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Pit slope design in phyllites for the Simandou large open pit project

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Abstract

At the Simandou iron ore project site in Guinea, phyllites with varying degrees of strength and alteration are prevalent. Their presence makes it difficult to adequately characterise, classify and estimate the strengths of the rock masses. The foliation in the phyllite has low strength, and thus induces anisotropic behaviour. This paper will present information on how these difficult-to-classify rock mass units were treated in the design of the Simandou open pit.

In order to geomechanically characterise the Simandou rock masses, significant attention was paid to the weathering and alteration processes of the phyllite. The strength of the rock masses closely correlated to the degree of alteration and weathering; the most weathered phyllites had the weakest strength.

The strength of the more altered rock mass used for the pit slope design was estimated from a combination of laboratory rock testing and back-analysis. The back-analysis was based on slope instabilities that had occurred on natural slopes in the Simandou area.

The geotechnical risks, most likely to arise, were identified and taken into account in the pit slope design. Since toppling was the most serious of these risks, the paper discusses it in greater detail. Parametric numerical analyses of toppling were performed to evaluate the importance of the variation in the strength and stiffness of the materials and the likely mode of failure, where toppling or shear (rotational-type failure) could develop.

1 Introduction

The Simandou Iron Ore Project is located in the Guinée Forestière and Haute Guinée regions of Guinea. It comprises two iron ore deposits named Pic de Fon and Oueleba. The deposits are located approximately 4 km apart at the southern end of the Simandou mountain range. Both deposits are approximately 7.5 km in length and up to 1 km wide with current planned maximum final pit depths of 330 and 250 m for Pic de Fon and Oueleba, respectively.

This paper discusses the approach used to characterise the varying grades of weak to moderately strong phyllites rock masses in order to design the pit slopes for the Oueleba deposit. A comprehensive geotechnical investigation was carried out. The geology of the Simandou Range consists of an upper and a lower banded iron formation (BIF), and a sequence of deformed itabirites, phyllites and quartzites. The investigation found that Oueleba's final walls would comprise phyllites in varied states of alteration. The alteration states were as follow:

1. Soil strength (residual) phyllites (PHS).
2. Very weak phyllites (PHV).
3. Weak phyllites (PHW).
4. Fresh to slightly weathered unaltered compact phyllites (PHC).

From the above naming of the phyllite alterations, it can be seen that strength is proportional to the degree of alteration. The challenge for the pit slope design was how to geotechnically characterise the phyllite rock masses and take into account their alteration, foliation and strength anisotropy.

As a consequence of the foliation weakness planes in the phyllites, toppling was identified as serious risk in the pit slope design. Attempts were made to estimate the strength of the intact rock and the discontinuities as accurately as possible based on both laboratory testing and back-analysis. The back-analysis was conducted for old landslides observed in the area. The paper discusses both the laboratory testing and the back-analysis. It also presents details on the numerical modelling of the potential toppling mechanism in the pit walls.

2 Engineering geological model (EGM)

An EGM was developed to form the basis of the slope design for the Simandou project. Components of the EGM included the geological characteristics of the phyllites, their mechanical properties, structures and hydrogeological conditions. As stated earlier, the degree of alteration at Simandou significantly influences the geotechnical characteristics of the phyllites and therefore affects the classification and strength characterisation of the weak Simandou rock masses.

As the final walls of the Oueleba pit would be formed in these phyllites, it was necessary to understand the alterations. Two prominent forms of alteration occur at Simandou:

1. Laterisation (which generally starts from the surface and reduces downwards).
2. Leaching of silica.

For the phyllites, the leaching of silica has had a greater impact on geotechnical properties than laterisation.

