

**ANALYSIS OF LANDSLIDE AT OUTSIDE DUMPING AREA
AIR LAYA COAL MINING SITE, INDONESIA**

Marwan Asof,
Sriwijaya University, Indonesia

Nurly Gofar,
University Teknologi Malaysia, Malaysia

Ahmad Rivai,
Bukit Asam Coal Mining Co., Indonesia

ABSTRACT

A landslide occurred at final fill slope Outside Dump Air Laya Coal Mining, Indonesia on 27 November 2002. The incident involved a translation movement of soil fill as long as 1.5 km towards Enim River and several rotational slope failures along the movement. It resulted in the fall down of an electrical tower supplying electrical current to the surrounding area. It also damaged the mud collection pond and dikes constructed on the toe of the fill slope as well as agriculture land. There were no fatalities and serious injuries in the incident. The paper presents a comprehensive investigation into the cause and mechanism of the landslide. A number of technical findings are of interest, including the failure mechanism of the fill and effects of surface cracks and antecedent rainfall on slope stability. The investigation concluded that the landslide was started by cracks existing on the fill surface. The prolonged and heavy rainfall that preceded the landslide resulted in significant water ingress into the ground. Both factors played contributory parts causing the landslide incident.

Key words: landslide, surface cracks, heavy rainfall, infiltration, undrained condition.

INTRODUCTION

A rapid soil movement occurred on the dawn of 27 November 2002 at final fill slope outside dump Air Laya coal mining site at Bukit Asam, South Sumatra, Indonesia (Figure 1). The incident resulted in the fall down of an electrical tower supplying electrical current to the neighboring area. It has also destroyed some mud collecting facilities and dikes, as well as agriculture land along the toe of the slope. It was fortunate that there were no fatalities and serious injuries in the accident.

The landslide involved a translation movement of soil fill as long as 1.5 km towards the river and several rotational slope failures along the movement (Figure 2). The failed part was 72 hectares or about 11% of the total dumping area. Rapid movement of soil has lasted for about 4 days i.e. between 27 and 30 November 2002. The highest rate of movement was 724 mm/hour took place on the second day and this movement was gradually diminished within the next 10 days.

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The amount of soil moved during that period was about 690,000 bulk cubic meters and the maximum change in elevation was 12 m. Note that the height of the dumping material at the time of failure was 60 m and the surface formed an average angle of 15°. Based on the slope angle and the area involved in the failure, this movement can be categorized as a landslide (Okamura, 2001), with a rapid movement (Hunt, 1984). The soil in the debris was sandy silt and sandy or silty clay with very low cohesion. At the time of failure, the soil was wet and very loose.



Figure 1 Soil Movement on the fourth day of the landslide

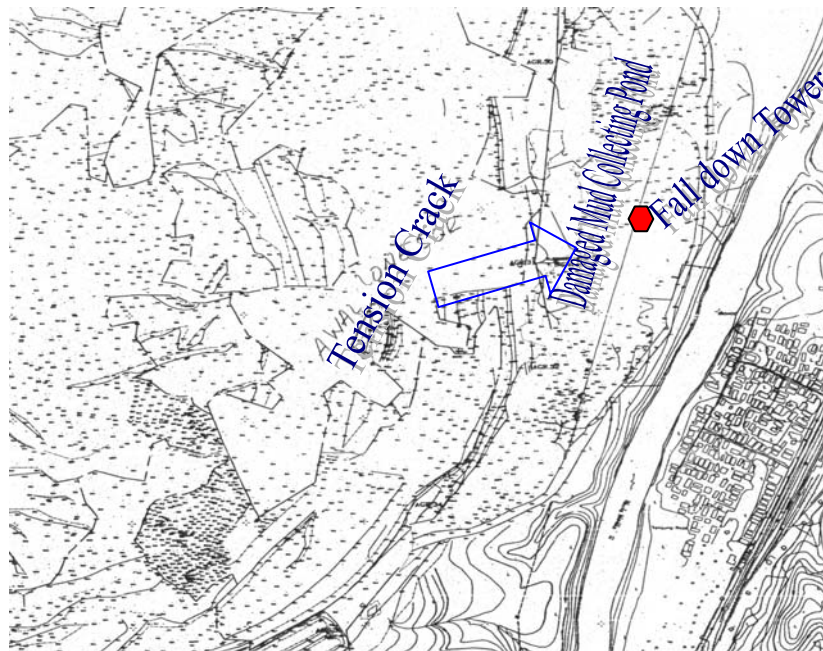


Figure 2 Situation of landslide

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DESCRIPTION OF THE SITE

The site was actually a dumping area which covers a 650-hectare of land located to the north of the open coal mining site, Air Laya. The geological structure in this area is marked by an unsymmetrical anticline at the north and a symmetrical syncline below the mining area. Geological investigation made before the operation of the open coal mining showed that there is no sign of instability such as fault found below the dumping area.

