UNIVERSITY OF THE PHILIPPINES

**ROCK MASS STRENGTH AS LINK BETWEEN GEOLOGY
AND TOPOGRAPHY IN THE CAN-ABAG MOUNTAIN RANGE,
SOUTHERN LEYTE, PHILIPPINES**

By

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DEDICATION

To Equa. For the love, company, and faith in this journey – our journey.

To Sari. For the precious distractions.

To Doc Sandra. For asking me to join your fieldwork at Guinsaugon... and for the valuable learning opportunities since then.

To the Guinsaugon landslide victims and survivors. For the inspiration to pursue this path that I have taken.

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ABSTRACT

ROCK MASS STRENGTH AS LINK BETWEEN GEOLOGY AND TOPOGRAPHY IN THE CAN-ABAG MOUNTAIN RANGE, SOUTHERN LEYTE, PHILIPPINES

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Rock mass strength is used to characterize the influence of geology on the topography of the Can-abag mountain range in Southern Leyte, Philippines. A geologic model comprised of Jurassic to Cretaceous ultramafic rocks (UM), pre-Miocene fine clastic rocks (FC1), Middle Miocene limestones (LS1), Late Miocene to Late Pliocene coarse clastic rocks (CC), Late Miocene to Early Pliocene fine clastic rocks (FC2), and Late Pliocene limestones (LS2) is proposed for the study area. Five litho-structural units are derived from this model for use in the succeeding discussion: coarse clastic rocks west and east of the Can-abag ridge (CCw, CCe), fine clastic rocks (FC), limestones (LS) and ultramafic rocks (UM).

FC and UM, both having weak rock mass strengths, are prone to isotropic slope failures, and are characterized by low-gradient, low curvature terrain. LS has high rock mass strengths, but because of limited topographic relief development, exhibits low-gradient, low curvature terrain similar to the weak rock masses. CCw and CCe both have high rock mass strengths and are characterized by high gradient, high curvature terrain. The former has slightly lower gradient and higher curvature than the latter due to the dominance of channelized transport processes. The latter, on the other hand, is influenced by steeply-dipping planar discontinuities which facilitate episodic massive landslides, which in turn reduce the curvature of the underlying unit.

The weak rock mass strength of FC is mainly due to the inherent weakness of the fine-grained intact rock. In contrast, UM, which has high intact rock strengths due to the crystalline texture, is reduced to a weak rock mass by numerous discontinuities

attributed to the long history of tectonic deformation. CCw and CCe, while having low intact rock strengths just above that of FC, have higher rock mass strengths because of good discontinuity conditions. LS has high rock mass strengths because of high intact rock strengths and good discontinuity conditions. Regional geologic structures such as bedding and fault planes provide potential conditions for planar slope failures within the rock masses. However, failures are spatially limited by the geometric relationships between the geologic structures and the existing topography. Steeply-dipping structures have direct significant contributions to massive landslides in terrains underlain by materials with strong isotropic rock mass strengths, such as in CCe.

The geologic environment – lithology, structures, stratigraphy, tectonic history – provides the material base on which surface processes act on to create the present landform. Rock mass strength, as a geomechanical manifestation of the geologic environment and as a physically-based and quantifiable concept, provided a good framework for understanding and explaining the spatial variability of topographic characteristics in relation to the geologic materials of the Can-abag mountain range in Southern Leyte, Philippines.

Some remarks regarding the use of gradient distributions in rock mass strength characterization, the use of slope gradient in landslide susceptibility analysis, and frequency-magnitude of landslides are put forward.

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

This work puts forward the following research problems: If geology exerts an influence on the topography of the study area, to what degree is this influence, and in what manner? The problem focuses on rock mass strength and slope stability as the linking factors. The author would like to demonstrate how the geologic environment determines the rock mass strengths and slope stability conditions, which, in turn, influence the development of the topography of the study area.

The pursuit of the problem was originally motivated by the results of geologic mapping related to a field course in the Master's program which revealed 1) subregions with distinct topography, and 2) rough correlations between these subregions and the rock types which comprise the underlying formations.

Studies have been conducted to characterize the relationship between material properties and topography in the western USA, New Zealand and in the Swiss Alps (Korup, 2008; Korup and Schlunegger, 2009; Roering et al., 2005; Schmidt and Montgomery, 1995). While these relationships have been elaborated, Korup (2008) adds that future work is needed to calibrate the relationship between actual rock mass strengths and their topographic signature.

1.2 Rationale

This work involves the disciplines of geomorphology and engineering geology. While there had been advances in these interdisciplinary fields, globally-speaking, there are vast opportunities for developing the knowledge and practice here in the Philippines. The tropical climate and relatively lower topographic relief setting in the Philippines has the potential for unique contributions to the existing body of knowledge.

On the practical side, an integrated understanding of geomorphology and engineering geology is recognized as an important component of much needed studies on geologic hazards and risk mitigation. Knowing how properties of underlying materials influence surface processes and forms, and how these materials are broadly controlled by the geologic environment provides primary input for land use planning and hazard zoning on the regional scale, as well as for ground investigations and mitigation measures on the site scale.

1.3 Objectives

The objectives of this work are: 1) to construct a geologic model of the study area that broadly subdivides geologic materials based primarily on lithology and geologic structures; 2) to show that these litho-structural units can be distinguished based on their topographic characteristics; 3) to show that these litho-structural units have distinct rock mass strength characteristics and stability conditions; and 4) to discuss how rock mass strength and slope stability conditions are controlled by geology, and how these influence the topography.

1.4 Methods

Geologic mapping is conducted to construct the geologic model and delineate the litho-structural units. Visual and statistical analyses of topographic data are used to distinguish the litho-structural units. Field- and literature-based characterization of rock mass strength properties are conducted, results of which are used in slope stability analysis. Slope stability conditions are analyzed using 2-dimensional circular and planar slope failures of the rock mass. Rock mass strengths and stability conditions are also used to distinguish the litho-structural units.

Figure 1. Location map of study area. (a) Philippines and Leyte Island (black). (b) Southern Leyte, and municipalities of Libagon (Lb), Saint Bernard (SB) and Liloan (LI) which cover the study area. Coordinates in Latitude-Longitude, WGS84. Vector data of administrative boundaries from *Global Administrative Areas (version 2.0)* (2012).

1.5 Study Area

The study area is located in the province of Southern Leyte, island of Leyte, Philippines (Fig. 1), between $10^{\circ} 9'$ and $10^{\circ} 26'$ North and between $125^{\circ} 0'$ and $125^{\circ} 9'$ East. It covers the municipalities of Libagon, Liloan and St. Bernard. From the capital city of Manila, the area is accessible via a plane ride to Tacloban City, Leyte, and then by a bus or van ride, which usually takes around 3 to 5 hours.

1.6 Outline

The main body of this work consists of chapters on geology, topography and rock mass strength, following the general sequence of objectives and methods presented earlier. A fourth chapter integrates the results and discussion of the previous chapters.

The chapter on geology reviews the existing literature on the geology of the study area, and presents the results of the geologic mapping conducted for this research. From the literature and the fieldwork results, a geologic model subdividing the area into distinct litho-structural units is formulated and presented. These units will be used in the succeeding chapters.

The chapter on topography discusses literature on the characterization of topography, as well as inferences on surface processes and material properties based on topography. Results are presented according to the different topographic parameters investigated, namely elevation, gradient, curvature and morphometric features. Litho-structural units are distinguished based on these results. Qualitative description of inferred geomorphic processes and material properties for each of the units, based on these results, are proposed for further discussion in the last chapter.

The chapter on rock mass strength reviews failure mechanisms and strength criteria for rock masses, basic slope stability analysis procedures, and the limiting role of slope stability on topography. Results of field- and literature- based rock mass strength characterizations of the litho-structural units, the corresponding stability analyses, and validation using a landslide inventory are presented.

The last chapter integrates the results of the geology, topography and rock mass strength chapters. The influence of geology on rock mass strength, and the limits imposed by rock mass strength, in terms of slope stability, on topography are discussed.

CHAPTER 2

GEOLOGY

2.1 Introduction

Topography often reflects the variations in properties of materials at or very near the earth's surface. The main objective of this chapter is to construct a geologic model of the study area that allows for a broad classification of geologic materials based primarily on lithology and structures. This model will provide the basis for investigating variations in topography and rock mass strength (later chapters), which will help explain the influence of lithology and structures on the topography of the study area.

The specific objectives of this chapter are 1) to identify and characterize lithologic formations and geologic structures in the area, 2) to construct a stratigraphic and structural model, and 3) to subdivide and characterize the study area based on lithology and geologic structures.

2.2 Review of Literature

2.2.1 Tectonic Setting

2.2.1.1 Overview

Leyte island is located in the eastern side of Central Philippines (Aurelio et al., 1991; Dimalanta et al., 2006). Its present regional tectonic setting (Fig. 2) is influenced by the interaction of the Philippine Sea Plate and the southeastern termination of the Eurasian Plate ((Mines and Geosciences Bureau (MGB), 2010)). These plates subduct westward and eastward, respectively (MGB, 2010), beneath an actively-deforming lithosphere called the Philippine Mobile Belt (Gervasio, 1966 cited in MGB, 2010), to which Central Philippines belongs. The Philippine Mobile Belt is bounded by a collision zone involving the North Palawan Block (Aurelio et al., 1991). Leyte also experiences left-lateral deformation due to the Philippine Fault Zone, which cuts across the island

Figure 2. Regional tectonic setting of Central Philippines. Inset map: Subduction of Eurasian and Philippine Sea Plates (EP, PSP) beneath the Philippine Mobile Belt (PMB) along Manila, Negros, Cotabato, and Philippine Trenches (MT, NT, CT, PT); North Palawan Block (NPB) - PMB collision along Mindoro-Romblon-Panay; Strike slip deformation of PMB along the Philippine Fault Zone (PFZ). Main map: Leyte island and Central Philippines are bound by the the NP-PMB collision and NT to the west, and the PT to the east. Leyte island is traversed by the sinistral PFZ. Image modified from Dimalanta et al. (2006)

(MGB, 2010). Aside from these active features, earlier tectonic regimes have also been recognized or proposed (Aurelio et al., 1991; Yumul et al., 2008; Metal Mining Agency of Japan and Japan International Cooperation Agency (MMAJ-JICA) (1990)).

2.2.1.2 Subduction zones

The eastward subduction of the Eurasian Plate is manifested morphologically by the Manila-Negros-Cotabato Trench System (MGB, 2010). The Luzon, Panay-Negros and Cotabato Volcanic Arcs reflect the activity of this subduction system (MGB, 2010). On the Philippines' eastern seaboard, the Philippine Sea Plate subducts westward beneath the East Luzon Trough and Philippine Trench. However, the active volcanic arc corresponding to this subduction can only be traced clearly from Bicol to Leyte (MGB, 2010). Based on the length of the subducted slab and measured displacement rates, the

initiation of this westward subduction was estimated to have occurred not earlier than 5 mya (MGB, 2010).

2.2.1.3 Philippine Fault Zone

The Philippine Fault Zone (PFZ) is an active deformation zone that cuts the Philippine Mobile Belt from Luzon to Mindanao over a length of no less than 1200km (MGB, 2010). It also passes through Leyte island and the study area. Although it has essentially left-lateral displacement (Barrier et al., 1991; MGB, 2010), some segments demonstrate vertical components of displacement (Makino et al., 2007; MGB, 2010).

In Leyte, the 5-8 km lateral displacement in Plio-Pleistocene sedimentary formations (Barcelona 1981,1986,cited in MGB, 2010) and the 110 km displacement since Middle Neogene between the northeast and southwest Leyte ophiolitic bodies (Cole et al., 1989, cited in MGB, 2010) are attributed to the PFZ. GPS observations suggest modern displacement rates of at least 2.6 cm/year (Catane et al., 2000 and Duquesnoy et al. 1994, cited in Besana and Ando, 2005). Fault data analysis in the Bondoc Peninsula-Masbate-Northern Leyte segment of the PFZ suggest that the structure is no more than 5 million years old (Aurelio et al., 1991).

Aurelio (2000) describes the coupling of deformation along the Philippine Trench and the Philippine Fault Zone as a shear partitioning mechanism (Fitch 1972, cited in MGB, 2010). This was believed to have resulted when the movement of the Philippine Sea Plate relative to Eurasia shifted from a northward to a northwestward direction around 4 mya(Aurelio, 2000).

Historical earthquakes in Leyte island are attributed to the Philippine Fault Zone (Besana and Ando, 2005). The southern segment covering the study area is locked and produces moderate earthquakes (M 6.0 or less) with several days of perceptible foreshocks (Besana and Ando, 2005). Sections of the fault within the study area have shown evidence of activity based on the PHIVOLCS active faults map (Tsutsumi and Perez, 2013).

2.2.1.4 Earlier tectonic regimes

Along the Central Segment of the PFZ in the areas of Bondoc Peninsula, Masbate and Leyte, Middle Oligocene to end-Miocene compressive and tensional events which are older than the the PFZ and Philippine Trench were identified (Aurelio et al., 1991). One of the compressive event was related to the collision of the North Palawan Block and the Philippine Mobile Belt (Aurelio et al., 1991).

The presence of a proto-Southeast Bohol Trench Yumul et al. (2008) and a Sulu-Zamboanga-Masbate Arc (Mitchell et al., 1986, cited in MMAJ-JICA, 1990) suggests a mid-Miocene closing of a marginal oceanic basin and the subsequent convergence of the Central Philippines and Mindanao blocks (Mitchell et al., 1986, cited in MMAJ-JICA, 1990). Also in this model, a proto-Philippine Fault Zone and a proto-Philippine Trench exist prior to this mid-Miocene convergence.

