Rainfall-Induced Failure on Unsaturated Fill and Highly Weathered Schist Slopes

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ABSTRACT

Rainfall-induced failure is common after intensive rainfall events in mountainous areas, particularly in areas covered by residual soils. The infiltration of the surface water can have a significant influence on the stability of slopes. The increase of infiltration into the soil reduces the matric suction of the soil and hence results in a reduction of the shear strength of the soil, which increases the potential for slope instability. The mountainous road slopes in Timor Leste are located on scarp nearby the ocean. Slope failures repeatedly happened at several locations in Timor Leste after the intensive rainstorm events during the wet season. At some locations, the failure of the slopes occurred again even after the repair of the slopes. A geotechnical investigation, a geological study, and a surface seismic geophysical survey were conducted at the site to obtain soil/bedrock profiles and properties for the numerical modeling in this study. Seepage and slope stability analyses were conducted to evaluate the increase of the pore water pressures during the wet season and the resulting reduction of the slope stability. The results of the seepage analyses demonstrated that the pore water pressure within the slopes was significantly increased due to the increase in infiltration during the wet season. The results of the slope stability analyses showed that the as-built fill slopes at the dry season had only an initial factor of safety of about 1.3. The factor of safety of the as-built slopes was further reduced to reach unity or less during the wet season. The results of the analyses concluded that the triggering mechanisms for the failures are the combinations of geotechnical and hydrological properties in addition to the physical properties of the slopes. Conceptual repair plans including (1) regrading of the existing slopes, (2) installation of tieback anchors, and (3) installation of geogrids were considered in this study. Recommendations for the design of roadway in an area that is susceptible to a rainfall-induced slope failure potential were provided in the paper.

Keywords: unsaturated soil, rainfall-induced failure, slope stability, seepage analysis, matric suction

1 INTRODUCTION

Rainfall-induced failure is one of the most detrimental natural hazards commonly occurring after intense rainfall events in mountainous areas, particularly in areas covered by residual soils in tropical regions. The rainfall-induced failure can occur on both natural slopes where geomaterials heterogeneously distribute due to the processes of natural soil formation and constructed slopes which are mechanically placed and compacted to particular loads. Rainfall intensity, soil/rock properties, and configuration of the slope have been widely accepted as the primary controlling factors in the rainfall-induced slope failure (Brand et al., 1984; Rahardjo et al., 2007). The amount of infiltration from the surface water can have a significant influence on the stability of slopes. The increase of infiltration into the soil reduces the matric suction of the soil and hence results in a reduction of the shear strength of the soil. The reduction of the shear strength reduces the resisting force of the slope, and therefore, increases the potential for slope instability.

A roadway under Package R2 of the Road Network Development Sector Project (RNDSP) in Timor Leste was constructed in around 2012. Figure 1a-b shows the location of the roadway. Cracks and slope failures at some sections were observed as early as in Jan 2013. It was reported that the slope failures commonly occurred after heavy rainfall events. Some of the slopes were repaired immediately after the failure. However, the failure of the repaired slopes happened again after the next raining season.

This paper aims to present the results of evaluation of the as-built conditions of the roadway for determination of causes of the slope failures. Site assessment, field investigations, and laboratory testing were conducted to provide soil/rock properties and site conditions for our geotechnical analyses. Seepage and slope stability analyses were conducted to determine
the causes of the failures. This paper presents the results of our evaluation and findings/suggestions for design and construction of roadways with a potential for the rainfall-induced failure.

2 SITE GEOLOGY

The Island of Timor is located at the southeast part of the Indonesian archipelago and is one of the islands of the southern outer Banda arc. This arc marks the zone of collision between the Indo-Australian Plate, the Pacific Plate, and the Eurasian Plate. The island being an accretionary wedge is formed as a result of a collision between the northern edge of the Australian Plate and the Banda arc. This continental collision is one of the reasons for the immensely deformed, sheared, and thrust lithologies that are seen in the syn-collisional metamorphic rocks of the Aileu complex at the project location.

The emergence of Timor which started in around 4-3 Ma is from extreme uplifting due to the locked continental collision. This is reflected in the mountainous nature of the island and the chaotic geology. The island is fragmented into an immense number of thrust and uplifted zones. Softer sediments, shale, sandstone, and bedded limestone are sheared, crushed and folded into complex lithologies. The weakness of the rocks and steep terrain with the occurrence of intense rainfall make erosion and sedimentation the most active geologic processes in Timor-Leste.