The original topography of the area was undulating with some streams, small valleys, mounds and hills. Initial elevation was 36.50 to 84.50 m above sea level with the lowest elevation marked as Enim River. The surface soil consisted of sedimented rock and lacustrine soil, and was covered by bushes, swamps, and rice fields.

Based on information collected during the investigation, the land clearance prior to the dumping was not made up to the geotechnical standard. Trees, mound and hills were cut only to accelerate the operation of the dumping equipment. This might have resulted in the presence of weak layer in between the original ground surface and the fill layer.

The dumping was started at 1986 as soon as the Air Laya open coal mining began to operate. The dumping process was made using *spreader* with a standard procedure and pattern designed to ensure that there is no lenses of weak soil within the fill, and that the soil fill is well compacted. However, this standard procedure cannot always be ensued due to the condition of the dumped material and the climate.

In order to reduce the risk of failure especially toward Enim river, the standard procedure had required that the maximum angle for each stage of dumping be between 12 to 18°, while the final slope angle was limited to 6-7°. Mud collecting ponds were constructed at the toe to prevent seepage of waste product into the river, while dikes were built along the end of fill area parallel to the river in order to prevent the dumped soil goes into the river.

The climate in this area shows a six-month of dry season (May - October), and six-month of rainy season (November to April). Based on 20-year return period, the maximum rainfall is 449 mm/month in March, while the minimum rainfall is 129 mm/month in August.

OBSERVATION PRIOR TO LANDSLIDES

Movement was observed for the first time at the end of year 2001 when the maximum thickness of dumped fill soil has reached 80 m. Some measured have been made to control the situation from becoming worse such as by removing the dumped material to other sites and by smoothing the surface. However, an observation made on 17 October 2002 has revealed clear signs of distress including the formation of deep tension cracks parallel to Enim River, the presence of water on the surface of the fill despite of dry condition (Figure 3), and some surface movements on the upper part of the slope.

Observation on ground water table made during the dry season in August 2000 (Gunadi, 2001) showed that perched water table has formed at depth of 17 m below surface of the dumping while measurement made during in December 2001 (Gofar, 2002) showed that the perched water table raised as high as 4 m below the fill surface during rainy season. Note that at the time of observation, the height of fill is about 60 and 80 m respectively



Figure 3 Signs of distress at fill surface prior to landslides

INVESTIGATION OF THE LANDSLIDE

An emergency team was formed immediately after the fatal landslide to save vital facilities in the surrounding area. The team was also assigned to commence an initial treatment to the failed land in order to prevent subsequent failures. Furthermore, an investigation team was put together to initiate a comprehensive investigation program into the cause and mechanism of landslide. The program includes the following stages:

1. Analysis of the topography and geometry of the slope
2. Analysis of field data.
3. Field investigation.
4. Laboratory test of samples obtained from boreholes and surface.
5. Slope stability analysis
6. Diagnosis of the cause and mechanism of the landslide.

Topography and Geometry of Landslide

A detailed study was made to establish the representative topography and geometry of the site. This study was conducted through field mapping of the landslide area, and review of existing maps made before dumping, prior to and after the landslide. The maps were combined using software *Surfer* (Golden Software, 1996) to establish the cross section of the dumping area and to find out the most critical cross-section for slope stability analysis. Review of existing maps prior to the dumping activity has shown that there is a possibility of the development of a weak lens below the existing crack that start the movement of the landslide. This lens extends on the direction of the landslide. The information was then used by the investigation team to estimate the failure plane and to arrange the layout of the borehole on the site.

Analysis of field data

Field data including the records of soil movement, rainfall, and observation of groundwater table prior to and after failure were collected to study the cause and the mechanism of the landslide.

Slip indicators were installed at various locations on the dumping area to monitor the movement of the soil. Indications of surface movement on the upper part of the slope were observed for the first time at the end of 2001 when the maximum thickness of dumping has reached 80 m. Some measures have been made to control the situation from becoming worse such as by removing the dumped material, smoothing the surface in order to cover existing cracks and limit the development of new cracks. However, an observation made on 17 October 2002 has revealed clear signs of distress including the formation of deep tension cracks parallel to Enim River. These findings showed that the effort made to control the situation has been a little bit too late.

