 100 kPa) results. Towards the top of the embankment (BH2 and BH3), the natural moisture content (43% to 53%) of the colluvium approached the liquid limit (46% to 61%), suggesting that the colluvium is soft to firm throughout the embankment. This is relatively consistent with the SPT (N=5 to 8), pocket penetrometer (25 to 100 kPa) and shear vane (S_u=60 kPa) results.

6 STABILITY ANALYSIS

6.1 GEOTECHNICAL MODEL OF SLIP FAILURE

The in-situ geometry, soil profile and groundwater data collected in the field were used to back-analyse the failure. Numerous circular and non-circular failure planes were analysed to determine the limiting equilibrium conditions (Factor of safety=1.0) which produced a slope failure similar to that observed on site. For embankment TP9383 it appeared that the recent failure had been the result of the reactivation of an old slip. Surface observations were indicative of a circular type failure through soft clay.

6.2 SOIL PARAMETERS

Although the immediate failure of the newly reconstructed slope suggested an undrained (c_u) failure in the underlying clay layers, a failure along a previous slip indicates a drained failure (ϕ', c') during periods of prolonged high water levels. Both of these possibilities were considered in the analysis and design of remedial works. The soil parameters initially assumed for analysis were determined from the borehole logs, laboratory test results and site observations. Soil parameters determined from the back-analysis are given in the table below.

Material	((kN/m^3)	c_u (kPa)	ϕ' (°)	c' (kPa)
Fill	20	---	25	5
Colluvium	18	30	25	0
Alluvium	18	30	25	0
Residual Soil	19	70	30	0

6.3 METHOD OF ANALYSIS

The slope stability analysis was undertaken using the two-dimensional slope stability analysis package "*SLOPE*", from the *Oasys GEOsuite* written by Ove Arup & Partners. Both circular and non-circular failure planes were analysed using the Bishop method with variably inclined interslice forces and the Janbu method with horizontally inclined interslice forces. The lowest factor of safety was obtained through calculation of a number of potential slip surfaces at a defined rectangular grid of centres, with the option of extending this grid to search for the overall minimum factor of safety.

6.4 BACK-ANALYSIS

All failure surfaces passed through a common point on the crest of the embankment which coincided with the scarp left by the actual failure plane on site, and surfaced at a point some distance beyond the toe where the evidence of failed material came to an end. The analysis indicated that the critical mode of failure may be an undrained failure in the soft clay or may be a drained failure in the soft clay due to high water levels. Both circular and non-circular failure planes yield a factor of safety close to unity in both cases.

7 STABILISING MEASURES

7.1 FACTOR OF SAFETY

Due to the high economic and human risk associated with a slope failure of this kind, a minimum factor of safety of 1.4 for redesign was adopted (Geotechnical Manual for Slopes, 1984) for the groundwater conditions recorded on site. In addition, a factor of safety of at least 1.2 was targeted for the highest probable assessed groundwater levels.

7.2 LOADING CONDITIONS

A surcharge due to train live loading was included as part of the preliminary stabilising measure design. Although not discussed here, an earthquake loading was also incorporated.

7.3 STABILISING OPTIONS

The following stabilising options were considered and analysed:

- *Small toe berm.* This involves excavating the fill and all soft material (colluvium & alluvium) that appear to be the cause of the problem and reconstructing the embankment with a small toe berm with slopes no greater than 1:1.5 (V:H). If soft material is found to extend below the water table, redesign would be required during the construction process, but would be based on the analysis carried out.
- *Large toe berm.* This involves excavating and replacing the fill and soft material (colluvium & alluvium) above the natural ground level and reconstructing the embankment with a larger toe berm with slopes no greater than 1:1.5 (V:H).
- *Install stone columns.* The use of stone columns would require cutting down the existing embankment to a level platform and stockpiling the excavated material.The use of stone columns would then be installed at close centres on a grid pattern over the full width and to beyond the toe of the embankment. A drainage blanket, with free draining granular material would then be placed over the stone columns to allow water to drain through the stone columns. This has the advantage that it would alleviate artesian water pressures if they develop in the underlying rock. The embankment would then be reconstructed over the drainage blanket with side slopes no greater than 1:1.5 (V:H).
- *Construct a bridge over the slip area.* This solution requires the piers and abutments to be piled to bedrock and sufficient soil removed between them to arrest the movement. Failure to do this could result in unacceptably high lateral pressures being developed on the sides of the bridge foundations and failure of the bridge itself.

The stabilising option adopted by the Philippines National Railway comprised constructing a bridge over the slip area. This option was selected because it was the most cost effective and could be completed within the project time scale.

8 INTERESTING ASPECTS OF THE PROJECT

Some personal aspects of this project and lessons to be learnt from working in a developing country such as the Philippines, which the author believes to be of special note are listed below.

Overseas work means different cultures, traditions, religions and languages. As an expatriate engineer you soon learn to be tolerant, respectful and understand the importance of these differences and their significance in the workplace. Working in an overseas country can be challenging, forces you out of your 'comfort zone' and builds self confidence. You quickly learn to take more responsibility, adapt to an unfamiliar work environment and use your own initiative to get the job done, often in a remote area where resources are limited.

Sophisticated equipment or methods of working are not always available in a developing country. As such, learning to retrieve as much information as possible from pragmatic drilling methods adopted by local contractors may be required. Working in an overseas country provides excellent experience in a geological environment which is often greatly different from home. Hence, new problems and new solutions. Australian practice is often not appropriate for solving geotechnical problems. Finance and technology are often the key governing constraints.

This project provided the author with a real insight into a variety of challenges of both a personal and technical nature. Overseas experience can therefore be invaluable to ones personal and professional development.

9 CONCLUSION

The following conclusions can be drawn from the experience on this embankment failure:

- A number of alternative preliminary stabilising measures were designed for the failing embankment.
- A comprehensive ground investigation must be carried out before the remedial works are designed, so that the designer has a full understanding of the failure mechanism and can develop an appropriate solution.
- Geotechnics formed an essential part of the remedial works.
- On a large infrastructure project such as this, it is essential that both consultant and contractor work closely together at the commencement of the project in order to produce an efficient and effective design. Unfortunately this was not the case on this project which resulted in expensive consequences.

10 ACKNOWLEDGEMENTS

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11 REFERENCES

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