The project is located in the syn-collisional metamorphic rocks of the Aileu complex. The Aileu formation is of Permian age and can be described as hard metamorphosed deep marine sediments. R.F. Berry (1979) indicated that the Aileu complex is mainly composed of phyllite and quartz phyllite grading to schists in the east. These are interpreted as shales, siltstones, and greywackes which were deposited rapidly on an outer shelf or continental slope. They are highly fractured, jointed, and weathered, which potentially could cause the slope instability of the rock mass.

3 SITE ASSESSMENT

The conditions of the slopes and road sections were observed at the site from the 13th to the 17th of November 2017. Ten locations were identified to have slope and/or drainage issues within the Package R-2 road section of the site. The ten locations that were identified are shown in Figure 1a. The types of slope failures observed during the site visit in November 2017 were categorized and presented in Table 1. This table indicates that the slope failures commonly occurred after heavy rainfall events either at fill slopes with tension cracks observed at the pavement surface or at cut slopes with highly weathered rocks.

4 FIELD INVESTIGATIONS AND LABORATORY TESTING

4.1 Field investigations

Field investigations including a geotechnical investigation, a geological study, and a surface seismic geophysical survey were performed at the site. The results of the field investigations were described as follows.

Surveying of cross sections was conducted at the failure locations. The surveying data provide information regarding the post-failure topographic conditions at the failure locations and the as-built topographic conditions at the non-failure locations. The results of the surveying were used in the analyses.

Exploratory drilling was carried out at the failure locations for both rock and fill slopes. The boring logs indicate that the soil/bedrock consist of sandy gravel and/or silty sand underlain by weak to moderately weak schist bedrock to the maximum boring depth of 10 m. The water level measurements were also carried out in the boreholes after completion of the field operations. It was observed that all of the boreholes were dry up to the maximum exploration depth of 10 m.

Geological surveys were also performed at the rock section of the failure locations. The selected rock sections were classified according to the Rock Mass Rating (RMR) classification system developed by Bieniawski (1973). The RMR is determined by several parameters such as the uniaxial compressive strength of intact rock mass, the rock quality designation, the spacing between joints, the condition and orientation of discontinuities, and the groundwater conditions. The rock mass was also classified according to the Geological Strength Index (GSI) introduced by Hoek et al. (1995) which was estimated from direct inspection of the structure and surface condition of the rock mass. The rocks at the majority of the failure locations had the RMR of less than 20 and the GSI of 20 indicating poor to very poor-quality rock.

Furthermore, seismic refraction tests were carried out in order to detect the soil/bedrock boundary, the thickness of the layers, and the depth of the groundwater table.

4.2 Laboratory testing

Laboratory testing was conducted on selected samples obtained from the borings in both the rock and fill slopes to determine the geotechnical engineering properties of the soil/bedrock for the geotechnical analyses. Laboratory testing that was performed for soil samples included measurements of water content, dry density, grain size distribution, and bulk specific gravity. For intact rock samples, unconfined compressive strength tests and point load tests were performed. The results of the laboratory testing were used in our seepage and slope stability analyses.
Fig. 1. a) location map; b) location of the roadway with ten failure locations; c) and d) failure conditions of fill and rock slopes at Km 67+900; e) failure condition of fill slope at Km 67+160; and d) schist outcrop at Km 67+160.
5 GEOTECHNICAL ANALYSES

The geotechnical analyses conducted for this study included (1) seepage analyses; and (2) slope stability analyses.

5.1 Seepage analyses

5.1.1 General

It was reported that rainfall-induced slope failures frequently happened after several heavy rainfall events at the site. Therefore, two-dimensional (2-D) seepage analyses were performed to evaluate the change in pore water pressure for the fill and rock slopes from the dry season to the wet season. The seepage analyses were conducted for three scenarios including (1) as-built slopes under steady-state conditions; (2) as-built slopes under transient conditions for a duration of 1 year; and (3) current slopes with proposed repair plans under steady-state and transient conditions.

5.1.2 Method of analyses

The seepage analyses were conducted using the computer program SEEP/W version 9.0.4 (GeoStudio, 2018). SEEP/W is a finite-element program that can be used to model flow through soil and bedrock and distribution of pore water pressures. The program can consider the effects of infiltration, surface seepage, runoff and ponding, and groundwater recharge.

Cross sections at the locations of Km 66+900 and Km 67+160 were selected to be critical for both seepage and slope stability analyses based on the severity and type of the failure conditions. The locations of the cross sections are presented in Figure 1b-f. The cross sections were generated using the information obtained from the cross-section surveying, surface seismic geophysical surveys, and boring logs.