Some cracks actually have initiated the failure in the soil with failure plane progressing into the weak layer. There is also a possibility that perched water has seeped through the crack and other voids in the fill, causing the development of pore water pressure within the fill. As mentioned before, the perched water table was observed at 4 m below the surface. This could be the reason for the presence of water at some locations on the fill surface despite of the prolonged dry season. Furthermore, the water might have flowed down into layer 2 (interface between original soil and fill) and induced a pore water pressure within that layer and an undrained condition to the soil. Measurement made on shallow boreholes after failure showed that the water level is at less than 1 m below the new ground surface.

The ground failure occurred after periods of heavy rainfall that started on 13 November 2002 after a period of a very dry season that last for about six months. This condition suggested that rainfall might play a contributory part into the landslide. A rainfall of more than 100 mm took place within 2 hours in November 13 followed by periods of rain continued until November 24 with cumulative rainfall of 318 mm (Figure 4, and 5). Analysis of rainfall data based on 20-year return period showed that the rainfall after November 13 was on the highest rating for 3, 5, and 7 day-rainfall data.

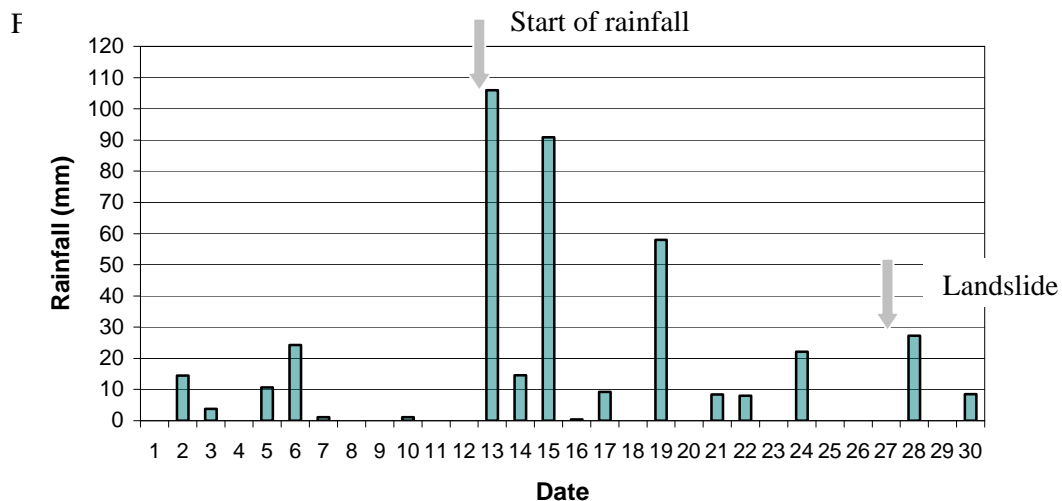


Figure 4 Daily rain-fall in November 2004

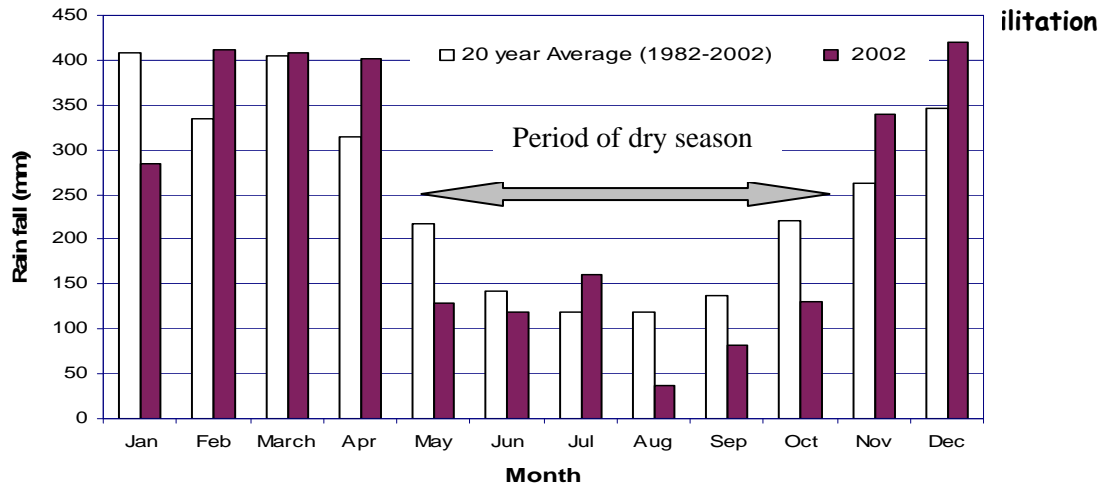


Figure 5 Monthly rain-fall in 2002 compared to 20 year average monthly rainfall

Ground Investigation

The ground investigation was planned based on the analysis of topography and the geometry of the landslide. It comprises of 11 bore holes and 2 check-bore-holes through the sliding area as shown in Figure 6.