The classification and strength characterisation of weak rock masses should have an approach that takes into consideration the strength and degree of alteration of the units. Currently, there is no acceptable approach and classification system that can be directly applied to weak rock masses. As a result, the classification and strength estimation of the rock mass had to be adapted based on distribution of alteration, resulting in following two major geotechnical domains:

- Altered Domains: Comprise the altered phyllites that are expected to behave as a 'homogeneous mass' with failure governed by intact strength (though most domains still retain structural fabric), and with the strength estimated by the Mohr–Coulomb failure criterion. For these materials, no rock mass classification was performed on drill core and they were simply treated as soil, with the laboratory soil strength test results used to represent their mass strengths. However, the influence of foliation on material behaviour and strength anisotropy was found to increase with decreasing alteration.
- Compact Domains: Comprise the compact units with rock mass behaviour. These units were stronger and their strengths were represented by Hoek–Brown intact rock strength parameters σ_{ci} and m_i (Hoek et al., 2002). RMR_{76} parameters (Bieniawski, 1976) were collected through geotechnical logging of the drill core and then related to the Geological Strength Index (GSI) for downgrading to rock mass strength. For these compact domains the stability of slopes is expected to be affected by either structurally controlled (foliation) failures or by composite controlled failures which combine failure along structures with failure through intact rock. To account for anisotropic behaviour, weaker strength was specified along the foliation plane directions.

Details on how the two domains were characterised are provided in the following section.

2.1 Geological considerations

To understand the alteration process of the phyllites, the geotechnical and geological teams worked closely together. Hobbs and Jeffcoate, (2010) describe the development of the altered phyllites at Simandou as starting with the leaching of the silica from the phyllites. This caused a progressive weakening of the phyllites. According to Hobbs and Jeffcoate, (2010), the alteration process of the phyllites (which has been broken down into four main stages) can be shown graphically (Figure 1) and described as follows:

- Stage 1a: fresh to slightly weathered compact phyllite (PHC).
- Stage 1b: alteration of compact phyllite to weak phyllite (PHW). The alteration comprises changing of micaceous phyllite into kaolinitic phyllite. These units have R2 rock strength.
- Stage 2: alteration of weak phyllite to very weak phyllite (PHV). The process involves the progressive leaching of free quartz. These units have R1 rock strength.
- Stage 3: alteration of the very weak phyllite (PHV) to soil strength phyllite (PHS). The alteration process involves the total loss of free quartz. The PHS is described as soil strength kaolinite clay.
- Stage 4a: additional alteration that strengthens the PHS. Under this alteration kaolinite converts (both partially and fully) into gibbsite, via leaching of silica from the mineral structure. The gibbsite mineral is stronger than kaolinite. This process alters PHS into PHV and PHV into bauxite (BAX).
- Stage 4b: alteration in which secondary goethite cement is added to kaolinitic phyllite, when in proximity to mineralisation. Under the process PHS is first strengthened to PHV and then to PHW. As the goethite mineralisation increases the PHV is laterised.
- Stage 4c: alteration of bauxite (BAX) via laterisation, with goethite cementation. This creates hard laterite.

Photographs of drilled core, representative of the four phyllite alteration classes are provided in Figure 2.