The seepage models under the steady-state conditions during the dry season for Sections Km 66+900 and Km 67+160 were established. The maximum negative pore water pressure was assumed to be -120 kPa in the models representing a dry condition of the soil/bedrock based on the typical value of the soil/bedrock at the site. Transient seepage analyses were conducted using the precipitation amounts in Timor-Leste (The World Bank Group, 2015) shown in Figure 2. This figure indicates that the dry season starts around May and June, and the wet season is from December to March. The transient seepage models were conducted for one year starting in June (beginning of the dry season). Transient seepage analyses were also conducted using the current slopes to obtain the pore water pressure conditions for conceptual repair measures in slope stability analyses.

The soil properties used in the seepage modeling are presented in Table 2. They were obtained from either laboratory tests or typical values shown in the literature. The soil water characteristics curves (SWCC) for the soil/bedrock were generated using the results of grain size distributions of the materials. The hydraulic conductivity functions for the soil/bedrock were

| Location No. | Station No. (Type) | Failure conditions |
|--------------|-------------------|--------------------|
| 1            | Km 63+040 to Km 63+200 (Fill) | Erosional failure, culvert box filled with soil/rock debris after heavy rainstorm; A separation on the drainage structure at the downstream side |
| 2            | Km 63+780 (Fill) | Tension cracks and several longitudinal cracks on road due to slope movement after intensive rainfall |
| 3            | Km 65+080 (Rock) | Box culverts blocked with rock/solution debris on the upstream side after heavy rainfall |
| 4            | Km 65+180 (Rock) | Damaged retaining wall at toe of rock slope after heavy rainfall |
| 5            | Km 65+670 (Rock) | Blockage of ditch and drains from eroded material, pot holes formed by falling rocks, erosional failure and rock fall of highly weathered schist after intensive rainfall |
| 6            | Km 66+800 (Rock) | Damaged quadrails at the shoulder of the road in the rock slope side due to highly weathered schist |
| 7            | Km 66+900 (Rock) | Eroded material and pathway due to old cut slope failure and erosional surface of highly weathered schist slope after heavy rainfall |
|              | Km 66+900 (Fill) | Crown cracks on circular orientation and tension cracks due to slope failure after heavy rainfall |
| 8            | Km 67+160 (Rock) | Slope wash and surficial erosion after intensive rainfall |
|              | Km 67+160 (Fill) | Slip failure along an old failure plane after heavy rainfall |
| 9            | Km 67+870 (Rock) | Erosional failure of moderately to highly weathered schist after heavy rainfall |
|              | Km 67+870 (Fill) | Tension cracks and vertical displacement due to slip failure after intensive rainfall |
| 10           | Km 68+100 (Rock) | Cut slope failure and erosional failure of highly weathered schist |
|              | Km 68+100 (Fill) | Blocked ditch at toe of slope after heavy rainfall |
|              |                   | Reoccuring slope failure of slip failure after intensive rainfall |

Table 1. Summary of failure conditions of ten failure locations
generated using the Fredlund and Xing equation (Fredlund and Xing, 1994) along with the saturated hydraulic conductivity values shown in Table 2 and the soil water characteristic curves.

5.1.3 Results of analyses
Figures 3a and 3b show the pore water pressure contours for the fill and rock slopes, respectively, at Section Km 66+900. The pore water pressure contours for the fill and rock slopes at Section Km 67+160 are presented in Figures 3c and 3d, respectively. The pore water pressure contours shown in these figures are the pore water pressure conditions that are the most critical for slope instability. The dates for the critical condition are noted in the figures and range from 10th November to 10th February. These figures indicate that the most critical condition happened in the wet season, which is consistent with the period when the slope failures were observed. The results of these models were used to evaluate the stability of the slopes during the wet season.

5.2 Slope stability analyses
5.2.1 General
Slope stability analyses were performed using the pore water pressure conditions obtained from the seepage analyses under the steady-state and transient conditions to evaluate the factor of safety of the rock and fill slopes for the as-built conditions.

5.2.2 Method of analyses
The slope stability analyses were performed using the computer program SLOPE/W version 9.0.4 (GeoStudio, 2018). SLOPE/W solves limit equilibrium slope stability problems by several different methods. The Morgenstern-Price method was chosen for this study because it considers both force equilibrium and moment equilibrium.

The slope stability analyses were conducted for both static and pseudo-static loading conditions. A factor of safety of 1.5 was used as a design criterion for the proposed remedial measures under the static loading conditions. A factor of safety of 1.2 was used as a design criterion for the proposed remedial measures under the pseudo-static loading conditions. Coefficients of seismic loads of 0.09 g (horizontal) and 0.045 g (vertical) were used in the pseudo-static loading analyses (Rong et al., 2012).