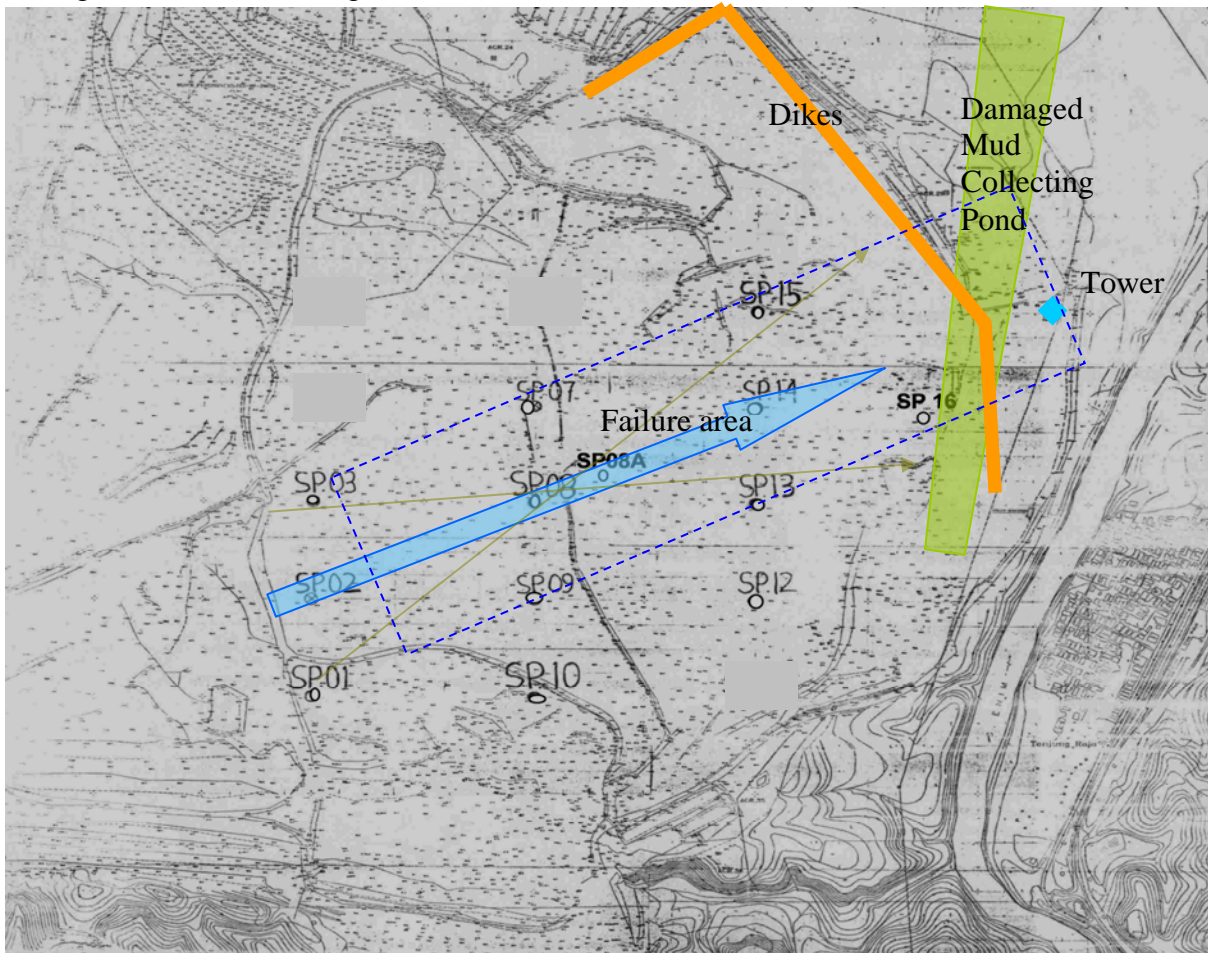


Figure 6 Layout of the boreholes

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The bore-log was drawn carefully to establish the soil profile. As expected, the general stratification of the fill area consisted of three layers. Layer 1 is the dumped material, mostly comprises of silty clay with some sand fraction. Layer 2 is the interface layer between the natural ground and the fill that comprises of loose silty clay mixed with wood chips, soft rock and red clay, and some organic matter. Layer 3 is the unconsolidated natural ground consists of unconsolidated weathered clay stone and siltstone. The soil profile along the most critical cross-section and the properties of soil at each layer is given in Figure 7. This profile passes a location where the failure starts at a tension crack (BH- SP8) to the toe where most of the damage occurred (Figures 8). Five boreholes were completed along the cross-section i.e. BH-SP2, BH-SP08, BH-SP08A, BH-SP14, and BH SP16.

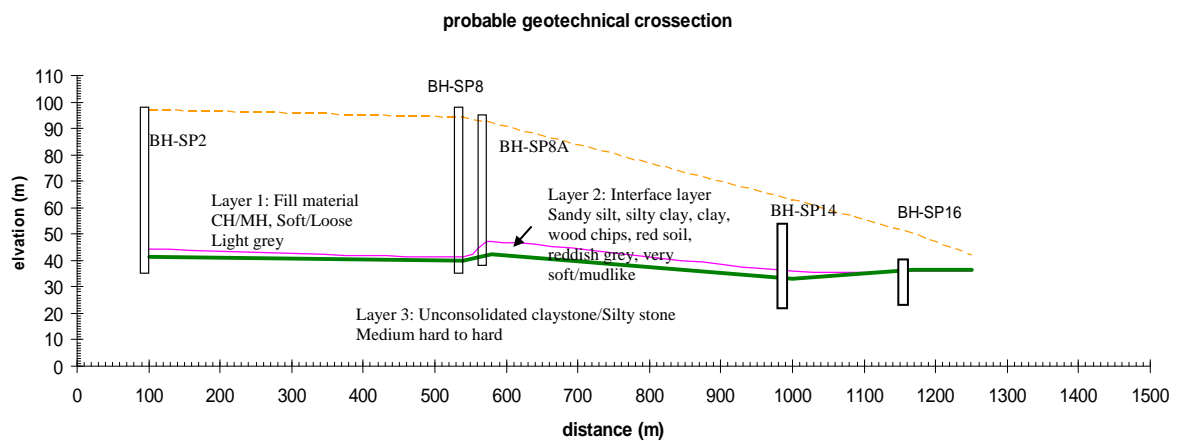


Figure 7 Soil Profile



a. Tension crack at the crest of failure



b. One view at the toe of the failure

Figure 8 Views of the crest and toe of the landslide

Results of Laboratory Tests

Disturbed and undisturbed samples were retrieved every 3 m for visual inspection and laboratory testing. Laboratory test results showed that the fill material (layer 1) consists of 22.74% sand, 32.7% silt, and 44.17% clay, while the plasticity index is 36.4%. Degree of saturation is close to 100%. X-ray tests showed that the fill material contains a large amount of montmorillonite, and the material showed a unique characteristic