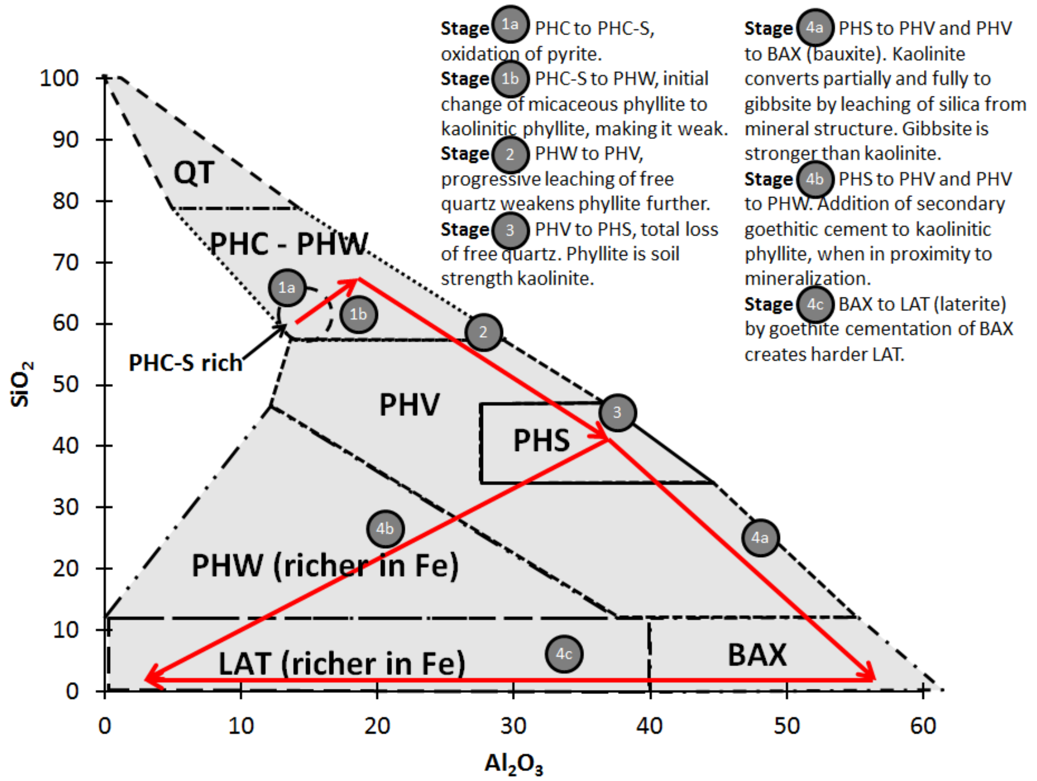

Figure 1 Alteration domains of RC holes using aluminium (AL) versus Silica (Si) contents. Domains are partially calibrated from exploration holes (after Hobbs and Jeffcoate, 2010)

PHS representing Stage 3 alteration

PHV representing Stage 2 alteration

PHW representing Stage 1b alteration

PHC representing Stage 1a

Figure 2 Images of phyllites in various stages of alteration

The four sub-divisions of phyllite (PHS, PHV, PHW and PHC) are summarised in Table 1 according to standard geotechnical descriptions (in particular the ISRM 1981 classifications of rock hardness and alteration). This approach made it easier to identify the various altered states in the field during core logging.

Table 1 Descriptions of the phyllite based on the strength and degree of alteration (Hobbs and Jeffcoate, 2010)

Geodomain	Description	Rock Index (ISRM, 1981)	UCS (MPa)	Degree of Alteration (ISRM, 1981)
PHS	Soil and soil like material	R0 and less	0.25–1	A5–A6
PHV	Very weak phyllite material	R 1	1 to 5	A4–A5
PHW	Weak phyllite material	R 2	5 to 25	A3–A4
PHC	Compact phyllite material	R 3	25+	A2

Based on this classification, the PHS, PHV and PHW materials were assigned to the Altered Domain, represented more by soil-type behaviour, while the PHC was allocated to the Compact Domain.

3 Strength parameters

As has been discussed, the geotechnical behaviours of the phyllites differ according to degree of alteration. The more altered the material, the more the behaviour could be predicted using conventional soil mechanics principles and considering its intact strength. On the other hand, the less altered the material the more it could be treated from rock mechanics perspectives.

Different laboratory tests were done for materials of the two geotechnical domains. Testing of the Altered Domain materials consisted of soil index tests such as grain size distribution and Atterberg limits (for classification purposes), and both single-stage and multi-stage consolidated-undrained (CU) triaxial testing for strength. For the PHC, laboratory rock testing comprised uniaxial (UCS) and triaxial compressive strength tests, and Brazilian indirect tensile strength tests.

The results of the soil classification index tests are presented in Table 2. The Altered Domain materials were classified as ML to MH (sandy silt with fines) to SM (silty sand with fines) according to the Unified Soils Classification System (USCS). The sand content was uncharacteristically high for the PHV, which derived from stage 2 alteration of PHW, with the progressive leaching of the free quartz.

Table 2 Soil classification index test results

Geodomain	Grain Size Distribution				Atterberg Limits		USCS Classification
	% Gravel	% Sand	% Silt	% Clay	Liquid Limit (%)	Plastic Limit (%)	
PHS	13	25	53	9	52	38	MH
PHV	5	59	33	3	49	37	SM
PHW	5	28	62	5	47	35	ML

PHS was assessed in the field and confirmed in the laboratory as having strength of R0 (i.e. UCS<1 MPa). In the field rock mass parameters were collected for the PHV, PHW and PHC. This included estimates of field strength (which matched laboratory results), rock quality designation (RQD), and discontinuity characteristics. However rock mass ratings were only applied to the PHC unit.