2.2.2 Stratigraphy

2.2.2.1 Overview

The basement unit is composed of Jurassic to Cretaceous ophiolitic rocks mostly outcropping in the southern portion of the study area (Mines and Geosciences Bureau (MGB) (1997); MMAJ-JICA, 1990). This unit is unconformably overlain by the Middle Miocene Danao Limestone in the south and the Early to Middle Miocene Central Highland Volcanics (MMAJ-JICA, 1990; MGB, 1997) in the north. Unconformably overlying these units are the Late Miocene to Early Pliocene Pangasugan (MMAJ-JICA, 1990; MGB, 1997) and Masonting (MMAJ-JICA, 1990) Formations. The Late Pliocene Dolores Formation (MGB, 1997) unconformably overlies all the older units in some areas.

Leynes and coworkers (1996) correlated the Cretaceous Ultramafics and Tertiary Limestone to the ophiolitic rocks and Danao Limestone (MMAJ-JICA, 1990; MGB,

1997). However, they represented the Central Highland Volcanics, and the Pangasugan, Masonting and Dolores Formations as the Tertiary Clastics of Cabac.

A simplified comparative stratigraphy is shown in Figure 3.

2.2.2.2 Jurassic to Late Cretaceous Ultramafic rocks

The ultramafic rocks are identified as serpentinized harzburgite with minor green dunite (MMAJ-JICA, 1990), serpentinized peridotite (MGB, 1997), and serpentinite (Leynes et al., 1996). The rocks are yellowish green to dark green, allotriomorphic-granular, medium- to very coarse-grained, massive (MGB, 1997) and highly fractured (Leynes et al., 1996). Outcrops are found along linear traces of the Philippine Fault Zone and its splays (MGB, 1997), particularly in Barangays Panian, Lipanto and Hindag-an of the municipality of St. Bernard, and in Barangay Ilag of the municipality of Liloan (Leynes et al., 1996). This unit is part of the Late Cretaceous Malitbog Ophiolite, which is presumed to have been emplaced during the same period (MGB, 2010).

2.2.2.3 Middle Miocene Danao Limestone / Early to Middle Miocene “Tertiary Limestone”

The limestones are of coralline-algal type (MMAJ-JICA, 1990), white, porous and locally marly (MGB, 1997). Some are buff- to cream-colored, bedded, fossiliferous and dense but with small vesicles (Leynes et al., 1996). These can be found in Barangays Himbangan and Lipanto in St. Bernard (Leynes et al., 1996), as well as in Barangays Himay-angan, Tabugon and Molopolo in Liloan (MMAJ-JICA, 1990; MGB, 1997). In Gacat, southwest of St. Bernard, the limestones are believed to unconformably overlie the ultramafic rocks (Leynes et al., 1996). Age dates of Middle Miocene from foraminiferal content (Florendo 1987 cited in MGB, 2010) and Early to Middle Miocene from fossil assemblage (Lubas et al. 1996, cited in Leynes et al., 1996) have been established. The unit is interpreted to have formed in a shallow marine environment (MMAJ-JICA, 1990). A minimum thickness of 200m is inferred for this unit (Leynes et al., 1996).

Period	Epoch	Stage	Ma (GTS, 2004)	MGB, 2010; MGB, 1997	MMAJ-JICA, 1990		Leynes et al., 1996
Quaternary	Holocene		0.0115	Alluvium	Alluvium		Alluvium
	Pleistocene		1.81				
Neogene	Pliocene	Late	3.6	Dolores Fm. ¹ / Inopacan Clastics ²	Masonting Fm. ²	Pangasugan Fm. ¹	Tertiary Clastics of Cabac
		Early	5.33	Pangasugan Fm. ¹ / Masonting Fm. ²			
	Miocene	Late	11.61				
		Middle		Calubian Limestone ¹ / Danao Limestone ²	Danao Limestone ²	Central Highland Volcanics ¹	Tertiary Limestones
				Baybay Volcanic Complex / Central Highland Volcanics ¹			
		Early	15.97				
Paleogene	Oligocene		23.03				
	Eocene		33.9				
	Paleocene		55.8				
Cretaceous	Late		65.5				
	Early		99.5	Malitbog Ophiolite	Serpentinized harzburgite of Malitbog Ophiolite		Cretaceous ultramafics
Jurassic			145.5				

¹Pilac, 1965²Florendo, 1987³Corby et al., 1951

Figure 3. Comparative stratigraphy of study area (MGB, 2010; MGB, 1997; MMAJ-JICA, 1990; Leynes et al., 1996).

2.2.2.4 Early to Middle Miocene Baybay Volcanic Complex / Central Highland Volcanics

The Central Highland Volcanics (CHV) consist of porphyritic hornblende pyroxene andesite with phenocrysts of hornblende, pyroxene and plagioclase (MMAJ-JICA, 1990), basalt and dacite flow breccia (MMAJ-JICA, 1990). Found in the rugged highlands of central Leyte (MGB, 2010), it extends southward to include the Can-abag mountain range (MMAJ-JICA, 1990; MGB, 1997). This unit, now renamed as Baybay Volcanic Complex, is presumed to have been emplaced during the Early to Middle Miocene (MGB, 2010). The Pliocene age of the Philippine Trench (MGB, 2010) suggests a pre-Philippine Trench subduction system as the source of this volcanic complex.

2.2.2.5 Late Miocene to Early Pliocene Pangasugan Formation

This unit consists of conglomerate intercalated with volcanic agglomerates and breccia, and tuff (MMAJ-JICA, 1990; MGB, 2010). There are occasional lenses of coarse sandstone and tuffaceous shale (MGB, 1997). The andesitic fragments in the conglomerate suggest that this unit was derived from the CHV (MGB, 2010). The unit is generally massive, poorly bedded, poorly sorted but well compacted; has a total thickness of 1200-1500 meters; and is intruded sporadically by andesitic dikes and sills (MGB, 2010). Without the benefit of fossil evidence, a Late Miocene to Early Pliocene age was inferred (Pilac1965, cited inMGB, 2010). Pangasugan formation is found widely in the Leyte Central Highlands (MGB, 2010), and at the western flanks of the CHV in the municipality of Libagon (MMAJ-JICA, 1990; MGB, 1997).

2.2.2.6 Late Miocene to Early Pliocene Masonting Formation

Marl, calcareous ash bed, pumice, volcanic sandstone and andesitic volcanoclastics comprise the Masonting Formation(MMAJ-JICA, 1990). It is dated Late Miocene to Early Pliocene (Florendo 1987,cited in MMAJ-JICA, 1990). It underlies the areas

of Barangays Ilag and Himay-angan of Liloan and Barangay Lipanto of St. Bernard (MMAJ-JICA, 1990).

2.2.2.7 Late Pliocene Dolores Formation

This unit consists of “pyroclastic materials and beds of conglomerate, sandstone, shale and limestone”, with “limy tuffaceous shale with thin bedded sandy and impure limestones at the bottom” (MGB, 1997). It is finer and better sorted than the Pangasugan Formation (MGB, 2010). A Late Pliocene age is determined from nannofossil content (MMAJ-JICA, 1990). This unit is referred to as the source lithology of the Guinsaugon landslide composed of a thick sequence of southwest-dipping beds of sandstones, breccias and mudstones (Catane et al., 2008; Futralan et al., 2010).

2.2.2.8 Middle Miocene to Pliocene “Tertiary Clastics of Cabac”

This unit is exclusively discussed in Leynes et al. (1996). It is made of a lower conglomerate member, a middle turbidite member and an upper limestone member. The lower conglomerate is exposed in Barangays Nailong to Mayuga in the municipality of Libagon. It is polymictic, with clasts of sandstone, serpentinite and two-pyroxene basalts.

The middle turbidite outcrops in Barangays Marangay to Himay-angan in Liloan. It has a basal conglomerate, and a repeating sequence of poorly consolidated sandstones, siltstones and well-indurated sandstones. The conglomerate has sandstone and volcanic clasts, whereas the well-indurated sandstone contains lithics (serpentinite, altered andesites, dacites), crystals of plagioclase, hornblende, biotite, and quartz, and recrystallized foraminifera and shells. It is estimated to be 1750m thick.

The upper limestone can be found from Himay-angan to Tabugon in Liloan. It consists of limestone breccia to calcirudite. Fossiliferous units contain volcanic lithics and crystals. An age of Middle Miocene or younger was determined from the fossil assemblage.

2.2.3 Geologic Structures

2.2.3.1 Beds

Leynes and coworkers (1996) reported the turbidite member of the Tertiary Clastics of Cabac to be dipping toward the SE in Marangay, and becoming almost horizontal towards Himay-angan in Lipanto. At the source area of the Guinsaigon landslide, the beds are dipping in the SW direction (Catane et al., 2008; Futralan et al., 2010). While there were general descriptions of the beds of the Pangasugan and Dolores Formations (MMAJ-JICA, 1990; MGB, 2010), it is not clear whether those measurements were taken within the study area.

2.2.3.2 Folds

An eroded anticline, with a N-S axis was interpreted from aerial photographs of the area from Libagon town proper to Magcasac, in Libagon Leynes et al. (1996). A similarly-oriented syncline was inferred from bed readings in Barangay Ilag in Liloan (MGB, 1997).

2.2.3.3 Unconformities

In NE-SW cross sections, the unconformable surfaces between the Central Highland Volcanics and the Ultramafic rocks (MMAJ-JICA, 1990); between the Pangasugan Formation and Central Highland Volcanics (MMAJ-JICA, 1990; MGB, 1997); and between the Dolores Formation and other older units (MGB, 1997) all dip in a general SW direction.

2.2.3.4 Faults

A normal fault was mapped as the contact between the Tertiary Limestone and Cretaceous Ultramafics in Lipanto (Leynes et al., 1996). This likely corresponds to the N-S trending linear contact between the said rocks in the geologic map of MGB (?).

Numerous works mention thrust faults involving the ultramafic unit. Thrust faults strike N15-20E in the sheared ultramafic rock mass in Lipanto (Leynes et al., 1996). Cross sections (Leynes et al., 1996, MGB, 1997) illustrate a northwest- to west-directed thrusting of ultramafic rocks over the Tertiary Clastics of Cabac and the Central Highland Volcanics, respectively. At the regional scale in the central Philippines, the ultramafic component of ophiolites are thrust slabs of peridotite (Florendo 1984, cited in MGB, 2010), specifically over Tertiary sequences (Florendo 1987, Aurelio 1992, cited in MGB, 2010), metamorphic basements (MMAJ-JICA, 1990), and melanges (Yumul, 2003).

The Philippine Fault Zone passes through the narrow, linear river valley from Barangays Kauswagan to Hinabian in the north; southward through the eastern footslopes of the Canabag Mountain range from Barangays Guinsaugon to Hindag-an; and into Cabalian Bay (MMAJ-JICA, 1990; MGB, 1997). The Hindag-an Panian Fault Zone is referred to as a splay of the Philippine Fault Zone (Leynes et al., 1996).

The major sliding plane at the Guinsaugon landslide is a fault (Catane et al., 2008; Evans et al., 2007; Lagmay et al., 2006; Orense and Sapuay, 2006) that strikes N23W and dips 45-56 degrees, with slickenlines oriented 15SE and 45-60SE (Catane et al., 2008). These are interpreted as a riedel shear synthetic to the Philippine Fault Zone (Cardiel et al., 2008). Consequent to this designation, the near-vertical, southeast-dipping slope (SP1 in Catane et al., 2008) bounding the major sliding plane to the north may be interpreted as the corresponding antithetic riedel shear, as shown in (Guthrie et al., 2009). A stereonet plot of 74 joint and fault readings within the landslide area shows predominant discontinuities steeply dipping to the northeast (Futalan et al., 2010).

2.3 Methods

Geologic mapping was conducted intermittently from 2006 to 2009, mostly during the months of April and May, except in 2008 when the fieldwork was conducted from October to November.

NAMRIA topographic maps (NAMRIA, undated *a,b*) were used as base maps. Ground positions were located in the maps using a hand-held GPS receiver, terrain analysis and compass triangulation. Traverses were mainly conducted along roads and highways, coasts, rivers and streams.

Rocks were identified in the field from outcrop exposures, hand samples obtained using a geologic hammer, and some core samples. A lithologic map was made by plotting lithologies mapped in the field. Lithology in places between or beyond the surveyed areas were inferred on the basis of stratigraphic principles, similarities or differences in topographic contour lines, and identification of boulders (floats) along river and stream beds. Stratigraphy was then inferred from the lithologic map, lithologic contacts mapped in the field, relative elevations, and from the literature.

The orientation of geologic structures were determined using a pocket transit. Analysis of structural data is done with the aid of stereonet, as implemented in GEORIENT v 9.5.0. Whenever there is sufficient data, contour and gridded densities of poles to planes are plotted. Otherwise, great circles or poles to planes are plotted. Mean orientation and resultant length are also reported. The latter expresses the uniformity of orientation of plotted data, where 0 and 1 correspond to perfectly random and uniform orientations, respectively. Fold axes are determined by fitting a great circle to the poles of the identified limbs of the fold. The pole to this great circle corresponds to the fold axis.

2.4 Results and discussion

2.4.1 Per Site

Results of the geologic mapping are first presented per location, and then summarized based on lithologies, geologic structures and stratigraphy. Adjoining localities have been arbitrarily grouped together (Fig. 4) primarily for convenience of discussion.