in that the material absorbs water very quickly to reach saturation. On the other hand, the permeability of fill soil is very low about 1.5 to $5.5 \cdot 10^{-8}$ m/sec, therefore water takes a long time to escape from the pores even under load. The variation of the shear strength obtained for fill soil is very large which indicates the disparity of the characteristics of the material being dumped.

Layer 2 also consists of various materials including red clay of high plasticity, sandy silt, silty clay, with some mixture of wood chips and organic material. The shear strength obtained from triaxial test gave a strong indication that the interface layer has lost its strength due to the presence of water. Layer 3 represents the original ground consisted of unconsolidated weathered clay stone and siltstone. Table 1 shows the average shear strength of the soil in each layer.

Table 1. Average shear strength of the soil

Test/ Parameter	Unit	Layer 1 (Fill soil)	Layer 2 (interface layer)	Layer 3 (original ground)
Triaxial UU				
c_u	kPa	36.27	12	453
Triaxial CU				
c_u	kPa	23.67	42.73	73.70
ϕ	(°)	16.62	5.80	26.30
Direct shear test				
c'	kPa	14.69	9.07	44.52
ϕ'	(°)	17.59	21	37.60

Surface samples were collected from the surface for identification of the debris. Laboratory test results shows that after the failure, soil has lost some of its plasticity (IP = 23.8%), which shows that the soil has lost its cohesion due to saturation. The soil also lost its shear strength especially in terms of cohesion as compared to the soil in the fill before failure (Table 2). This indicates that there was a decrease in shear strength of the soil before failure due to saturation.