Both field and laboratory observations led to the conclusion that the influence of the foliation on strength of the phyllite increased with decreasing alteration. Failure surfaces from triaxial testing observed on the PHS, and to a lesser extent in the PHV samples did not follow any pre-defined plane of weakness, such as foliation planes. However, depending on the direction of the applied test loading, failure surfaces generally followed the foliation in the stronger PHW and PHC rock units.

P–Q plots from the consolidated-undrained triaxial stages for PHS, PHV, PHW were generated. (P is $(\sigma_1+\sigma_3)/2$ and Q is $(\sigma_1-\sigma_3)/2$, while σ_1 is the failure load and σ_3 is the confining pressure used in the triaxial test). These plots were used to estimate the Mohr–Coulomb strength parameters of cohesion and friction angle (c and ϕ).

The fresh to slightly weathered compact phyllite (PHC) was a moderately strong rock with a light grey colour. It was fine grained and with thin to medium siliceous bands. Based on statistical assessment of the rock mass parameters, the PHC geodomain was classified as having a fair rock mass quality rock mass rating (according to RMR_{76} or GSI) of 55. The intact Hoek–Brown (H–B) (Hoek et al., 2002) parameters from the laboratory tests were σ_{Ci} equal to 26 MPa, m_i equal to 9.2 and σ_{Ti} equal to 2.8 MPa. Both saw cut and natural discontinuity shear strength testing were carried out on the PHC. The friction angle for the natural discontinuities averaged around 18° and was less than that for the saw-cut discontinuities (which averaged about 27°). This was attributed to the natural phyllite sample surfaces being coated in fine silt or clay. The foliation in the phyllite was considered to be very persistent.

The strength parameters for the phyllites, considered in the pit design, are summarised in Table 3. The internal angle of friction for all the phyllite domains varied only between 23 and 25°. This could possibly be due to the removal of the siliceous minerals. Table 3 also includes estimates of the cohesion and friction angle for the PHC from the Hoek–Brown parameters, just to compare the reduction in cohesion with increasing degree of alteration. Table 3 shows that the cohesion of the PHC (A2) reduces to 17% and 10% of the value obtained when the degree of alteration increases to A3 (PHW) and A4 (PHV), respectively. This relationship is illustrated on Figure 3.

Table 3 Phyllite design parameters

Geodomain	Description	Cohesion (kPa)	Internal Angle of Friction (degrees)	Comments
PHS (A5–A6)	–	30	23	Derived from laboratory testing
PHV (A4–A5)	Perpendicular to foliation	100	24	Derived from laboratory testing
	Parallel to foliation[1]	39	23	
PHW (A3–A4)	Perpendicular to foliation	150	25	Derived from laboratory testing
	Parallel to foliation[2]	75	25	
PHC (A2)	Perpendicular to foliation	840	42	Derived from Hoek–Brown failure criterion for a confinement of 2 MPa
	Parallel to foliation	230	28	

[1] Value obtained from direct shear test results, which corresponds to about 40% of the strength perpendicular to the foliation.
[2] Assumed value as 50% of the cohesion for the direction perpendicular to the banding, recognising that the degree of alteration / weathering is lower compared to the friable materials.