Figure 4. Index map of locations in study area. Outlined polygons represent the grouped localities and the approximate extent of actual field survey conducted. (a) Hinabian – Malinao – Kauswagan (b) Magatas – Guinsaugon – Sug-angon – Ayahag – Nueva Esperanza – Panian (c) Panian – Himbangan – Hindag-an (d) Lipanto – Himay-angan (e) San Roque – Tabugon – Molopolo (f) Himay-angan – Ilag – Nailong (g) Magaupas – Magkasag – Cawayan (h) Biasong – Bogasong – Pangi – Nahulid – Nahawong. Base map is a composite of elevation and shaded relief maps, derived from the NAMRIA digitized topographic contour maps. Coordinate system is UTM WGS84.

2.4.1.1 Hinabian – Malinao – Kauswagan

The Hinabian – Malinao – Kauswagan area is located in the northeast part of the study area (Fig. 4). It is separated from the rest of the study area by the linear, NW-SE trending section of the Lawigan River, which defines the trace of the Philippine Fault Zone. The elevation ranges from 140 to 560 masl, lower than the Can-abag mountain range directly west of the area.

Ultramafic rocks, sandstones, siltstones and limestones underlie the Hinabian-Malinao-Kauswagan area. In Barangay Hinabian, ultramafic rocks are exposed along a continuous ~10 m section of the dirt road leading to the barangay chapel. The rocks exhibit a greenish black color and highly serpentinized surface. In the area beyond the chapel and at a higher elevation, highly fractured, dark brown siltstones are exposed along walkways. At the base of an isolated hill, these clastic rocks strike N5E and dip 50NW, and are found below yellowish-white recrystallized limestone.

Along the dirt road leading to Barangay Malinao, there are intermittent exposures of the same suite of ultramafic rocks and siltstones (Fig. 5). The barangay proper is mostly underlain by siltstones and sandstones (Fig. 6a). Beds strike N56E and dip 70SE. There are at least two small limestone hills within the barangay. The limestones are recrystallized, and near their base, ultramafic rocks are exposed.

The road between Barangays Malinao and Kauswagan is highly vegetated. However, numerous angular pebbles and cobbles of ultramafic rocks found along the path and along small streams crossing the path suggest that the area is underlain by those rocks. At a river crossing, on the way to Kauswagan, the ultramafic rocks are exposed in the channel walls for a span of at least 20m.

Around 50 meters from the ultramafic rocks, and along a channel bend, well bedded, very fine grained sandstones are exposed along the channel (Fig. 6b). The unit strikes N11E and dips 6NW, and has a minimum total thickness of 5 meters. Crystalline limestones bound the pathway from the river to the Kauswagan proper.

For the Hinabian – Malinao – Kauswagan area, limestones were observed to form hilltops with either siltstones or ultramafic rocks at the base. This suggests a young age of the limestones relative to the rocks at their base. Assuming a non-fault contact between the ultramafic rocks and the siltstones, the former is assumed to be unconformably overlain by the latter. The intermittent exposures of these two rocks suggest that the siltstones may be of limited thickness or, in some areas, completely eroded to expose the ultramafic rocks. Siltstones and sandstones are also variably oriented within the area.

2.4.1.2 Magatas – Guinsaugon – Sug-angon – Ayahag – Nueva Esperanza – Panian

This area is part of the the east-facing flank of the Can-abag mountain range, and is bordered by the Can-abag ridgeline to the west and by the Lawigan River to the east (Fig. 4). Steep and planar slope faces, oriented and located similarly to the Guinsaugon sliding plane (Catane et al., 2008; Guthrie et al., 2009), are arranged in a right-stepping manner. Two of these, located west of Ayahag and Sug-angon, have strike lengths of more than 1000 m and heights of 300-400 m. The steep slopes abruptly change to low gradient, hummocky topography, which in turn grades into the Lawigan river flood plain. In the hummocky areas, particularly between Magatas and Ayahag, there are linear ridges oriented NW-SE, parallel and close to the main trace of the Philippine Fault Zone.

The source area and path of the Guinsaugon landslide is underlain by a thick sequence of southwest-dipping beds of sandstones, breccias and mudstones (Figs. 8, 9a, 9b, 10a, 10b, 11). Volcanic rocks of andesitic composition are found intruding the sedimentary rocks. Bedding planes near and along the path of the landslide are oriented N45W 28SW, N52W 32SW, and N8W 21SW.

The surveys conducted in Magatas, Sug-angon, Ayahag and Nueva Esperanza were mainly along the road connecting these barangays. There were no outcrops along the road, although there are numerous blocks of mostly sandstone and breccias in the general area. Where streams intersect the road, the sediments found are breccias,

Figure 5. Siltstones (brown, foreground) in contact (yellow line) with ultramafic rocks (greenish black, background) along road to Malinao. Geologic pick for scale (near contact). See location in Figure 7.

(a)

(b)

Figure 6. Malinao to Kauswagan clastic rock outcrops. (a) highly fractured siltstones near Malinao proper. (b) NW-dipping beds of sandstones along river near Kauswagan. Bedding plane edges and orientation in yellow. 3-m pole for scale. Photograph looking south. See locations in Figure 7.

Figure 7. Location map of Hinabian-Malinao-Kauswagan and Magatas-Guinsaugon-Sug-angon-Ayahag-Nueva Esperanza-Panian field photographs. Point (yellow circles) labels correspond to figure numbers. Elevation and shaded relief image composite basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). Contour interval = 20 m.

sandstones and volcanic rocks, very much similar to those found in the deposit of the Guinsaugon landslide. There is a notable absence of ultramafic rocks and limestones along the streams.

Sandstones and breccias are exposed in the Panian River, upstream of the Panian bridge. The sandstones are fine- to medium-grained, medium- to thickly-bedded, and oriented N40W 12SW. The breccias are grain-supported, with cobble- to boulder-sized clasts of andesite. The breccias overlie the sandstones along a contact that trends N42W 26SW.

In a minor tributary of the Panian River, ultramafic rocks line the channel wall, and exclusively comprise the stream sediments. This indicates that the contact between the ultramafic rocks and the sandstones and breccias, although not found, occurs between the tributary and the main trunk of Panian River.

The Magatas – Guinsaugon – Sug-angon – Ayahag – Nueva Esperanza – Panian area is underlain mainly by southwest-dipping (Fig. 12) breccias, sandstones and mudstones. Ultramafic rocks and limestones are not exposed based on a survey of stream sediments draining the upstream areas. The former can only be observed in the south at a stream in Panian, where it is believed to be in contact with breccias and sandstones.

Topographic features such as steep slope faces and linear ridges, as well as secondary geologic structures are consistent with the geometry of deformations related to a left lateral strike-slip fault. The combination of steep slopes mantled at the base by hummocky topography suggests the frequent occurrence of landslides similar to that of Guinsaugon, which deposited colluvium thick enough to limit the exposure of bedrock along the road connecting Guinsaugon and Panian.

2.4.1.3 Panian – Himbangan – Hindag-an

The area is bound on the west by a broader but lower section of the Can-abag ridge, and to the east by the Lawigan River floodplain and the Cabalian Bay (Fig. 4). It is where the trace of the PFZ goes offshore (MGB, 1997). Linear ridges oriented NW-SE and

Figure 8. Landslide source area showing northeast-dipping sliding plane (SP, with strike and dip symbol), and traces of southwest-dipping bedding planes (B, yellow dotted lines). Trees at the landslide crown for approximate scale. Photograph looking northwest. See location in Figure 7.

parallel to the PFZ are also present. The elongate Himbangan River watershed shifts the Can-abag ridgeline from a northwest-southeast trend to a northeast-southwest trend. A linear scarp-like feature is found along the coast of Himbangan, and is very visible across the Bay at the St. Bernard town proper. Hindag-an is characterized by gently-sloping, low-relief headwater areas which become highly dissected downstream.

The area is underlain by ultramafic rocks, siltstones, and limestones. In Barangay Panian, the ultramafic rocks are highly sheared and weathered, occurring as gray, angular blocks in a matrix of gray to brown soil material. At an abandoned quarry site, these rocks are thrust over highly fractured, dark brown siltstones. The thrust contact strikes N22E and dips 14SE, whereas the siltstone bedding plane strikes N2E and dips 56NW. Around 300 m south of the quarry area and beside the highway is a hill underlain by limestone, although this unit's contact with the ultramafic rocks and siltstones were not determined.

(a)

(b)

Figure 9. Sandstones and mudstones along Guinsaugon landslide path (a) Southwest-dipping beds of sandstones. Green field notebook for scale. Photograph looking west. (b) Interbeds of well-indurated sandstones and mudstones, dipping SW. 1-m tape for scale. Photograph looking south. See locations in Figure 7.

(a)

(b)

Figure 10. Breccias along Guinsaugon landslide path. (a) Massive breccia with sub-angular volcanic clasts. (b) Massive breccia boulder transported by the landslide. See locations in Figure 7.

Figure 11. Uniform SW dip of bedding planes along the Guinsaigon landslide. Plane pole density plot; N=8; Mean orientation N34W 23SW; Mean resultant length=0.98. Data from this study, Catane et al. (2008), and Futralan et al. (2010).

Figure 12. Uniform SW dip of bedding planes from Guinsaigon to Panian (incorporating data from Figure 11). Plane pole density plot; N=10; Mean orientation N25W 22SW; Mean resultant length=0.98.

In Barangay Himbangan, ultramafic rocks are exposed in a linear scarp at least 700 m long and roughly parallel to the highway. This scarp can be easily seen across the Cabalian Bay from the town proper of St. Bernard. Similar rocks are exposed at the coast, downslope of the scarp. The rocks form a steep slope, whose base is mantled by talus.

The main channel of the Himbangan River is also underlain by ultramafic rocks. Upstream, the ultramafic rocks are gray to bluish gray and highly sheared. Talus fans from rock falls (Fig. 13a) are common along the channel. Sections downstream show outcrops that are greenish black, massive and less fractured (Fig. 13b). While most of the river sediments are derived from ultramafic rocks, there are occasional limestones which indicate the presence of limestone outcrops somewhere upstream. Downstream of the confluence of the main channel and a tributary, less than a kilometer from the highway, sediments made of breccias and volcanic rocks begin to appear. This suggests that the contact between the ultramafic rocks and breccias is within the drainage basin of the tributary.

Along the road between Barangays Himbangan and Hindag-an, ultramafic rocks are thrust over siltstones (Fig. 14a). The cream-colored siltstones have beds oriented N72E 11SE. The fault contact strikes N50E and dips 11SE. On the fault plane, there is lineation with a pitch of 60NE, indicating movement parallel to the NW-SE direction. About 150 m south of this site is another thrust fault involving the same rocks. The siltstone bed trends N80E 10NW, whereas the fault trends N56E 31SE.

The barangay proper of Hindag-an is underlain by highly sheared ultramafic rocks. These rocks are along the roadcut, about 100 m south of the proper, as well as in the lower reaches of stream channels originating from the upland in the west. A well-defined shear zone approximately 0.4 m wide and oriented N35W 25SW was measured in one of the roadside outcrops (Fig. 14b). Limestones underlie the headwaters of these streams, as seen from limestone boulders and travertine in the streams. From the sea, the limestone can be seen as vertical exposures of black and white rocks (Fig. 15a).

Along a sharp road bend at the boundary of Hindag-an and Lipanto, limestones (Fig. 15b) overlie ultramafic rocks. The contact is oriented N63E 39SE. The limestone here is lime mudstone which have undergone some degree of recrystallization. Some fossils are present. This limestone unit extends down to the coast, where locals of Hindag-an built a religious shrine. A stereonet plot of 3 joint measurements show steeply dipping and variably oriented discontinuities in the outcrop.

After the road bend, another roadcut exposes highly sheared ultramafic rocks thrust over siltstones. The contact is oriented N75W and 42SW and has lineations with pitch of 50SE, indicating movement parallel to NW-SE direction. The siltstones are indurated and contain recrystallized limestone clasts.

Joints measured from the ultramafic outcrops show at least five dominant orientations (>8% point density) at N45W 86SW, N71W 80NE, N72E 32NW, N71E 65SE, and N12E 79NW (Fig. 16), four of which are steeply-dipping.

In the Panian – Himbangan – Hindag-an area, a highly linear scarp-like feature trends NW-SE, parallel to the PFZ. Ultramafic rocks are thrust over siltstones. The ultramafic rocks are in turn, overlain by the limestones. Of the four thrust faults, three dip to the southeast while one dips southwest. Slickenlines in two of the faults and the dip directions of the faults indicate a northwest-directed thrust movement (Fig. 17).

2.4.1.4 Lipanto – Himay-angan

Lipanto is bound on the west by the Can-abag ridge and on the east by Cabalian Bay (Fig. 4). Similar to Hindag-an, it has gently sloping, low-relief headwaters, and more dissected downstream areas. Linear ridges oriented N-S are present north of Lipanto proper. There are also steep, linear slope faces oriented NW-SE found northwest and south of the barangay proper. Himay-angan is characterized by a narrow alluvial valley oriented NE-SW. The valley is bound on the northeast by the Can-abag ridge and on the southwest by Sogod Bay, where the Himay-angan River drains into. The ridges on both sides of the valley are no more than 260 masl.

(a)

(b)

Figure 13. Himbangan to Hindag-an. (a) landslide in sheared ultramafic rocks, upstream Himbangan River. (b) blocky ultramafic rocks, downstream Himbangan River. Height of person in both photographs is 1.6 m. See location in Figure 18.

(a)

(b)

Figure 14. Himbangan to Hindag-an. (a) disintegrated ultramafic rocks (almost soil) thrust over siltstones. Geologic pick for scale, bottom center. (b) well-defined shear zone within very blocky ultramafic rock mass. 1-m measuring tape for scale. See locations in Figure 18.

(a)

(b)

Figure 15. Himbangan to Hindag-an. (a) limestone peaks and cliffs in upstream Hindag-an as seen from the sea. (b) limestone outcrop near Hindag-an-Lipanto boundary. 2-m pole for scale. See location in Figure 18.