Table 2. Shear strength of surface soil after failure

Location	Condition	Number of sample	c' (kPa)	ϕ' (°)
Crest	Before failure	24	17.09	15.10
	After failure	10	5.37	13.10
Toe	Before failure	30	39.84	24.07
	After failure	25	3.65	17.28

NUMERICAL SIMULATION

Slope stability analyses were carried out using computer program SLOPE/W from Geoslope v. 5.02 for several candidate cross sections. Morgenstern-Price method, which allows for non circular failure plane, was used in the analysis. It was found that the most critical condition was found at cross-section b-b. This cross-section passes through the real failure which started with a crack at point SP08 and end at mud collection pond. The subsequent simulation was conducted using this cross section as shown in Figure 6.

The simulation was made for two cases i.e.: (1) as planned condition, and (2) at failure. Soil properties used for analysis are shown in Table 1. Same block failure plane which starts at the tension crack at position of SP08 or coordinate (365200, 590400)), passed through layer 2, and end at the mud collecting pond at (365766, 590630) was assigned of both cases.

For case 1, shear strength parameters obtained from Triaxial CU test were used for all layers. In this case, perched groundwater table presents at specified level obtained from boreholes prior to failure. This is done to show that the design of fill is safe as long as the conditions did not deviate from the original plan. The output of SLOPE/W for case 1 is shown in Figure 9. The factor of safety obtained for this case is 2.432.

Case two simulates the condition at failure. Unconsolidated undrained shear strength is used for layer 2, while shear strength data from Triaxial CU test were used for layer 1 and 3. In this case, water level was at the ground level and tension crack was filled with water. The factor of safety was found to be 0.495 which is much lower than Case 1. The output of SLOPE/W for case 2 is shown in Figure 10.

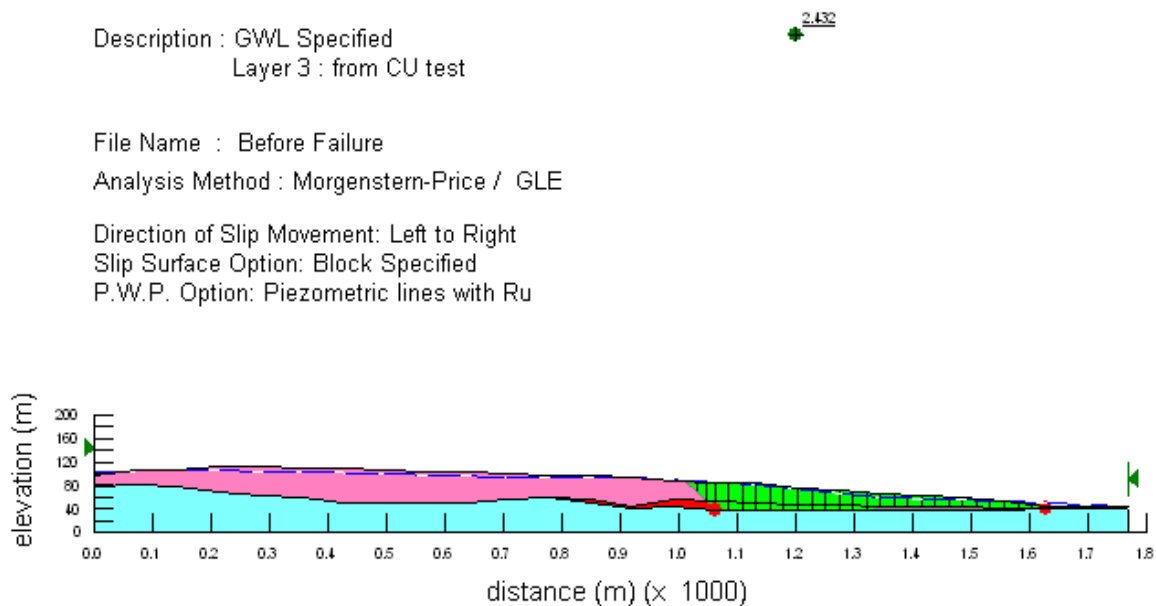


Figure 9 Output of SLOPE/W for original condition (before failure)