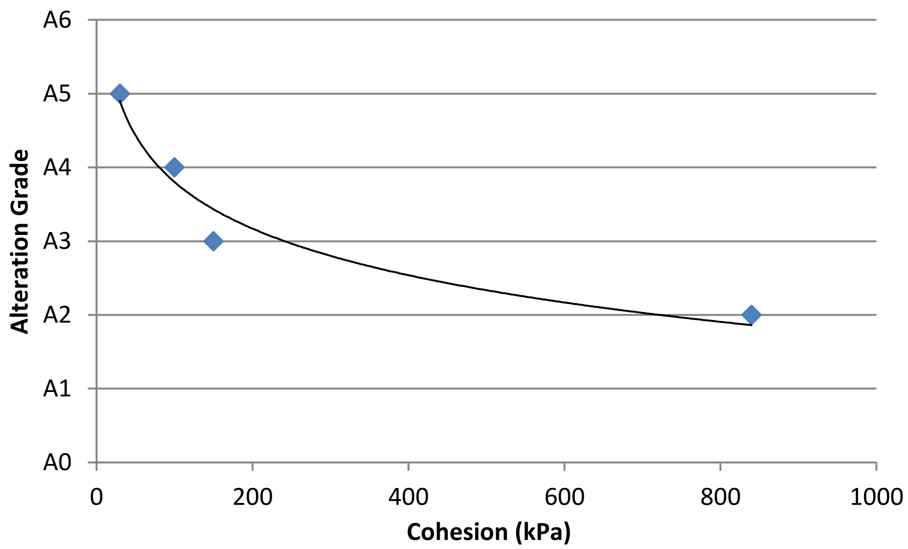

Figure 3 Cohesion versus alteration grade in phyllite

3.1 Back-analysis of existing landslides of natural slopes

Strength parameters for mainly the PHS and PHV were further calibrated from back-analysis of existing landslides on natural slopes, where the failure surface was interpreted to have developed in altered phyllites. This work was carried out by Fred Baynes (Baynes Geologic, 2012) and by Rio Tinto (Creighton, 2013, written comm.). The natural slope geometry prior to failure was estimated from adjoining topographical sections. From this assessment most of the natural slopes had slope angles ranging from 20 to 25°, and most of the failures occurred in the altered phyllites. A series of back-analyses was carried out to establish what combination of shear strength parameters would result in a Factor of Safety of 1.0 (which represents the equilibrium condition or the condition for landslide failure) for a 20 and a 25° slope. In the analyses the groundwater table was varied from being ‘at surface’ and to ‘5 m below surface’ (these were deemed reasonable for the Simandou locale). The combinations of shear strength parameters cohesion and phi from these analyses were plotted in Figure 4. The blue vertical and horizontal lines represent the ranges of laboratory test results for friction angle and cohesion, respectively. The other coloured lines and bands encompassed the cohesion-friction angle pairs from the back-analyses. (Pictures of an example landslide are shown on Figure 5.)

The majority of the area of the bands lies within the rectangular region bounded by the range of laboratory results. This demonstrates that the shear strength parameters derived from the laboratory results mirrored the 'average' conditions at the time the landslides occurred. This partly contributes to the belief that failure in the PHS and PHV units (with field strength estimates of R0 to R1) could be expected to be primarily controlled by the intact material strength, and not planes of weakness.