Figure 16. Variably-oriented, steeply-dipping joint planes in ultramafic rocks, Hindag-an. Plane pole density plot show at least 5 dominant joint sets (at > 8% density). N=31; mean resultant length=0.45.

Figure 17. NW-directed thrusting indicated by slickenlines on thrust faults involving ultramafic rocks and siltstones. Faults (great circle) N=4; Mean orientation N71E 21SE; Mean resultant length=0.95. Slickenlines (red dots) N=2; Mean orientation 22 S53E (plunge, trend); Mean resultant length =0.94.

Figure 18. Location map of Panian-Himbangan-Hindag-an field photographs. Point (yellow circles) labels correspond to figure numbers. Elevation and shaded relief image composite basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). Contour interval = 20 m.

The Lipanto – Himay-angan area is underlain by siltstones, sandstones, limestones and ultramafic rocks. Ultramafic rocks are exposed along the highway between Hindag-an and Lipanto. A section of this road near a bridge serves as the boundary between sandstones, siltstones and ultramafic rocks. The sandstone and siltstone interbeds are located on the eastern side. The rocks are dark brown, highly fractured, and have reddish black alterations. Individual beds measure 1-20 cm. Beds are oriented N10W 19SW, N1W 27SW, N16W 11SW, N40W 22SW, N38W 15SW, N52W 24SW, N47W 30SW (Figs. 19 and 21).

Ultramafic rocks are on the other side and extend westward into the mountains. Like those in the earlier discussed locations, the rocks here are greenish-black and highly fractured. Travertine and karstic limestone boulders suggest the presence of limestones on top of the ultramafic rocks, or further upstream.

Lipanto proper is similarly underlain by ultramafic rocks, sandstones and siltstones. A major stream exposes ultramafic rocks which are massive and very blocky in some sections, and highly sheared in other sections. Stream sediments are composed of ultramafic rocks and varied lithologies such as gabbros, andesites and breccias. The gabbros contain coarse pyroxene and plagioclase crystals. The andesites are sub-angular and fine grained. The breccias have ultramafic clasts in a white, coarse crystalline matrix. The gabbros and andesites may have been derived either from the upper section of the ophiolite suite to which the ultramafic rocks belong, or may have been formed by arc magmatism. Local guides report presence of limestones and sandstones-siltstones outcrops further upstream based on comparison with river floats. Variably-oriented joints in ultramafic rocks were measured along the river. Resultant length is 0.655.

South of Lipanto proper, the road is underlain by siltstones and sandstones. A continuous section is exposed by a landslide that occurred in February 2006. The exposure is 4 to 6 meters high, and has a total length of around 450 meters. The section exhibits a coarsening upward sequence: from light brown, highly fractured siltstones; to gray, fine- to very fine-grained sandstones; to dark gray pebbly sandstones

and conglomerate with granule-sized clasts. Beds are oriented N69E 34SE, N41E 40SE, and N47E 36SE (Figs. 19 and 21). Minimum section thickness, based on a 350m-long cross section is 206m.

As the highway turns southwestward, very fine- to fine-grained sandstones are exposed on the southeast side of the road. The sandstones are dark gray, with beds striking N23E and dipping 32NW (Figs. 20,19 and 21). Further westward down the road, the beds are oriented N7W 36SW, N 23W, N20E 33NW and N60W 25SW (Fig. 21) before reaching Himay-angan proper. Medium-grained sandstones are exposed on a hill across Himay-angan River which has undergone small rotational and translational failures. The sandstone beds are oriented N5W 23SW (Fig. 21).

In Lipanto – Himay-angan, ridges and steep slope faces trend N-S and NW-SE, respectively. There were no directly observed contacts between the ultramafic rocks, sandstones-siltstones and limestones. Limestones found further upstream in Lipanto may be overlying the ultramafic rocks, as well as the sandstones and siltstones, based on elevation. The sequence of sandstones and siltstones along the highway from Lipanto to Himay-angan show a southward progression of SW- to SE- to W-dipping bedding planes. This suggests one or two NE-SW-trending, and SW-plunging fold axis/axes (Fig. 21).

2.4.1.5 San Roque – Tabugon – Molopolo

This is the southernmost tip of the study area (Fig. 4). It is only around 2 km wide, measured perpendicular to the main ridgeline and has maximum elevation of 160 masl. Similar to Himay-angan, it has a main drainage oriented NE-SW, parallel to the road connecting Tabugon to Molopolo, and draining in both directions. Several ridges are aligned parallel to the main drainage.

The area is underlain by limestones, ultramafic rocks, sandstones and breccias. Limestone outcrops are found along the highway and also east of the steel bridge in Barangay San Roque (Fig. 23). The limestones are gray to brown, massive and have

(a)

(b)

Figure 19. Lipanto-Himay-angan. (a) SW-dipping sandstone-siltstone interbeds near Hindag-an-Lipanto boundary. 1-m pole for scale. Photograph looking northeast. (b) SE-dipping sandstones at Lipanto landslide (Feb. 2006). Compass for scale. Photograph looking southwest. See locations in Figure 22.

Figure 20. NW-dipping sandstones along highway near Lipanto-Himay-angan boundary. Bedding plane edges and orientation symbol in yellow. Photograph looking south. Person is 1.6 m in height. See location in Figure 22.

already been recrystallized. Representative joint measurements show steeply dipping ($>40^\circ$), variably-oriented discontinuities (Fig. 24).

Ultramafic rocks are exposed at a steep roadcut along the highway leading to Barangay Tabugon (Fig. 23). The rocks are gray to dark green in color, and highly fractured. Joints are variably oriented and dip more than 48 degrees.

Sandstones and breccias are then found further north just after passing the road going to Barangay Molopolo (Figs. 23 and 25). The sandstones are thickly-laminated to thickly-bedded, and are medium- to fine-grained. Bedding planes are oriented N20E 39NW. The breccias are found below the sandstones. Clasts of ultramafic rocks, mudstones, and individual fossil gastropods are contained in a sandy matrix. The ultramafic clasts are pebble- to cobble-sized, and sub-rounded. The mudstone clasts are sub-angular, boulder-sized and range from dark gray to grayish white. Some of the mudstone clasts are massive while others exhibit stratification. The individual fossil fragments were most likely 'recent' fauna, relative to the deposition of the breccia. These features of the breccia imply a proximal mudstone outcrop source; a distal terrestrial

(a) Hindag-an (red): N=7; Mean orientation N31W 20SW; Resultant length= 0.99. Lipanto (violet): N=3; Mean orientation N52E 36SE; Resultant length=0.99. Beta (fold) axis: 17-S27W. *Left*: pole and beta plot; *Right*: great circles.

(b) Lipanto (violet): N=3; Mean orientation N52E 36SE; Resultant length=0.99. Himay-angan (green): N=6; Mean orientation N1W 26SW; Resultant length = 0.97. Fold axis (β): 15-S33W. *Left*: pole and beta plot; *Right*: great circles.

Figure 21. Bedding plane plots indicating NE-SW-trending and SW-plunging fold axis/axes. (a) Hindag-an-Lipanto. (b) Lipanto-Himay-angan.

Figure 22. Location map of Lipanto-Himay-angan, San Roque-Tabugon-Molopolo, and Himay-angan-Ilag-Nailong field photographs. Point (yellow circles) labels correspond to figure numbers. Elevation and shaded relief image composite basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). Contour interval = 20 m.

ultramafic sediment source; and the sub-aerial exposure of ultramafic rocks prior to the deposition of the conglomerate.

The road between Tabugon to Molopolo runs west to east. Greenish dark gray ultramafic rocks outcrop at the southern side of the road, along a stream. On the north side, a gently sloping, grassy terrain is underlain by angular blocks of both fresh and weathered ultramafic rocks in a soil matrix. The peak and slope immediately north of this area is interpreted as the possible sources of the deposit. A landslide from this peak reportedly delivered ultramafic blocks to the shore north of Barangay Molopolo. South of the barangay, limestones are exposed by a small landslide near the shore. Both limestones and ultramafic rocks comprise the beach sediments in the barangay, giving the deposits a salt and pepper appearance.

The sandstones and breccias in Tabugon unconformably overlie ultramafic rocks and mudstones, based on the clasts in the breccia. Individual gastropod fossils, instead of limestone clasts, suggest the absence of limestones older than the conglomerates. The fringing reef-type of location of the limestones in the area and the above evidence suggest that the limestones in Molopolo and San Roque are younger than the ultramafic rocks, mudstones, sandstones and conglomerates.

2.4.1.6 Himay-angan – Ilag – Nailong

The main feature of the area (Fig. 4) is the near-vertical cliff faces found upstream of Ilag River which are also visible from the highway (Fig. 26). The cliffs rise upto 300-400 masl and are defined by closely-spaced contours oriented NW-SE and NE-SW. The northern tributary of the Ilag River passes through an area flanked by these cliffs. The ridge on top of these cliffs are broad, unlike the sharp ridges in Guinsaugon-Panian, and reach elevations of 400-500 masl.

Limestone outcrops along the highway in Himay-angan proper (Fig. 26). Textures range from crystalline to calcarenite. Joint measurements show >50 degrees dip angle for density contours >4%. Clusters with >8% point density are at N7E 60SE, N21E

(a)

(b)

Figure 23. San Roque to Tabugon. (a) Limestone outcrop in San Roque. Person at left is 1.6 m in height. (b) Ultramafic rocks in Tabugon. 3-m pole for scale. See locations in Figure 22.

Figure 24. Variably-oriented, steeply-dipping (>40 degrees) representative joints in limestones, San Roque. Great circle plot; $N=9$; Resultant length=0.40.

72NW, and N74E-81SE (Fig. 27). Limestone blocks can be found at the base of the northeast-trending ridge north of and parallel to the Himay-angan River. This suggests that the limestone along the highway in Himay-angan proper extends to the ridge.

Ultramafic rocks form an isolated hill, with typical grassy vegetation cover, west of Himay-angan proper. These rocks are also found at the mouth of Ilag River along the highway (Fig. 26). The rock mass appears layered, with dominant discontinuities oriented N23W 50SW. This unit may be the layered peridotite component of the ophiolite.

Ilag River has two major tributaries. The tributary coming from the northwest is underlain by interbeds of sandstones and mudstones (Fig. 26 and 28). The sand- and silt-sized components are light brown in color, whereas the clay-sized are gray. Highly localized folding is evident from the variations of bed orientations in a ~ 30 -meter long outcrop along the river: N72W 60NE; N02W 11SW; N02E 66NW; N01E 75NW; N36E 02NW. Downstream of this area, exposed beds are oriented N35E 61 SE and N35E 66 SE (Fig. 29). The presence of limestone blocks along the river, as well as the steep, black-and-white colored cliffs bounding the river, suggest that limestones overlie this sandstone-mudstone unit. The tributary of Ilag River which originates from the southeast contains pebble-sized mudstones and sandstones, and boulder-sized ultramafic rocks and limestones, suggesting the exposure of these rocks upstream of this area.

(a)

(b)

Figure 25. San Roque to Tabugon. (a) Breccia containing mudstone boulders, ultramafic pebbles and gastropod shells. Geologic pick for scale. (b) close-up of clasts in (a). Five-peso coin for scale. See locations in Figure 22.

(a)

(b)

Figure 26. Himay-angan to Ilag. (a) Limestone outcrop along highway in Himay-angan. 3-m pole for scale. (b) Limestone cliffs at Ilag. See locations in Figure 22.

Figure 27. Variably-oriented, steeply-dipping joints in limestones, Himay-angan. Plane pole density plot; N=54; Resultant length=0.41.

The slopes bounding the main trunk of the Ilag River is mainly underlain by light gray, fine- to very fine-grained sandstones. While there was no outcrop, a landslide originating from the ridge southeast of the river left a deposit which contains mainly this type of rock. Occasionally found within the deposit are well-rounded pebbles of ultramafic rocks. These pebbles are interpreted to be clasts of the sandstones.

Along the highway in Nailong, there are several outcrops of claystones, very fine sandstone to siltstones, siltstones and fine sandstones. Bedding planes are oriented N10W 50SW, N20W 49SW, N20W 40SW, N33W44SW (Fig. 29). In one of these outcrops, cobble- to boulder-sized ultramafic rocks and recrystallized limestone made of coral fragments were found embedded as clasts in the matrix.

In Himay-angan – Ilag – Nailong, limestones appear to be overlying ultramafic rocks, sandstones, and siltstones, based on topographic position. Based on the clast lithology, sandstones and siltstones at Nailong unconformably overlie the ultramafic rocks and limestones. Most of the bedding planes of sandstones and mudstones in the area dip at angles greater than 45 degrees, and these possibly define at least two fold axes oriented NE-SW (Fig. 29).

(a)

(b)

Figure 28. Ilag. (a) Ultramafic rocks along highway in Ilag. 3-m pole for scale, at right. (b) West-dipping sandstone-mudstone at upper Ilag. Photograph looking northwest. Person at upper left background for scale. See locations in Figure 22.

(a) Upper Ilag (red): N=5; Mean orientation N27E 33NW; Resultant length= 0.75. Lower Ilag (violet): N=3; Mean orientation N35E 63SE; Resultant length=1.0. Beta (fold) axis: 1-S17W (plunge-trend). *Left:* pole and beta plot; *Right:* great circles.

(b) Lower Ilag (violet): N=3; Mean orientation N35E 63SE; Resultant length=1.0. Nailong (green): N=4; Mean orientation N20W 45SW; Resultant length=0.99. Beta (fold) axis: 32-S17W. *Left:* pole and beta plot; *Right:* great circles.