Description : GWL at Surface, tension crack
Layer 3 : from UU test

0.495

File Name : at Failure

Analysis Method : Morgenstern-Price / GLE

Direction of Slip Movement: Left to Right

Slip Surface Option: Block Specified

P.W.P. Option: Piezometric lines with Ru

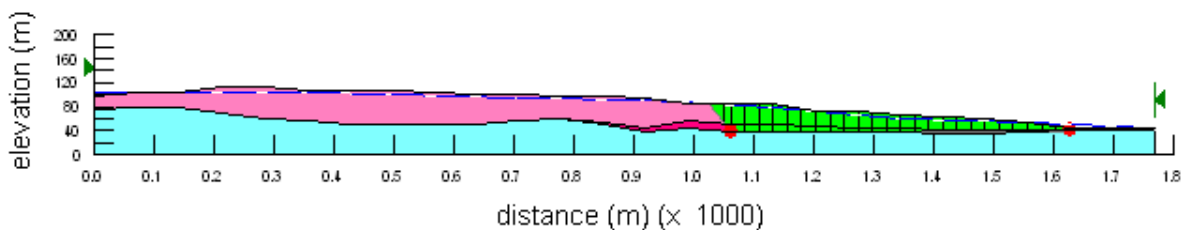


Figure 9 Output of SLOPE/W for “at-failure” condition

DIAGNOSIS OF THE LANDSLIDE

The stability analyses indicate that the dumping area would have been safe if the condition did not depart from original plan. The geological study of the area has shown that this area is stable in which there are no indications of fault or bedding to the direction of the toe. The topography and geometry of the slope has shown that the slope angle is actually very mild i.e. less than 10° . In addition to that, the dumping process was planned to follow standardized procedure to ensure the uniformity and compaction of the fill. Mud collecting ponds were constructed at the toe to prevent seepage of waste product into the river; while dikes were built along the toe to prevent the dumped soil goes into the river (Figure 6).

The study also showed that there are some possible causes of the landslide i.e.: (a) Incomplete clearance before dumping has resulted in the presence of weak layer at the interface, (b) Swell-shrink characteristic of the fill material that lead to the development of surface crack, (c) Non uniformity in the size of the material being dumped has led to the formation of perched water table in the fill and development of pore water pressure in the voids, (d) Extended dry season prior to failure has suggested that some cracks have propagated deep enough to reach the interface layer causing the soil to loose its strength, and (e) Prolonged and high intensity rainfall has triggered the failure by saturating the fill and filling the tension crack with water which added to the driving force at the crest.

Based on the facts mentioned above, the investigation has established evidence that the soil movement was actually has started long before the landslide occurrence. It started as a development of some surface cracks. Due of its position, several cracks has developed and propagated into deeper soil layer. This crack was filled with water through seepage that flows through voids within the fill itself. The saturation process eventually caused the reduction of the shear strength of soil both within the fill and the interface layer. This process was accelerated by the prolonged and high intensity rainfall that started on November 13.

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The landslide occurred only after the water has reached the weak interface layer and reduces the shear strength of the soil completely.

The likely mechanism agrees well with the observation made on the site. The failure has started at a tension crack, followed by some rotational failures along the movement (Figure 10) and end at the mud collecting pond at the toe of the slope. This landslide can be classified as multiple retrogressive failures (Anderson and Richards, 1987).

The saturation process also caused a change in the thickness of the fill by 12 m which supposed to induce a change in the volume of fill of about 8,400,000 cubic meters. The actually the amount of displaced soil found at the toe of the landslide (690000 bulk cubic meter) is very small compared to the change in the thickness of the fill soil. This showed that most of the volume change could be attributed to collapse compression of the fill.

Furthermore, the investigation also indicates that the landslide was localized and will not affect the surrounding area. Continued monitoring showed that no subsequent movement was observed after the landslide. Nonetheless, some safety precautions have been done for example the reconstruction of the mud collection ponds and dikes, as well as termination of dumping on the failed area.