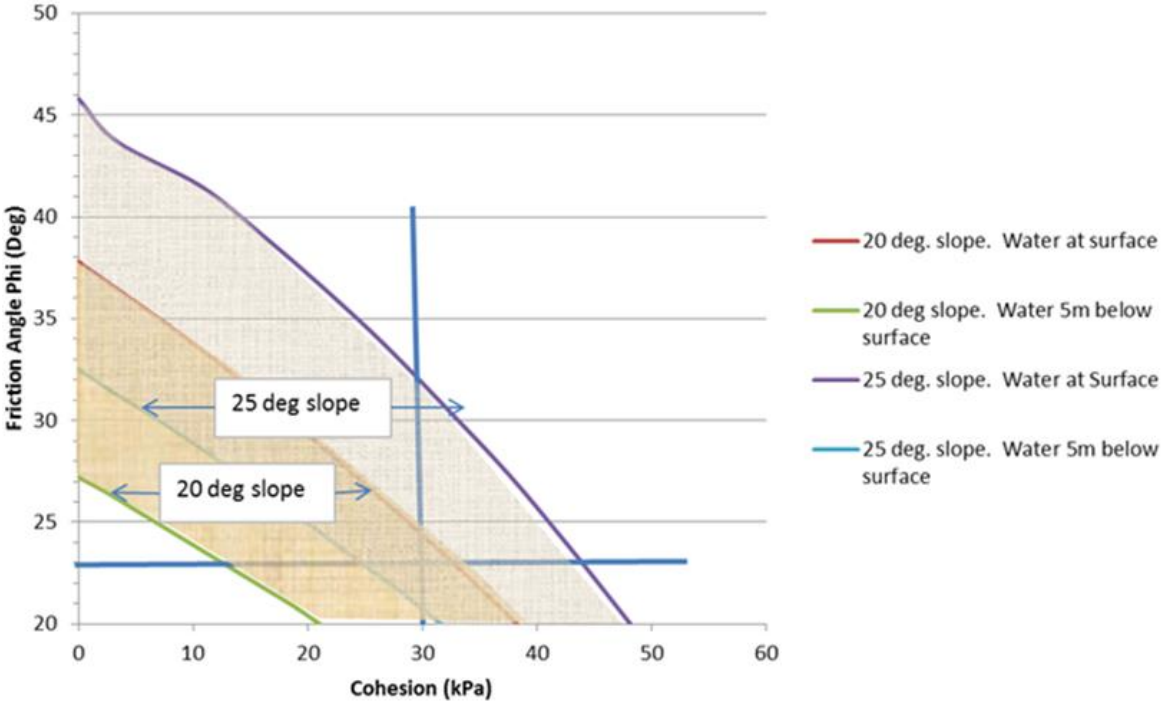

Figure 4 Back-analysis outcomes for natural slopes cohesion and friction angle (Creighton, 2013, written comm.)

(a) (b)

Figure 5 Examples of landslides interpreted; (a) interpreted landside indicated with an arrow; (b) detail of the landslide indicated in (a)

4 Slope design approach

Limit equilibrium methods and finite element analysis (shear strength reduction method) were used to design the overall slopes of the Oueleba pit. The limit equilibrium program, SLIDE v6 (Rocscience, 2013b), was primarily used to assess the stability of the slopes to be developed in the altered phyllites. The stability assessment of the PHS was carried out with the material assumed to behave as a 'homogeneous mass', with the strength estimated by the Mohr–Coulomb failure criterion. As the alteration decreased and strength increased from PHS through to PHC rock masses, the influence and persistence of foliation became more apparent and accompanying anisotropic behaviour was considered.

In the limit equilibrium models, the influence of foliation was assessed using the Generalised Anisotropic Strength model. This strength models allowed for explicit accounting of the:

1. Reduced strength of the foliation relative to intact rock.
2. Relative orientation angle(s) of the foliation in relation to the pit wall.

The azimuth of the wall was considered when applying the anisotropic behaviour: the effect of foliation was included in an analysis only if the strike of the foliation was within ±25° of the wall azimuth. Foliation orientation distribution throughout the pit was mapped by borehole acoustic televiewer survey and surface mapping. Oriented core logging was initially attempted but achieved only limited results due to the fragility of the weathered phyllite.

A Hoek–Brown disturbance factor (D) greater than 0 was only applied to walls in the PHC to account for potential blast damage. A graded D was considered as follows:

- D of 1 for rock mass within a distance of 30 m from the slope face.
- D of 0.85 for rock mass within a distance of 25 m from the 30 m disturbed zone.
- D of 0.7 for rock mass within a distance of 20 m from the 25 m disturbed zone.

The analyses did not consider any blasting damage for the altered materials as they were likely to be excavated mechanically using dozers, excavators, or shovels.

Pore pressures behind the pit walls and beneath pit floors were estimated by Schlumberger Water Services. The predicted groundwater pressures were imported into Slide slope stability models as grids of pore pressure values.

An assessment of the structurally controlled mechanism was investigated through kinematic analyses as the stability of slopes within the PHW and PHC phyllites would be either structurally controlled failures or composite failures formed by a combination of structure and failure through intact rock.

Toppling was identified as impacting the slope design of the phyllites. The factors governing toppling failure included spacing, orientation, continuity and shear strength of the structural sets involved, the strength of the intact material, in particular its tensile strength, and the slope angle.

Sensitivity analyses were performed using the finite element program Phase^2 (Rocscience, 2013a) in order to examine the influence of some of these factors. Figure 6 is an example of one of the results of the numerical models. It displays the contours of maximum shear strain along with an exaggeration of the slope's deformation. The plots help understand the failure mechanism.