Figure 29. Bedding plane plots indicating NE-SW-trending and SW-plunging fold axis/axes. (a) Upper Ilag - Lower Ilag. (b) Lower Ilag - Nailong.

2.4.1.7 Magaupas – Magkasag – Cawayan

This area is part of the the west-facing flank of the Can-abag mountain range, and is bordered by the Can-abag ridgeline in the east and by Sogod Bay in the west (Fig. 4). It is characterized by highly dissected terrain with narrow ridges and channels. These are very evident and can be easily observed from the national highway. For Cawayan, this topography is consistent from its headwaters down to the coastal plain. However, the upstream areas of Magkasag and Magaupas are not as dissected as their downstream areas, and are instead characterized by relatively moderate terrain. The boundary between these types of topography is marked by steep slope faces oriented NE-SW and NW-SE, and can be seen clearly in Magkasag. The Can-abag ridge at Cawayan is sharp, but becomes broader and lower as it approaches Magkasag and Magaupas.

Between Magaupas and Nailong, conglomerates with a sub-rounded to sub-angular boulder-sized clasts in medium to coarse sand matrix are exposed. Bedding planes are roughly oriented at N45W 35SW. A waterfall in Magaupas River exposes clast-supported breccia. The clasts are andesitic and range from cobble to boulder in size. Floats of sandstones, limestones and ultramafic rocks are observed in the vicinity of the waterfall (Fig. 30). Clast-supported breccias are also found along the road in Barangay Magaupas. The rocks consist of angular, cobble- to boulder-sized andesites, in a sandy to pebbly matrix.

A landslide similar to that of Guinsaigon was mapped at Barangay Magkasag (Fig. 30). It exposed a fault oriented N35W 72NE. The fault gouge is at most 2cm thick, has slickenlines oriented 15SE, and is rough in the northwest direction. Breccias with pebble-to boulder-sized volcanic clasts in a coarse-grained sand matrix comprise the original lithology of the gouge. This fault is intersected by near-vertical surface striking N40E. The surface of this intersecting structure is not gouged, but also exposes similar lithology as the fault plane. Beneath the breccias are mudstones and medium-grained, pebbly and laminated sandstones with bedding planes oriented N18W 30SW (Figs. 30

and 32). The upstream and downstream areas of Magkasag are underlain mainly by sandstones and breccias, respectively.

A series of roadside outcrops of conglomerates and breccias can be found between Barangays Magkasag and Cawayan. The rocks range from being clast-supported to matrix-supported. Clasts are pebble- to boulder-sized, while the matrix is medium- to coarse-grained sand. Bedding planes are oriented N17W 46SW, N40W 30SW, N01W 42SW and N15W 32SW.

Along Cawayan River, sandstone beds are oriented N48W 20SW, N15W 32SW, and N32W 31SW. Further upstream, breccias underlie these sandstone beds. The contact is oriented N60W 35SW. Mean bedding plane orientation is N38W 28SW, and the resultant length is 0.984.

The Magaupas – Magkasag – Cawayan area is underlain by southwest-dipping beds of sandstones, breccias, conglomerates, and mudstones (Fig. 33).

2.4.1.8 Biasong – Bogasong – Pangi – Nahulid – Nahawong

This area is part of the west-facing flank of the Can-abag mountain range, and is bordered by the Can-abag ridge in the east and by Sogod Bay in the west (Fig. 4). It is characterized by highly dissected terrain with narrow ridges and channels, as well as broad, planar slopes dissected by narrow channels. The narrow ridge and channel topography, similar to Cawayan, persists in Biasong and Bogasong, and in the upper parts of Pangi and Nahulid, close to Can-abag ridge. The broad planar slopes with narrow channels are found in the lower parts of Pangi and Nahulid, and in most of Nahawong.

Breccias are exposed along the highway in Biasong, across the Biasong Springs (Fig. 35a). The rocks are massive and thickly-bedded. Bedding planes are oriented N28E 34NW. Clasts are pebble- to boulder-sized, sub-rounded to angular andesites and the matrix is sandy to granular. Along Bogasong River, sandstones are exposed on a ~15-meter high cliff. The bedding plane is oriented N32W 15SW. The outcrop is continuous for about 200 meters, indicating a minimum thickness of ~50m.

(a)

(b)

Figure 30. Magaupas to Magkasag. (a) Breccia in waterfall in Magaupas. Person is 1.6 m in height. (b) landslide source area in Magkasag exposing a steep, gouged fault plane (*left*), and another steep, but ungouged, structure (*right*). Photograph looking upward and west. See locations in Figure 34.

Figure 31. Magkasag. SW-dipping mudstones and sandstones below the breccia exposed by the landslide in Fig. 32. Photograph looking west. 1-m tape for scale. See location in Figure 34.

(a)

(b)

Figure 32. Magkasag. (a) base of the exposed fault plane (N35W 72NE), showing breccias behind the gouge. Person in photograph facing NW. (b) Close-up of gouge features. Surface is generally rough along the NW direction. Compass pointing SE. See locations in Figure 34.

Figure 33. Uniform SW dip of bedding planes from Magaupas to Cawayan. Plane pole density plot; N=10; Mean orientation N28W 32SW; Resultant length=0.98.

Figure 34. Location map of Magaupas-Magkasag-Cawayan field photographs. Point (yellow circles) labels correspond to figure numbers. Elevation and shaded relief image composite basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). Contour interval = 20 m.

Conglomerates and sandstones outcrops are found along the highway between the Pangui and Bitanhuan Rivers in Barangay Pangui. In one outcrop, matrix-supported conglomerate is overlain by clast-supported conglomerates. The conglomerates are cobble- to boulder-sized porphyritic andesite clasts in a matrix of medium-grained lithic sand. Bedding planes are oriented N-S 35W and N40E 16NW. In another outcrop, thinly-bedded, brown sandstones overlie medium-bedded, gray conglomerates (Fig. 35b). The sandstone beds strike N10W and dip 32SW, whereas the conglomerate beds strike N36E and dip 35NW. Both rocks are not well cemented, and thus, easily disintegrate. In another site, beds of pebbly, fine- to medium-grained sandstones are oriented N40E 30NW. Mean bedding plane orientation in Pangui is N19E 28NW and resultant length is 0.976.

Along Bitanhuan River in Barangay Nahulid, there is an exposure of alternating beds of very coarse- to coarse-grained sandstones and silt to coarse-grained sandstones. The bedding planes are oriented N14W 28SW. Upstream, beds of yellowish brown, very fine-grained sandstones strike N13W and dip 53SW. The beds are cut by small extensional faults oriented N75W 85NE. Further upstream are outcrops of matrix-supported breccia. The breccia consists of granule- to boulder-sized, sub-angular clasts of andesite in a volcanic matrix. Mean bedding plane orientation in Nahulid is N13W 41SW and resultant length is 0.976

Sandstones are exposed in Barangay Nahawong along a road leading to the mountain. The sandstones are thinly- to thickly-bedded and do not exhibit grading. Bedding planes are oriented N26W 32 SW, N45W 25SW, and N12W 31SW. While most of these units consist of sandstones, there are also intermittent layers of pebbly sandstones (Fig. 37a). As the road turns north, a ~30m stretch of sandstone body is exposed along the ditch. The sandstones are gray, and made of alternating fine- and very coarse-grained sand (Fig. 37b). There are also layers of coarse-grained sandstone with pebble- to cobble-sized igneous clasts. The bedding planes are oriented N72W 16SW. Immediately after this outcrop, the rocks become highly weathered, reddish brown,

(a)

(b)

Figure 35. Biasong to Nahawong. (a) Northwest-dipping, massive and thickly-bedded breccia along highway in Biasong. Bedding plane edge and orientation in yellow. Photograph looking northeast. 2-m pole for scale. (b) Southwest-dipping, thinly-bedded sandstones (brown) overlying northwest-dipping conglomerate (gray) in Pangi. Photograph looking east. Bedding plane edges (dotted) and orientation, and lithologic contact (dashed) in yellow. 3-m pole for scale. See locations in Figure 38.

Figure 36. Uniform SW dip of bedding planes from Biasong to Nahawong. Plane pole density plot; N=13; Mean orientation N3W 26SW; Resultant length=0.98.

breccias. Bedding planes are observable across the road and dip to the northeast. While the clasts and matrix are distinctly visible, these have all been weathered to clay. Mean bedding plane orientation in Nahawong is N33W 25SW and resultant length is 0.982.

The Biasong – Bogasong – Pangi – Nahulid – Nahawong area is underlain by west-dipping sandstones, breccias, and conglomerates. (Fig. 36).

2.4.2 Summary

2.4.2.1 Lithology

A summary of major lithologies observed in the sites is presented in Table 1. The sites were divided into two groups: those with limestones and ultramafic rocks (a, c, d, e, f), and those with breccias or conglomerates (b,g,h). These two are mutually exclusive, such that one group does not have the other group's characteristic lithologies. The ultramafic-limestone group does not have breccias or conglomerates; and the breccia-conglomerate group does not have ultramafic rocks or limestones. However, these groups both have common lithologic associations with sandstones and mudstones.

The ultramafic-limestone group was further divided spatially based on the specific lithologies: ultramafic rocks, limestones and fine clastic rocks. The sandstones

(a)

(b)

Figure 37. Nahawong. (a) SW-dipping pebbly sandstones upslope of Nahawong. 1.2-m pole for scale. Bedding plane edges and orientation in yellow. Photograph looking northeast. (b) SW-dipping sandstones further upslope of Nahawong. 0.45-m pole for scale. Bedding planes strike directly perpendicular to photograph, and dip to the right (southwest). Photograph looking southeast. See locations in Figure 38.

Figure 38. Location map of Biasong-Bogasong-Pangi-Nahulid field photographs. Point (yellow circles) labels correspond to figure numbers. Elevation and shaded relief image composite basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). Contour interval = 20 m.

and mudstones in this group were not distinguished from each other because of their interbedded relationships, hence the collective term fine clastic rocks. Similarly, the breccia-conglomerate group did not require further subdivision, but instead is referred to as coarse clastic rocks, in relation to the fine clastic rocks of the other group.

Site	breccias, conglomerates	sandstones	mudstones	ultramafic rocks	limestones	andesites
a. Hinabian - Malinao - Kauswagan		x	x	x	x	
b. Magatas - Guinsaugon- ... -Panian	x	x	x			x
c. Panian - Himbangan - Hindag-an			x	x	x	
d. Lipanto - Himay-angan		x	x	x	x	
e. San Roque - Tabugon - Molopolo		x		x	x	
f. Himay-angan - Ilag - Nailong		x	x	x	x	
g. Magaupas - Magkasag - Cawayan	x	x	x			
h. Biasong - Bogasong - ... -Nahawong	x	x	x			

Table 1. Summary of mapped lithologies per site. X indicates presence of lithology. Refer to Figure 4 for locations of sites.

The area can now be subdivided into four distinct lithologic groups: coarse clastic rocks (CC) comprised of breccias, conglomerates, sandstones, and mudstones; fine clastic rocks (FC) comprised of sandstones and mudstones; ultramafic rocks (UM); and limestones (LS) (Fig. 39). The limited and unmappable exposures of andesites in the study area, particularly along the path of the Guinsaugon landslide, did not warrant the assignment of a distinct group for this lithology.

2.4.2.2 Stratigraphy

2.4.2.2.1 Hinabian – Malinao – Kauswagan

For the Hinabian-Malinao-Kauswagan area, limestones were observed to form hilltops with either siltstones or ultramafic rocks at the base. This suggests a young age of the limestones relative to the rocks at their base. Assuming a non-fault contact between the ultramafic rocks and the siltstones, the former is assumed to be unconformably overlain

Figure 39. Lithologic map of the study area. Shaded relief image basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*).

by the latter. The intermittent exposures of these two rocks suggest that the siltstones may be of limited thickness or, in some areas, completely eroded to expose the ultramafic rock.

2.4.2.2.2 Magatas – Guinsaugon – Sug-angon – Ayahag – Nueva Esperanza – Panian

The Magatas-Guinsaugon-Sug-angon-Ayahag-Nueva Esperanza-Panian area is underlain mainly by southwest-dipping breccias, sandstones and mudstones. Ultramafic rocks and limestones are not exposed based on a survey of stream sediments draining the upstream areas. The former can only be observed in the south at a stream in Panian, where it is believed to be in contact with breccias and sandstones.

2.4.2.2.3 Panian – Himbangan – Hindag-an

Ultramafic rocks are thrust over siltstones. The ultramafic rocks are in turn, overlain by the limestones.

2.4.2.2.4 Lipanto – Himay-angan

There were no directly observed contacts between the ultramafic rocks, sandstones -siltstones and limestones. Limestones found further upstream in Lipanto may be overlying the ultramafic rocks, as well as the sandstones and siltstones, based on elevation.

2.4.2.2.5 San Roque – Tabugon – Molopolo

The sandstones and breccias in Tabugon unconformably overlie ultramafic rocks and mudstones, based on the clasts in the conglomerate. Individual gastropod fossils, instead of limestone clasts, suggest the absence of limestones older than the conglomerates. The location of the limestones in the area along the coastline and the above inclusion-based evidence suggest that the limestones in Molopolo and San Roque are younger than the ultramafic rocks, mudstones, sandstones and breccias.

2.4.2.2.6 Himay-angan – Ilag – Nailong

In the interiors of Himay-angan-Ilag-Nailong, limestones appear to be overlying ultramafic rocks, sandstones, and siltstones, based on topographic position. However,

sandstones and siltstones exposed near the coast at Nailong contain ultramafic rock and limestone clasts, suggesting the presence of younger fine clastic rocks.