The soil properties used in the slope stability analyses for the steady-state and transient analyses are presented in Table 3. The soil properties were obtained from laboratory tests, back-calculated values, or typical values shown in the literature.

5.2.3 Results of analyses
As it was stated before, the rainfall-induced failure of the cut and fill slopes repeatedly occurred after heavy rainfall events during the wet season. Therefore, the slopes had a factor of safety of 1.0 or less at the time of the failure. A factor of safety of unity is the minimum factor of safety at which a slope will remain stable. The majority of the slopes contains sand, gravel, and severely weathered rock based on the results of the field

Table 2. Summary of soil properties used in the seepage analyses

| Road Section | Soil Type | Saturated Hydraulic Conductivity, K (m/s) | Porosity |
|--------------|-----------|------------------------------------------|----------|
| Km 66+900    | Fill      | $1 \times 10^{-4}$ (1)                   | 0.21 (2) |
|              | Rock      | $1 \times 10^{-5}$ (1)                   | 0.20 (2) |
|              | Weathered Rock | $1 \times 10^{-5}$ (1)             | 0.39 (2) |
| Km 67+160    | Fill      | $1 \times 10^{-4}$ (1)                   | 0.21 (2) |
|              | Rock      | $1 \times 10^{-7}$ (1)                   | 0.18 (2) |

Notes: (1) Assumed typical values (2) Calculated values or typical values
Table 3. Summary of strength parameters used in the slope stability analyses

| Road Section | Soil Type              | Total Unit Weight (kN/m\(^3\)) | Angle of Internal Friction, \(\phi'\) for Sat. Soil (degrees) | Angle of Internal Friction, \(\phi_b\) for Unsat. Soil (degrees) | Cohesion (KPa) | UCS (MPa) | GSI |
|--------------|------------------------|---------------------------------|-------------------------------------------------------------|-------------------------------------------------------------|----------------|------------|-----|
| Km 66+900    | Fill/Sand/Gravel       | 26.4 (1)                        | 40 (2)                                                      | 27 (3)                                                      | 0              | -         | -   |
| Km 66+900    | Rock                   | Impenetrable bedrock in model   |                                                             |                                                             |                |            |     |
| Km 67+160    | Fill/Sand/Gravel       | 26.4 (1)                        | 25 (2)                                                      | 17 (3)                                                      | 0              | -         | -   |
| Km 67+160    | Rock                   | 29.61                           |                                                             |                                                             | 22.4 (1)       | 20 (1)    |     |

Notes: (1) Values obtained from laboratory tests
(2) Back-calculated values
(3) Assume that \(\phi_b = 2/3 \phi'\)

Fig. 3. Pore water pressure contours at the most critical for slope instability of a) fill slope at Section Km 66+900; b) rock slope at Section Km 66+900; c) fill slope at Section Km 67+160; and d) rock slope at Section Km 67+160.

investigation. Thus, the soil/bedrock properties for the sand, gravel, and severely weathered rock were back-calculated in SLOPE/W to obtain a factor of safety of 1.0 for the slopes. Figures 4 shows the results of the back calculations. A friction angle, \(\phi'\), of 40 degrees was back-calculated for the sand and gravel. A cohesion of 0 kPa was used for the sand and gravel. A friction angle, \(\phi_b\), of 25 degrees was back-calculated for the severely weathered rock (Nelson and Thompson, 1977). The cohesion in residual strength for the residual materials is typically zero. Thus, a cohesion value of zero (0) was chosen for the severely weathered rock in the analyses. These back-calculated \(\phi'\) values are in general agreement with the range of typical values of the
materials obtained from the literature. Consequently, these values shown in Table 3 were used in slope stability analyses for the rest of the analyses.

The pore water pressures generated from the dry season to the wet season obtained from the transient models were used to evaluate the change in the factor of safety of the slopes due to the intensive rainfall events. Figure 5 shows the reduction of the factor of safety with time (from the dry season to the wet season) for the fill and cut slopes at Sections Km 66+900 and Km 67+160. The slope stability results are consistent with the observation of the time of the failure. It should be noted that the as-built slopes at the dry season had an initial factor of safety of about 1.3.

6 CONCEPTUAL REPAIR PLANS

Options for the proposed conceptual repair plans that were analyzed in the slope stability analyses include (1) regrading of the existing slopes, (2) installation of tieback anchors and (3) installation of geogrids. Seepage modeling was also conducted for the current slopes at Sections Km 66+900 and Km 67+160 with the proposed repair plans under the steady-state and transient conditions. The stability of the proposed conceptual repair plans was analyzed using the worst pore water pressures during the year obtained from the seepage analyses to ensure the remediated slopes remain stable during the wet season. The slope stability analyses were conducted for both static and pseudo-static loading conditions.