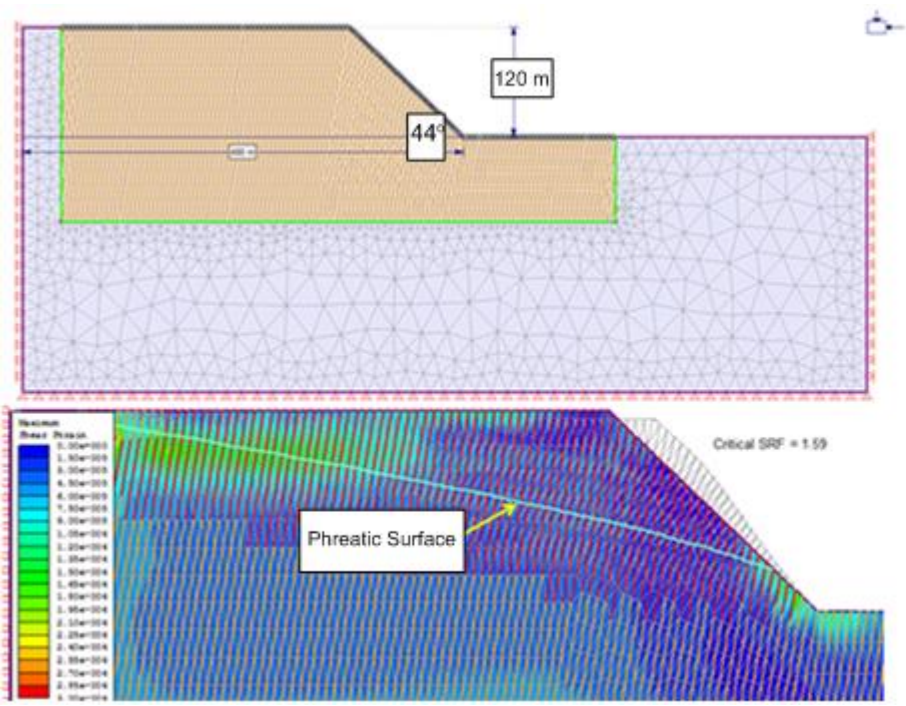

Figure 6 Example toppling model with maximum shear strain contour results and deformed outline of slope

An important question was posed as to whether toppling would in reality be possible in the Altered Domains. To answer this question, a model with the same geometry was run considering dry conditions and with different combinations of intact rock strength, UCS, and Young's modulus, E. Figure 7 below is a plot that helps explain changes in the possible mode of failure. The vertical axis, which is labelled 'Relative Horizontal Displacement', represents the horizontal displacement of the slope at failure from which the maximum horizontal displacement for the starting strength reduction calculations has been subtracted (i.e. additional displacements from the starting shear strength reduction analysis). For Factors of Safety higher than 1 the reference stage is for strength reduction factor of 1.

The chart has two regions roughly demarcated by the broken line. Points in the region above the line accompany toppling failure, while those below represent the rotational-type failure typical of soils. An interesting observation of the chart is that rotational-type failure can occur at any strength level, if Young's modulus is low. Although the broken line was arbitrarily drawn, it nevertheless offers another interesting insight: there were combinations of strength and stiffness that plotted closer to the boundary line and which reflected combined failure mechanisms rather than clear toppling or obvious-rotational-type failure.

This chart shows that including Young's modulus in the discussions has helped clarify the anticipated failure mechanism. The chart on Figure 7 indicated that it was highly unlikely for toppling to occur in the Altered Domain due to their soil-like nature (low stiffness). On the other hand, for the compact zone, which has Young's modulus typical of rocks, toppling failure could occur.

The Phase^2 analysis results also indicated that the potential for flexural toppling within the PHC was low for the maximum inter-ramp slope height of 120 m, with inter-ramp angle of up to 44°.