2.4.2.2.7 Magaupas – Magkasag – Cawayan

The area is underlain exclusively by sandstones, conglomerates and breccias. Ultramafic rocks and limestones are not exposed in these areas, and are absent even in stream sediments.

2.4.2.2.8 Biasong – Bogasong – Pangi – Nahulid – Nahawong

The area is underlain exclusively by sandstones, conglomerates and breccias. Ultramafic rocks and limestones are not exposed in these areas, and are absent even in stream sediments.

2.4.2.2.9 Summary

By integrating the stratigraphies in the literature with the results of field mapping for this study, the following model, with reference to the established lithologic groups, (Fig. 40) is proposed:

The ultramafic rocks (UM) serve as the basement for the northeastern and southern subregions. These are often found topographically below the limestones and fine clastic rocks; thrust over fine clastic rocks; and as rounded pebble-sized clasts in fine clastic rocks. The Jurassic to Cretaceous ages cited by previous works for the ophiolitic rocks ((MGB, 1997); (MMAJ-JICA, 1990); Leynes et al., 1996) were adopted for this unit. The ultramafic rocks have minimum vertical thickness of 200-300m based on elevation contours and outcrop locations.

Fine clastic rocks (FC1) overlie the ultramafic rocks based on topographic positions and the close juxtaposition of both rock units. Unconformities and thrust faults define the contacts between the two units. The clastic units may be between Late Cretaceous and Miocene in age based on the ages of the underlying ultramafic rocks and the overlying limestone units. The thickness is difficult to estimate because of the

variable orientation and limited exposures of the fine clastic rocks. However, a minimum thickness of 200 m was measured along a uniformly-dipping section in Lipanto.

Limestones (LS1) at the core of the mountain range in the southern region and at the northeastern region unconformably overlie the ultramafic and fine clastic rocks based on topographic positions and actual contacts. The Early to Middle Miocene ages of the Tertiary Limestone (Leynes et al., 1996) and Danao Limestone (MGB, 1997) were adopted for this unit. A minimum thickness of 150 m is measured from the near-vertical cliff faces in Ilag.

Though unmapped in the area, the Early to Middle Miocene Baybay Volcanic Complex / Central Highland Volcanics (MGB, 1997; MGB, 2010) was assumed to be the basement or source of the coarse clastic rocks (CC) found in the north-central subregion. The coarse clastic rocks were identified with the Pangasugan, Masonting and Dolores Formations, and to the lower clastic members of the Tertiary Clastics of Cabac. A minimum thickness of 1600 m was measured across the Can-abag Ridge.

A younger fine clastic rock (FC2) unit was inferred to unconformably overlie the older ultramafic rocks, fine clastic rocks, and limestones based on the occurrence of these older rocks as clasts of some massive breccia beds within the fine clastic units. A Late Miocene age was designated based on the minimum age of the underlying limestone units.

Limestones exposed near the coastlines of the southern region represent the younger limestones (LS2) based on topographic positions and the reported upper limestone member of the Tertiary Clastics of Cabac (Leynes et al., 1996).

2.4.2.3 Geologic Structures

2.4.2.3.1 Philippine Fault Zone

Linear ridges, scarps and channels that are roughly parallel to the main fault trace of the PFZ suggest the strong influence of this structure on the topography of the study area. The Philippine Fault Zone segment in the study area has a trend of N18W, as traced

Period	Epoch	Stage	Ma (GTS, 2004)	MGB, 2010; MGB, 1997	MMAJ-JICA, 1990	Leynes et al., 1996	THIS STUDY		
Quaternary	Holocene		0.0115	Alluvium	Alluvium	Alluvium	Alluvium/colluvium		
	Pleistocene		1.81						
Neogene	Pliocene	Late	3.6	Dolores Fm. ² / Inopacan Clastics ²		???????? Tertiary Clastics of Cabac	LS2 FC2 CC		
		Early	5.33	Pangasugan Fm. ¹ / Masonting Fm. ³	Masonting Fm. ²	Pangasugan Fm. ¹			
	Miocene	Late	11.61						
		Middle		Calubian Limestone ⁵ / Danao Limestone ²	Baybay Volcanic Complex / Central Highland Volcanics ¹	Danao Limestone ²	Central Highland Volcanics ¹	Tertiary Limestones	LS1
		Early	15.97						
Paleogene	Oligocene		23.03				????????????????		
	Eocene		33.9						
	Paleocene		55.8						
Cretaceous	Late		65.5						
	Early		99.6	Malitbog Ophiolite					
Jurassic			145.5		Serpentinized harzburgite of Malitbog Ophiolite		Cretaceous ultramafics	UM	

¹Pilac, 1965²Floredo, 1987³Corby et al., 1951

Figure 40. Stratigraphy of study area

from Kauswagan in the north up to Himbangan in the south. The expected orientation of deformation features (Fig. 41) from this principal deformation zone, according to a simple shear mechanism of left-lateral strike-slip faulting and a friction angle of 30 degrees Sylvester (1988) are: 1) synthetic R shears oriented N33W, 2) antithetic R' shears oriented N93W, 3) synthetic P shears oriented N03W, 4) thrust faults and fold axes oriented N27E, and 5) extension fractures oriented N63W.

The orientations of the main sliding planes of the Guinsaugon (Cardiel et al., 2008; Catane et al., 2008; Guthrie et al., 2009) and the Magkasag landslides, and the faults and joints along the path of the Guinsaugon landslide (Futalan et al., 2010) are close to the orientations of the principal deformation zone and synthetic R shears of the PFZ. With the young age of the CC unit where these structures are mapped, it is likely that these structures were formed as a result of the deformation due to the PFZ.

Lineaments were traced from the elevation contour map and shaded relief images of the study area (Fig. 43). Orientation (degrees) and length (meters) attributes of the lineaments (n=205) were used to plot a rose diagram with sector size of 5 degrees. Dominant orientations (>6000m cumulative length per sector) are found at N80-90W, N55-60W, N25-35W, N10-20W, N0-5E, N20-35E and N55-65E (Fig. 42). Comparing the above simple shear model to the lineament orientations, all dominant lineament orientations, except for one, have corresponding features in the strike slip model, at least in terms of orientation (Tab. 2). While this correspondence suggests the influence of the PFZ, it does not discount the influence of structures not related to the PFZ such as bedding planes, unconformities, and secondary structures formed earlier than the PFZ.

2.4.2.3.2 Beds and folds

The coarse clastic rocks (CC) have a mean bedding plane orientation of N23W 26SW and resultant length of 0.96 (Fig. 44). In Biasong-Bogasong-Pangi-Nahulid-Nahawong, mean bedding plane orientation is N3W 26SW and resultant length is 0.96 (Fig. 36). In Magaupas-Magkasag-Cawayan, mean bedding plane orientation is N28W 32SW and

Figure 41. Deformation features from strike-slip model. Principal deformation zone (PDZ). Synthetic R shears (R). Antithetic R Shears (R'). P shears (P). Thrust faults and fold axes (T/F). Normal faults (N). Grey arrows indicate principal displacement, whereas block arrows indicate direction of compression and extension.

Figure 42. Rose diagram of lineaments. Sector size=5 degrees

Figure 43. Lineament map. Shaded relief image basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*).

Strike-slip model		Lineaments	
feature	orientation	orientation	Difference with strike-slip model (degrees)
Principal deformation zone	N18W	N10-20W	0
R shear	N33W	N25-35W	0
R' shear	N93W	N80-90W	3
P shear	N03W	N0-5E	3
Thrust fault / fold axis	N27E	N20-35E	0
Normal fault	N63W	N55-60W	3
?	?	N55-65E	?

Table 2. Comparison of lineaments with features of the strike-slip model.

resultant length is 0.98 (Fig. 33). In Guinsaugon, mean bedding plane orientation is N34W 23SW, and the resultant length is 0.98 (Fig. 11). The consistency of dip directions as reflected in the resultant lengths (0=random orientation, 1=uniform orientation) along and across the Can-abag ridge, suggests that the coarse clastic rocks in these areas define a homoclinal structural ridge. With this structural anisotropy, the lithologic group CC may be subdivided into units west and east of the Can-abag ridge. The slopes generally dip in the same direction as the bedding planes in the former unit, whereas the slope and bedding planes dip oppositely in the latter unit.

A geologic section across the Can-abag ridge (Fig. 48) was constructed by the Busk method using measurements of bedding plane orientations at Guinsaugon and Bogasong. This section, underlain exclusively by CC and nearly parallel to the general dip direction of the rock unit, shows bedding planes uniformly dipping southwest with a total minimum thickness of 1600 m. It also shows a younging direction from east to west, assuming there have been no overturning of beds during tectonic deformation.

The fine clastic rocks (FC1,FC2) have a mean bedding plane orientation of N27W 20SW and resultant length of 0.82 (Fig. 45). These rocks have more variably oriented bedding planes than the coarse clastic rocks. At > 3% point density, there are two

Figure 44. Summary of bedding planes in coarse clastic rocks. N=34; resultant length=0.96; Mean orientation N23W 26SW.

Figure 45. Summary of bedding planes in fine clastic rocks. N=34; resultant length=0.82; Mean orientation N27W 20SW; Beta (fold) axis 15 S21W.

clusters for fine clastic rocks, whereas there is only one for the coarse clastic rocks. Fine clastic rocks are also more steeply-dipping. Maximum dip angles for the 3% point density clusters is 79 degrees, compared to the 57 degrees for the coarse clastic rocks. Overall, the two dominant orientations suggest folding of rocks along SW-plunging axes (Fig. 45). The orientation of the fold axes roughly coincides, and may be associated, with the strike-slip model (Tab. 2). This, however, does not preclude a pre-PFZ cause of these deformations.

There were no bedding planes observed in the limestones (LS1,LS2). However, it may be assumed, based on principles of original horizontality and subsequent deformations, that the older limestones (LS1) may possibly have bedding planes similarly oriented as those of the younger coarse clastic rocks.

Figure 46. Thrust faults involving ultramafic rocks over fine clastic rocks. Faults (great circle) N=4; Mean resultant length=0.95; Mean orientation N71E 21SE. Slickenlines (red dots) N=2; Mean resultant length =0.94; Mean orientation 22 S53E (plunge, trend).

2.4.2.3.3 Thrust Faults

Thrust faults mainly involved ultramafic rocks on top of fine clastic rocks. The faults have a mean orientation of N71E 21SE (mean resultant=0.95), and have slickenlines with mean orientation of 22 S53E (plunge/trend) (mean resultant-0.94) (Fig. 46). The SE-dipping faults and SE-trending slickenlines suggest a northwest thrusting direction. These structures may be related to the Late Cretaceous emplacement of the ophiolitic rocks (MGB, 2010), the mid-Oligocene to Miocene compressive events (Aurelio et al., 1991; Metal Mining Agency of Japan and Japan International Cooperation Agency (MMAJ-JICA), 1990) or to the younger deformation caused by the PFZ starting 5mya (Aurelio et al., 1991).

2.4.2.3.4 Lithologic contacts

Contacts between the coarse clastic rocks (CC) and the other rocks have not been directly observed in the field. The contact between CC and ultramafic rocks (UM) in Panian, Himbangan and Magaupas were inferred from contrasting lithology of sediments in streams draining adjacent watersheds. A northeast-southwest trending, curvilinear contact is defined by these localities and may be interpreted as a thrust fault based on published cross sections (Leynes et al., 1996; (MGB, 1997)) and observed thrust faults of Leynes et al. (1996). Alternatively, the contact may also be an unconformity between

the overlying CC and the underlying UM. However, the thrust fault interpretation is more favored based on the abrupt absence of UM across the boundary towards the area of CC, and vice versa.

Contact with the CC can be inferred along the PFZ. The northern areas west of the PFZ is underlain by CC, whereas those east of the PFZ are underlain by UM, limestones (LS1), and fine clastic rocks (FC1).

Fault contacts between the FC1 and UM show a southeast to northeast dip direction at low angles, and together with northwest-southeast trending slickenlines, indicate a northwest-directed thrusting of UM over FC1.

There were no directly observed contacts of LS with other rocks except with UM at Hindag-an, where the former overlies the latter. Elevation differences suggest that LS1 also overlies FC1.

2.4.2.4 Geologic model

The study area consists of a Jurassic to Cretaceous ultramafic basement (UM) overlain by a pre-Miocene to Pliocene sequence of sedimentary rocks (FC1, LS1, FC2, LS2) in the southern and northeastern part of the study area. These formations are juxtaposed against the Late Miocene to Pliocene coarse clastic rocks (CC) in the north-central part of the study area along an inferred thrust fault and along the Philippine Fault Zone. A volcanic basement is inferred for the CC unit, but is unmappable in the area (Figs. 47 and 48).

FC1 is interpreted as either the sedimentary carapace of the ophiolite complex to which UM belongs; or a younger, pre-Miocene unit, unconformably overlying UM. In either case, but more especially for the former, FC1 and UM have been brought into contact by thrust faults during one or more of the following events: Late Cretaceous ophiolite emplacement (MGB, 2010), Oligocene to Miocene compressions (Aurelio et al., 1991), and PFZ-related compression (Aurelio et al., 1991). The inferred folds and steeply-dipping beds of FC1 support these compression events. LS1 unconformably

overlies both FC1 and UM. Depending on the age of the thrust faults, LS1 may or may not have been affected by the structures.

CC is interpreted as the direct product of the volcanoclastic sedimentation derived from the Early to Middle Miocene Baybay Volcanic Complex (MGB, 2010). It may not have shared the same depositional basin as the older UM-FC1-LS1 units based on the drastic changes in topography and underlying lithologic formations. This interpretation thus requires a fault contact to juxtapose CC with UM-FC1-LS1, and constrains the age of the interpreted thrust fault contact to Late Miocene or younger, corresponding to either the end-Miocene or the PFZ-related compression events (Aurelio et al., 1991). Left-lateral strike-slip displacement due to the Philippine Fault Zone is interpreted to have caused the juxtaposition of CC and UM-FC1-LS1 in the northeastern part of the study area. The SW-dipping beds of CC form a homoclinal ridge defining the north to central section of the Can-abag ridge.