6.1 Static Loading Conditions

The results of the slope stability analyses for the proposed conceptual repair plans under the static loading conditions were summarized in Table 4. Regrading of the slopes was performed in order to check the effect of slope gradient on the slope stability. Considering the constructability of the road slopes, a 1.5V:1H slope was analyzed with the most critical pore
water pressure conditions obtained from the seepage analyses. The results indicate that the 1.5V:1H fill slope could be still unstable under the worst pore water pressure conditions. The factor of safety of the 1.5V:1H fill slope could be increased to be above unity by placing the fill on a firm bedrock and/or reducing the fill depth.

Slope stability analyses were also conducted with the proposed repair plan with tieback anchors installed onto the fill and rock slopes. Table 4 shows that the tieback anchors could be effective in increasing the stability of the fill and rock slopes to an acceptable level during the wet season.

Slope stability analyses were conducted with the proposed repair plan with geogrids installed onto the fill slopes as well. The results shown in Table 4 indicate that geogrids could also be effective in increasing the fill slope stability to an acceptable level during the wet season.

6.2 Pseudo-Static Loading Conditions
Slope stability analyses were conducted for the proposed conceptual repair plans with the tieback anchors and geogrids under the pseudo-static loading conditions. The results of the analyses under the pseudo-static loading conditions are also shown in Table 4. The table indicates that the fill and rock slopes with the tieback anchors and geogrids could be stable under the pseudo-static loading conditions during the wet season.

7 DISCUSSIONS AND CONCLUSIONS
It was reported that the slope failures repeatedly happened after the intense rainstorm events during the wet season. At some locations, the failure of the slopes occurred again even after the repair of the slopes was made. Moreover, in the rock slopes at some failure locations, the surface of the rock slopes consisted of highly weathered schist. The seepage and slope stability analyses were conducted to evaluate the increase of the pore water pressures during the wet season and the resulting reduction of the slope stability. The results of the seepage analyses demonstrated that the pore water pressure within the fill and rock slopes was significantly increased due to the increase of the infiltration during the wet season. The results of the slope stability analyses indicated that the factor of safety of the fill and rock slopes was reduced to unity or less during the wet season. The increase of the pore water pressures in the

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**Table 4. Summary of factor of safety for conceptual repair plans**

| Road Section | Slope Type | Regrading 1.5V:1H | Tiebacks | Tiebacks w/ Seismic Load | Geogrids | Geogrids w/ Seismic Load |
|--------------|------------|-------------------|----------|--------------------------|----------|--------------------------|
| Km 66+900    | Fill Slope | 0.81              | 1.52     | 1.01                     | 1.47     | 1.29                     |
|              | Rock Slope | -                 | 1.50     | 1.29                     | -        | -                        |
| Km 67+160    | Fill Slope | 1.00              | 1.50     | 1.14                     | 1.57     | 1.26                     |

Fig. 5. Reduction of factor of safety with time for the fill and rock slopes at Sections Km 66+900 and Km 67+160.
fill and rock slopes during the wet season had reduced the shear strength of the fill materials and the weathered rock and caused the instability of the fill and rock slopes.

For the design of roadway in an area that is susceptible to a rainfall-induced slope failure potential, it is recommended that residual strength of the soil be used for design of the slope stability for a long-term condition. In the geotechnical engineering practice, it is common that cohesion of clay soil is used in the design. The cohesion of the clay soil mainly comes from the cementation and soil suction of the soil. The soil cementation is the product from the chemical bonding of soil particles in a very low level of shear strain. The bonding of the soil particles could be broken if the soil experiences a significant strain. The soil could become saturated during intense rainfall events. The cohesion attributed from soil suction could reduce to near zero. Consequently, it is recommended the cohesion of the clay soil be used cautiously when the design of slope in an area with a rainfall-induced slope failure potential.

It is also recommended that the current weathering condition for the existing rock slope and a potential weathering condition for the rock slope at the end of design life of the structure be considered in the design stage since the weathering of rock can significantly reduce the shear strength of the rock.

Maintenance of the roadway plays an important role to maintain the performance of the roadway and to avoid unexpected conditions which may result in the reduction of a lifetime of the road. The maintenance work that should have completed at this study site include (1) sealing of cracks on the pavement surfaces; and (2) clean up the drainage to avoid the blockage of drainage structures due to the product of debris flow. The lack of the maintenance of the road further reduced the stability of the slopes and the lifetime of the road.

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