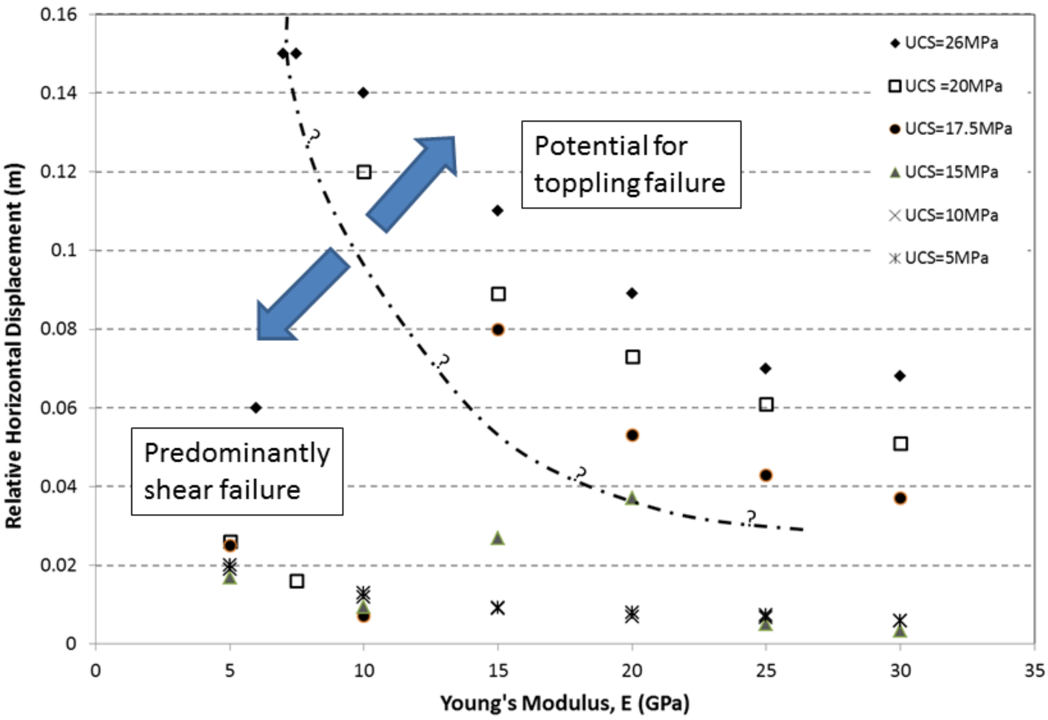

Figure 7 Observed change in the mode of failure as a function of the stiffness and strength of the rock mass

The likely failure mechanisms for the different altered materials are summarised in Table 4 below.

Table 4 Anticipated failure mechanisms for the phyllites

Geodomain	Description	Failure Mechanism
PHS	Soil and soil-like material	Failure mechanism controlled by intact strength
PHV	Very weak phyllite material	Failure governed by varying contributions of relic structure and intact strength
PHW	Weak phyllite material	
PHC	Compact phyllite material	Failure mechanism predominantly influence by strength of foliation with likely mechanism toppling due to persistent foliation dipping into the slope

5 Concluding remarks

As is typical of open pit mining, the rock mass units at Simandou needed to be characterised and then slope angles designed. The challenge at Simandou arose from the weak (phyllite) rock masses present at the site, with various states of alteration. Collection of geotechnical parameters from drill core was very difficult, as the drilling process damaged the core. In addition, currently, there is no rock mass classification system developed for weak rock masses that take into account the degree of alteration and the contribution of the intact material strength.

As a result engineers and geologists worked closely to identify the different rock mass domains based on alteration. These domains were used to define the schedule for the laboratory strength testing and for estimating the strength envelope that would be representative of the mass strength. The Altered Domains were treated as soil material, with the intact material strength, obtained from the laboratory testing considered to be representative of the mass strength. The Compact Domains were treated as rock materials, with the intact rock strength being downgraded by the rock mass classification carried out in drill core.

The influence of the strength anisotropy due to the foliation was observed to reduce with increasing degree of alteration. This is because the more highly altered rocks, such as the PHS and PHV phyllites at Simandou, have become sufficiently weakened that the eventual effect of anisotropy does not alter the already weak material and, consequently it tends to behave as isotropic.

Numerical modelling helped establish that toppling was a potential risk for wall sections in compact phyllite. The risk of toppling reduced with increasing phyllite alteration. In phyllites of intermediate alteration more complex failure mechanisms (that combined toppling with rotational-type failure) could arise. In the highly altered phyllites rotational-type failure was predicted. These trends were highlighted by a chart plotting stiffness (Young’s modulus) and rock strength against relative horizontal displacements.

Recognising that the potential for toppling is aggravated by groundwater pressure, recommendations were provided for installing a depressurising system, as this is recognised to be the most effective remedial technique to control the development of toppling.

The Simandou geotechnical work shows the need to use numerical modelling in addition to limit equilibrium methods for slope design in complex geological materials. Because numerical models take aspects such as material deformation behaviour (Young’s modulus) into account, they are capable of highlighting slope responses which do not neatly fall into one or another form of failure.

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