The younger fine clastic rocks (FC2), associated with sedimentation from the UM-FC1-LS1, units, may have been deposited at the same time as CC and may have been affected also by the end-Miocene and PFZ compressions (Aurelio et al., 1991), based on the steeply dipping beds. The younger limestones in the south (LS2) were deposited later, unconformably overlying the older formations.

From the above geologic model, a broad classification of geologic materials based primarily on lithology and structures is proposed. The units based on this classification will be referred to as litho-structural units from hereon and in the succeeding chapters. UM is adopted as a distinct litho-structural unit based primarily on lithology. Between the older FC1 and younger FC2, there were no striking differences in the lithology and structures observed in the field that would have indicated significant differences between the geomechanical properties of both units. Same line of reasoning applies for the LS1 and LS2 units. Hence, FC1 and FC2 are treated as one litho-structural unit, FC. Similarly, LS1 and LS2 are treated as one unit, LS. The orientation of the bedding planes of the CC unit in relation to the Can-abag ridge may have imposed differences in material strength

Figure 47. Geologic map of study area. Cross sections A-A' and B-B' shown in Figure 48. Shaded relief image basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*).

and geomorphic processes between areas west and east of the ridge, as suggested by the asymmetric topographic profile of the ridge (Fig. 48). Hence, CC is subdivided into the western and eastern units, CCw and CCE, respectively. A map of the litho-structural units CCE, CCw, FC, LS and UM are shown in Figure 49.

Figure 48. Geologic cross sections. A-A': Section traversing Magkasag-Ilag-Lipanto-Himay-angan-Molopolo-San Roque. B-B': Section traversing Guinsaugon-Bogasong. Refer to Figure 47 for location of section lines. Horizontal and vertical units in meters. Vertical exaggeration = 2.5.

Figure 49. Map of litho-structural units in the study area. Shaded relief image basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*).

2.5 Conclusion

A geologic model comprised of six formations (UM, FC1, LS1, CC, FC2, and LS2) which have undergone deformations due to ophiolite emplacement, regional compression and strike-slip faulting is proposed for the study area. Five litho-structural units are derived from this model: coarse clastic rocks west and east of the Can-abag ridge (CCw, CCe), fine clastic rocks (FC), limestones (LS) and ultramafic rocks (UM). These litho-structural units, each representing distinct geomechanical properties, will be characterized in terms of topography, rock mass strength and stability conditions in the succeeding chapters.

CHAPTER 3

TOPOGRAPHY

3.1 Introduction

Topographic relief, the spatial variation of surface elevation, serves as an indicator of geomorphic processes and earth material properties. It may bear the signature of past processes, a classic example of which, is the U- and V-shaped valleys carved by glaciers and streams, respectively (Dietrich et al., 2003; Easterbrook, 1999). It may also reflect differences in the properties of materials at the surface, such as rock mass strength, as seen in the varying slopes of escarpments underlain by different lithologies (Moon and Selby, 1983). The objective of this chapter is to characterize the topography of areas underlain by the litho-structural units identified in the previous chapter. Results will be used to distinguish one unit from another, and to qualitatively indicate dominant geomorphic processes and surface material properties.

3.2 Review of Literature

3.2.1 Topographic characterization

3.2.1.1 Morphometric parameters and features

Based on the characterization of points on a surface (elevation, aspect, gradient and curvature) (Evans, 1972), and the representation of a surface as a bivariate quadratic function of elevation (z) and horizontal positions (x,y) (Evans et al., 1979),

$$z = ax^2 + by^2 + cxy + dx + ey + f \quad (3.1)$$

Wood (1996) presented equations for determining the elevation-derived morphometric parameters (Tab. 3). He discussed the use of these to classify a topographic surface into one of the six morphometric features: plane, ridge, channel, pit, peak, pass (Tab. 4).

Results of his work have been translated into the GRASS-GIS module, *r.param.scale* (GRASS Development Team, 2012).

Wood (1996) distinguishes at least six types of curvatures based on the plane along which the measurement is defined. These are the profile, plan, longitudinal, cross sectional, minimum, and maximum curvatures. However, he identifies cross sectional curvature as the curvature index most related to geomorphological processes such that the classification system in Table 4, involving mainly gradient and cross sectional curvature, has been shown to have better results than an earlier version, where longitudinal curvature was also a factor. In the current version, the misclassifications encountered when longitudinal curvature is factored are addressed, and the continuity of linear features such as ridges and channels are improved.

3.2.1.2 Statistical analysis of topographic data

Strahler discussed the application of statistical tools in the quantitative description of topographic data over a certain area (Strahler, 1954; Strahler, 1956). Data is presented visually as a histogram, and described in terms of the mean, standard deviation, variance and type of the distribution, whether it is normal, log-normal or otherwise. A method for comparing the differences between the means of two samples using the Central Limit Theorem and the t-test was presented. A similar comparison of standard deviations using the F-test was also presented. Normality is tested using the chi-squared test. While the methods employed were straightforward use of basic statistical principles and equations, Strahler's work provided a blueprint for carrying out quantitative analysis in geomorphological work.

3.2.2 Relationships between topography, surface processes and materials

Topographic variation has been shown to correspond to changes in surface processes and geologic properties in the Oregon Coast Range by Roering et al. (2005). They characterized deep-seated landslides as surfaces with low gradient and low absolute

Parameter	Description	Equation
gradient	1 st derivative of elevation measured along a vertical plane parallel to the steepest downslope direction; it is the magnitude of the vector representing the resultant of the rate of change in elevation along the x and y directions (both horizontal);	$\arctan \sqrt{d^2 + e^2} \quad (3.2)$
aspect	downslope direction defined by a surface, measured as horizontal angle (azimuth); it is the direction of the vector representing the resultant of the rate of change in elevation along the x and y directions (both horizontal)	$\arctan \left(\frac{e}{d} \right) \quad (3.3)$
cross sectional curvature	2 nd derivative of elevation measured along an intersecting plane defined by the surface normal and the perpendicular aspect direction.	$-2 \left(\frac{bd^2 + ae^2 - cde}{d^2 + e^2} \right) \quad (3.4)$
maximum curvature	2 nd derivative of elevation along an intersecting plane where curvature is maximized	$-a - b + \sqrt{(a - b)^2 + c^2} \quad (3.5)$
minimum curvature	2 nd derivative of elevation along an intersecting plane where curvature is minimized;	$-a - b - \sqrt{(a - b)^2 + c^2} \quad (3.6)$

Table 3. Morphometric parameter descriptions and equations. From Wood (1996).

morphometric feature	gradient	cross-sectional curvature	maximum curvature	minimum curvature
peak	0	undefined	+	+
ridge	0	undefined	+	0
	+	+		
pass	0	undefined	+	-
plane	0	undefined	0	0
	+	0		
channel	0	undefined	0	-
	+	-		
pit	0	undefined	-	-

Table 4. Simplified classification of morphometric features based on values of gradient and curvatures. Zeros (0) indicate parameter values within $\pm$ threshold values; positive and negative signs (+,-) indicate parameter values $>$ threshold values, and $<$ -(threshold values), respectively. Maximum and minimum curvatures are used when cross sectional curvature is undefined. Setting the gradient threshold value very close to zero reduces the basis for classification to gradient and cross sectional curvature, and practically reduces the morphometric parameter classes to planes, channels and ridges. From Wood (1996).

curvatures. Using this topographic signature, they found that the proportion of the area affected by such process, increases to the north, and conversely, the proportion affected by debris flow and fluvial incision (high absolute curvature surfaces) increased to the south. Comparing this trend to facies and structural maps, they concluded that deep-seated landslides, are more common in the weaker, silt-rich portion of the Tyee formation in the north, and occur more frequently in the direction of bedding dips and with increasing dip angle.

Schneider et al. (2008) showed how topography reflects the ratio of sediment transport via channelized processes (fluvial- and debris flow-transport) to sediment transport via hillslope processes (landslides). A landscape evolution model (Simpson and Schlunegger, 2003) integrating both channelized and hillslope transport is used to generate a landscape. Model results for more dominant channel processes have “rough, high-relief, highly incised surfaces, that contain a dense and hence closely spaced channel network and highly branched valleys”, whereas the landslide-dominated landscapes have “smooth topographies, low relief, greater channel spacing, and less sinuous valleys” (Schneider et al., 2008). This process-form relationship was found to to be consistent in a

study area in the eastern Swiss Alps (Schneider et al., 2008). Four adjacent N-S trending valleys in the eastern Swiss Alps exhibit contrasting morphometries between their east- and west-facing valley sides which respectively correspond to landforms resulting from hillslope- and channel-dominated transport processes modeled by Simpson and Schlunegger (2003). The authors attribute this contrasting patterns to the southeast-dipping shales and sandstones which cause enhanced landsliding (hillslope process) in the east-facing valley sides.

In a study of the northern Cascades Range in Washington and the Santa Cruz Mountains in California, Schmidt and Montgomery (1995) demonstrated that the “relief of mountain ranges can reflect landscape-scale material strength”. Using the Cullman two-dimensional limit equilibrium slope stability model, the authors back-calculated the large-scale strength parameters (friction angle, ϕ , and cohesion, c) of slopes from inventories of maximum slope heights and gradients measured in the field. The model is used to predict the critical slope height at a given slope gradient for a particular set of strength parameters. The calculated strength parameters, ϕ and c , were shown to correspond to laboratory- and field-measured strengths of the weakest members of the rock formations in the area, thus concluding that rock strength, at large scales, can be low enough to limit relief. The authors also showed that dip slopes have lower back-calculated strengths than anti-dip slopes.

Burbank and Pinter (1999) noted slope distributions with a uniform mean slope of 32 ± 2.2 degrees in the middle gorge of the Indus River near Nanga Parbat, northern Pakistan, in the northwestern Himalayas despite variable bedrock uplift and denudation rates. The authors add that slopes respond to the rapid incision by the Indus River, which exceed 2 mm/year , by bedrock landsliding. In this setting, they suggest that the mean slope angles are controlled by rock strength.

In New Zealand, Korup (2008) illustrated how the regional gradient distributions remain fairly constant for terrains with uniform geology. The invariance of gradient distributions (uniform modal gradient) vis-a-vis orders of magnitudes of variations in

uplift and erosion rates in those areas indicate that the region has achieved threshold conditions where the lithology determines topographic relief. Furthermore, he presented other areas underlain by different lithologies and showed significant difference in their gradient distributions. Areas underlain by granite and gneiss have the highest modal gradients, followed by those underlain by greywacke and schist, and then by those underlain by siltstones and mudstones. The author notes how rock mass strength follows a similar trend for these areas (Augustinus, 1995; Reid and Page, 2003). Korup thus concludes that modal gradient can be a proxy for rock mass strength in threshold, landslide-dominated topography of uniform geology. He, however, adds that future work is needed to calibrate the relationship between actual rock mass strengths and their topographic signature.

Clarke and Burbank (2010) expanded the work of Korup (2008) by characterizing the rock mass strength of two different lithologic groups in the Fiordland and western Southern Alps of New Zealand. The former area is underlain by granite and gneiss, whereas the latter is underlain by schist and greywacke which have lower intact rock strengths. The authors showed, through measurements in P-wave velocities in intact rock (laboratory) and in fractured rock (field), that the granite and gneiss are more fractured than the schist and greywacke. They explained that while the intact rock strength is different for the two rock groups, differential fracturing resulted to the two rock groups having near-equal rock mass strengths. This explains the similarity in the modal gradient values of the two areas despite differences in underlying lithology.

Kühni and Pfiffner (2001) discusses elevation, relief and gradient variations in the Swiss Alps and relates these to the erodibility of underlying lithologies. In general, areas underlain with low to very low erodibility lithologies such as granite and gneiss have higher elevation, relief and average slopes than those underlain by medium to high erodibility lithologies such as sandstone, shale, slates, flysch and carbonates. The authors also noted an almost constant average slope (25 degrees) within the elevation intervals of

1700 – 2900 m, and suggested that this reflects a mean critical slope angle and a regional stability threshold for the Swiss Alps.

Densmore et al. (1997) conducted analogue modeling to illustrate how bedrock landslides influence hillslope evolution. In the experiments, red and white beans were used to demonstrate the effect of different strengths owing to anisotropy in block shape. The former are elongate and tended to settle with their longest axis in a horizontal position. This produced a crude stratification which imparted greater stability to the pile compared to that formed by the white beans. The results both showed small failures originating near the toe, which later propagated up the slope to instantaneously remove a large mass. In the red beans set-up, individual large failures, referred to as “slope-clearing events” (SCE’s) were less frequent but have greater mass than those in the white beans set-up. Also, during the course of the experiment, formation of steep toes and scarps were more common in the red beans set-up. The results suggest that slope materials with higher shear strength (red beans) can maintain steeper slopes and have greater resistance to shear failure. While the greater resistance reduces the frequency of large failures, the individual failure tends to be of a greater magnitude.

3.3 Methods

3.3.1 Overview

Morphometric parameters and feature classifications were generated from elevation data, and subdivided according to their corresponding litho-structural units. For each unit and morphometric parameter, the mean, median, mode and standard deviation were computed, and the distribution was analyzed. From these results, presented as maps, graphs and tables, the litho-structural units were described and distinguished from each other, and characteristic geomorphic processes and material properties were inferred.