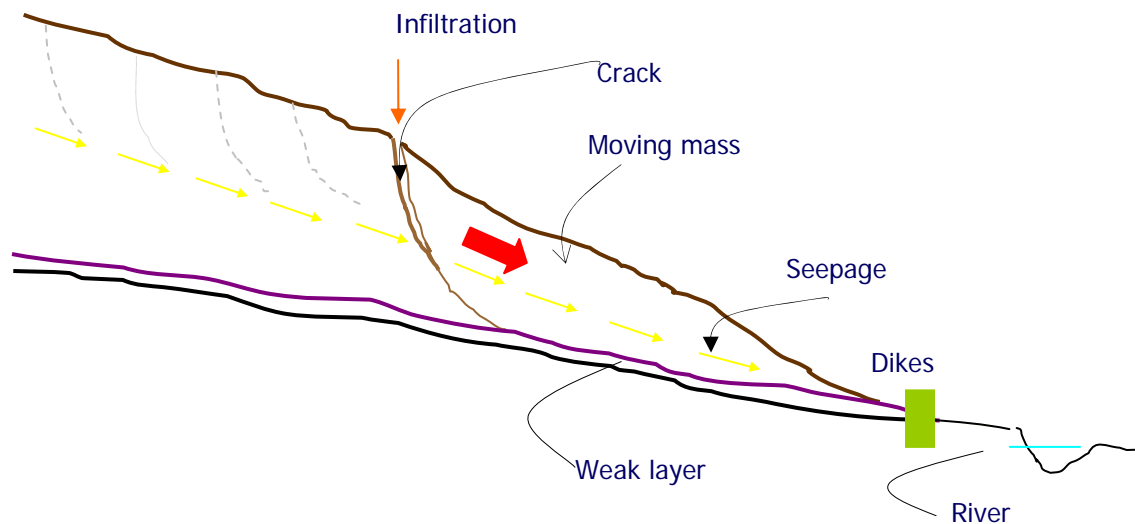


Figure 10 Mechanism of Failure

CONCLUSION

Comprehensive investigation has been made into the cause and mechanism of the landslide. It can be concluded from the study that the landslide was started by the development of tension crack at the surface that incurred a reduction of shear strength of the fill soil itself as well as the interface layer. The prolonged and heavy rainfall that preceded the landslide played a contributory part resulting in significant water ingress into the ground which triggered the mass movement causing landslide.

REFERENCES

- Anderson, M.G. and K.S. Richards Eds. (1987) *Slope Stability, Geotechnical Engineering and Geomorphology*, John Wiley and Sons.
- Charles, J.A. and Watts, K.S., 1996, "The assessment of the Collapse Potential of Fills and its Significance for Building on Fill", *Proc. Instn Civ. Engng.* v 119:1: p.15-28.
- Fajar R.A. (2001) *Penentuan Parameter Kekuatan Tanah Tak Jenuh Untuk Analisis Kemantapan Lereng*, Thesis Magister. ITB, Bandung.
- Fourie B. (1996) "Predicting Rainfall-induced Slope instability", *Proc. Instn. Civ. Engng.*, 119: pp.
- Geo-slope (2002) *Slope/W User's guide for Slope Stability Analysis version 5*. GEO-SLOPE Internationa Ltd., Calgary, Alberta, Canada.
- Gofar N., Budhi Setiawan (2002) "Pengaruh Peningkatan kadar air terhadap Potensi Keruntuhan Lereng Tanah" *Proceedings Seminar Nasional Slope 2002*. Kerjasama Univ Parahyangan dengan HATTI, Bandung.
- Gofar N., Budhi Setiawan, Ratna Dewi (2002) "Pengaruh Perubahan Kandungan Air terhadap Kegagalan tanah Timbunan" *Proceedings PIT HATTI IV*. Surabaya.
- Gofar N. (2004) "Prediction of Collapse Potential of Fills", *Proceedings SEPKA 2004*, FKA, University Technology Malaysia, Skudai.
- Gunadi, C. (2002) *Penentuan Kuat Geser Jangka Panjang Batu Lempung melalui ujian rayapan geser langsung di laboratorium*, Thesis Magister, Program pascasarjana, ITB, Bandung
- Hunt R.E. (1984) *Geotechnical Engineering Investigation Manual*, McGraw-Hill.
- Hunt R.E. (1986) *Geotechnical Engineering Analysis and Evaluation*, Mc Graw Hill
- Kramadibrata, Suseno (1996), "Rock Engineering Mining Problems in Indonesia" *Proc. '99 Japan-Korea Joint Symposium on Rock Engineering*, Fukuoka, Japan 2-4 Agustus .
- Nash D (1987) A Compaarative Review of Limit Equilibrium Methods of Stability Analysis Chaper 2 in *Slope Stability, Geotechnical Engineering And Geomorphology*, MG Anderson and KS Richards (Eds) John Wiley and Sons
- Rahardjo H. et. al. (2000) *Rainfall Induced Slope Failures, Geotechnical Engineering Monograph 3*. NTU-PWD Geotechnical Research Centre, NTU, Singapore.
- Transportation Reserach Boards (1990) *Guide to Earthwork Construction State of the Art Report 8*, National Research Council, Washington DC.
- Turnbull W.J. Hvorslev M.J. (1967) "Special Problems in Slope Stability", *Journal of the Soil Mechanics and Foundations Division, ASCE*, July, pp.

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