Figure 50. Morphometric parameter and feature generation in GRASS-GIS.

3.3.2 Generating morphometric parameters and features

Morphometric parameters and feature classifications were generated from the NAMRIA digitized 1:50000 20-m elevation contour map of Southern Leyte using GRASS-GIS 6.4.3 (GRASS Development Team, 2012). Contour lines were imported, converted to vector point data, and interpolated to generate a raster elevation data with 10-m grid resolution or pixel edge width. Gradient, aspect, curvature, and morphometric features rasters were computed from the elevation raster, and these data were exported to text-format grids for further processing. The above procedures and the relevant GRASS-GIS modules are presented as a flowchart (Fig. 50). GRASS Development Team (2012) provides further discussion of the individual modules used.

3.3.3 Analysis of morphometric parameters and features

Spatial distribution of morphometric parameters and features were visually assessed using maps produced in GRASS-GIS. Statistical analysis was implemented in customized Python 2.7 codes (Python Software Foundation, 2014) using the core packages of the Scipy Stack (Jones et al., 2011) (Fig. 51). Python is “a high-level general-purpose programming language that can be applied to many different classes of problems...[It] combines remarkable power with very clear syntax.” (Python Software Foundation, 2014). The Scipy Stack is “a collection of open source software for scientific computing in Python” (Jones et al., 2011). Its core libraries include Scipy (Jones et al., 2011), Numpy (Walt et al., 2011), Matplotlib (Hunter, 2007), and Pandas (McKinney, 2010). The use of Python and the Scipy Stack was necessary since the analysis involved large volume of data and specialized operations which are difficult to implement in a GIS environment such as GRASS-GIS, or in a typical office spreadsheets application.

The text-format grids were imported into Numpy arrays, converted to text files, and then imported and integrated into a single Pandas dataframe. In the dataframe, data were arranged in rows and columns. Each row represents one pixel in the map, and the columns contain the corresponding morphometric parameters, morphometric feature class, and litho-structural classification of the pixel.

For each morphometric parameter and feature class (column in dataframe), data was subdivided based on the litho-structural units. Each subdata was treated as a population corresponding to the litho-structural unit. For each population, the cumulative density function, (CDF) derived from the normalized cumulative histogram, was plotted. The probability density function (PDF), similar to a smoothed normalized histogram and determined using gaussian kernel density estimation (Scott, 1992), was also plotted.

Population parameters mean, median, standard deviation and mode were computed. The first three are straightforward implementations of statistical functions. The last, however, was derived from the parameter value where the PDF is maximum. The

Figure 51. Statistical analysis of morphometric parameters and feature classifications in Python.

normality of the distribution was determined using the Kolmogorov-Smirnov Goodness-of-Fit Test (KS test)(Kolmogorov, 1992; Massey, 1951; Smirnov, 1939). In this test, the CDF of a random sample from the distribution ($N = 1000$) is compared with the CDF of a normal distribution defined by the mean and standard of the distribution. The null hypothesis, H_0 , of the KS test is that the distribution is derived from a normal population. Rejection of this hypothesis means the distribution is not normal. For this and other succeeding hypothesis tests, a significance level, $\alpha = 0.05$ (5%), was used. P-values returned by the tests that are less than α result to rejection of the null hypotheses.

For each population parameter (mean, median, mode, standard deviation), a sampling distribution of the corresponding statistic was performed. Based on the Central Limit Theorem (Cam et al., 1986; Feller, 1945), the mean of a sampling distribution of a statistic, provides an estimate of the corresponding population parameter. For example, the mean of the sampling distribution of the median provides an estimate of the population median. For sufficiently large sample sizes, the sampling distribution becomes normal and its mean closely approaches the population median. Because the sampling distributions provide an estimate of population parameters, these can be used to test for differences in population parameters.

Each sampling distribution (number of samples=100, sample size=50) is expected to follow a normal distribution based on the Central Limit Theorem (Cam et al., 1986; Feller, 1945), but was still tested for normality using the KS test. Mean and standard deviation were computed. The distribution was plotted as an empirical CDF, a normal CDF, and a normal PDF. Difference between the means of the sampling distributions were determined from the Welch's Test (Welch, 1947), a modified Student's T-test for samples for non-equal variances. For this test, the H_0 is that the two samples being compared have equal means. Rejection of the null hypotheses means that the corresponding population parameters are not equal.

The population distributions were also compared and the differences were assessed using the KS test on random samples ($N=1000$) from each of the distributions

being compared. The H_0 is that the two samples are derived from the same distributions. Rejection of the null hypothesis means that the two distributions are not the same.

3.4 Results and Discussion

3.4.1 Morphometric Analysis

3.4.1.1 Elevation

The elevation map (Fig. 52) shows generally decreasing values from the north towards the south. UM, FC and LS in the south generally have lower elevations compared to CCe and CCw. High local variability can be observed on the western side, within the CCw unit. Large, uniform drop of elevation can be observed close to the main ridge, within the CCe unit, and this abruptly changes to low, uniform elevations towards the alluvial valley in the east.

Density plots (Fig. 53) and KS goodness-of-fit tests (Tabs. 5b, 5a) show mutually distinct, non-normal population distributions for all litho-structural units. All population parameter estimates, except for the mode of CCe, approach the normal distribution (Tab. 6, Fig. 54) and are close to the actual population parameters (Tab. 5a). Differences of means, medians, modes and standard deviations for all litho-structural unit pairs are significant (Tab. 6), as clearly shown by the sampling distribution plots (Fig. 54).

FC has the smallest mean, median, mode and standard deviation, whereas CCe has the highest values. The units, arranged in increasing order of mean elevation are FC, UM, LS, CCw, and CCe. The order of the litho-structural units is roughly the same for the mean, median, and mode, except from some switching between LS and CCw (Fig. 54, Tab. 6).

3.4.1.2 Gradient

The gradient map (Fig. 55) shows a trend similar to the elevation map. It has generally decreasing values from the north towards the south. UM, FC and LS in the south generally have lower gradients compared to CCe and CCw. High local variability can

Figure 52. *Left:* Elevation map (units in *m*). Shaded relief image basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). *Right:* map of litho-structural units for reference.

Figure 53. Elevation population distribution expressed as cumulative (top) and probability (Bottom) density plots.

Unit	Population						
	N	Mean	Median	Mode	Std Dev	KS normal test	
						P-value	H ₀ : normal
CCw	694,584	285	241	97	200	0.00	FALSE
CCe	257,203	409	413	463	213	0.01	FALSE
FC	102,696	117	103	25	83	0.00	FALSE
LS	111,385	259	267	203	129	0.00	FALSE
UM	170,656	191	141	101	146	0.00	FALSE

(a)

Unit	KS 2-sample test							
	P-value				H ₀ : same distribution			
	CCe	FC	LS	UM	CCe	FC	LS	UM
CCw	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
CCe		0.00	0.00	0.00		FALSE	FALSE	FALSE
FC			0.00	0.00			FALSE	FALSE
LS				0.00				FALSE

(b)

Table 5. Elevation distribution analysis. (a) Population parameters (in *m*) and hypothesis test for normal distribution. (b) Hypothesis test for equal distributions.

Figure 54. Sampling distribution plots of elevation. *Top*: cumulative density function. *Bottom*: probability density function

Sample Statistic	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P- value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
Mean	CCw	287	28	0.75	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	408	30	0.59	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	119	10	0.69	TRUE			0.00	0.00			FALSE	FALSE
	LS	259	18	0.95	TRUE				0.00				FALSE
	UM	193	19	0.79	TRUE								
Median	CCw	244	38	0.86	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	409	44	0.11	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	106	14	0.80	TRUE			0.00	0.00			FALSE	FALSE
	LS	265	27	0.94	TRUE				0.00				FALSE
	UM	144	21	0.81	TRUE								
Mode	CCw	170	47	0.21	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	386	99	0.00	FALSE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	91	27	0.13	TRUE			0.00	0.00			FALSE	FALSE
	LS	285	57	0.05	TRUE				0.00				FALSE
	UM	103	14	0.57	TRUE								
Std dev	CCw	198	15	0.14	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	208	15	0.92	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	83	10	0.73	TRUE			0.00	0.00			FALSE	FALSE
	LS	125	9	0.56	TRUE				0.00				FALSE
	UM	145	12	0.31	TRUE								

Table 6. Elevation sampling distribution parameters (in *m*), test for normality of sampling distribution, and test for equal estimates of the mean.

be observed on the western side, within the CCw unit. Large continuous areas of steep slopes are found close to the main ridge, within the CCe unit, and these abruptly grade into low, uniform gradient towards the alluvial valley in the east.

Density plots (Fig. 56) and KS goodness-of-fit test (Tab. 7b) show mutually distinct population distributions for all litho-structural unit pairs except for that between CCe and CCw. LS and CCw have non-normal distributions (Tab. 7a). All population parameter estimates, approach the normal distribution (Tab. 8, Fig. 57) and are close to the actual population parameters (Tab. 7a). Differences of means, medians, modes and standard deviations for all litho-structural unit pairs are significant , except for the medians of CCe-CCw and FC-LS (Fig. 57, Tab. 8).

For gradient, all parameters can generally be used to distinguish litho-structural units from each other. The units, arranged in increasing order of mean gradient are FC, LS, UM, CCw, CCe. FC has the lowest median, whereas LS has the lowest mode. In terms of standard deviation, the sequence is increasing from FC, UM, LS, CCw, to CCe (Fig. 57, Tab. 8).

Figure 55. *Left:* Slope gradient map (units in *degrees*). Shaded relief image basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). *Right:* map of litho-structural units for reference.

Figure 56. Gradient population distribution expressed as cumulative (*top*) and probability (*Bottom*) density plots.

Unit	Population						
	N	Mean	Median	Mode	Std Dev	KS normal test	
						P-value	H ₀ : normal
CCw	694,584	25	25	22	12	0.04	FALSE
CCe	257,203	26	26	25	12	0.29	TRUE
FC	102,696	18	18	19	8	0.11	TRUE
LS	111,385	19	18	15	11	0.00	FALSE
UM	170,656	20	20	19	10	0.24	TRUE

(a)

Unit	KS 2-sample test							
	P-value				H ₀ : same distribution			
	CCe	FC	LS	UM	CCe	FC	LS	UM
CCw	0.16	0.00	0.00	0.00	TRUE	FALSE	FALSE	FALSE
CCe		0.00	0.00	0.00		FALSE	FALSE	FALSE
FC			0.01	0.00			FALSE	FALSE
LS				0.00				FALSE
UM								

(b)

Table 7. Gradient distribution analysis. (a) Population parameters (in *degrees*) and hypothesis test for normal distribution. (b) Hypothesis test for equal distributions.

Figure 57. Sampling distribution plots of gradient. *Top*: cumulative density function. *Bottom*: probability density function

Sample Statistic	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P- value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
Mean	CCw	25.1	1.5	0.57	TRUE	0.02	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	25.6	1.6	0.94	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	18.1	1.1	0.23	TRUE			0.00	0.00			FALSE	FALSE
	LS	19.4	1.6	0.97	TRUE				0.00				FALSE
	UM	20.4	1.4	0.94	TRUE								
Median	CCw	24.9	2.2	0.92	TRUE	0.08	0.00	0.00	0.00	TRUE	FALSE	FALSE	FALSE
	CCe	25.5	2.1	0.80	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	18.1	1.3	0.45	TRUE			0.59	0.00			TRUE	FALSE
	LS	18.2	2.1	0.90	TRUE				0.00				FALSE
	UM	20.3	1.9	0.90	TRUE								
Mode	CCw	24.4	5.3	0.70	TRUE	0.05	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	25.9	5.0	0.80	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	18.6	2.7	0.99	TRUE			0.00	0.01			FALSE	FALSE
	LS	17.0	3.2	0.71	TRUE				0.00				FALSE
	UM	20.0	4.0	0.94	TRUE								
Std dev	CCw	11.4	1.0	0.91	TRUE	0.04	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	11.7	1.1	0.79	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	8.4	0.9	0.38	TRUE			0.00	0.00			FALSE	FALSE
	LS	10.5	1.1	0.61	TRUE				0.00				FALSE
	UM	9.6	0.9	0.60	TRUE								

Table 8. Gradient sampling distribution parameters (in *degrees*), test for normality of sampling distribution, and test for equal estimates of the mean.

3.4.1.3 Curvature

The curvature map (Fig. 58) shows extreme positive and negative values from the north approaching near-zero values towards the south. UM, FC and LS in the south generally have values closer to zero compared to CCe and CCw. High local variability can be observed on the western side, within the CCw unit. The steep slopes found close to the main ridge, within the CCe unit, have values close to zero.

Density plots (Fig. 59) and KS goodness-of-fit tests (Tab. 9a) show non-normal population distributions for all litho-structural units. All unit pairs except for that between CCe and CCw, CCe and FC, and UM and FC have mutually distinct distributions (Tab. 9b). All population parameter estimates approach the normal distribution (Tab. 10, Fig. 60) and are close to the actual population parameters (Tab. 9a). Differences of means and medians are insignificant, except for pairs involving LS. For the modes, only LS-UM are distinguishable. Differences of standard deviations are significant for all unit pairs, except for LS-UM. (Fig. 60, Tab. 10).

Unit	Population						
	N	Mean	Median	Mode	Std Dev	KS normal test	
						P-value	H ₀ : normal
CCw	694,584	33	30	29	736	0.00	FALSE
CCe	257,203	36	30	27	631	0.00	FALSE
FC	102,696	34	20	37	511	0.00	FALSE
LS	111,385	78	50	17	552	0.00	FALSE
UM	170,656	34	30	24	541	0.00	FALSE

(a)

Unit	KS 2-sample test							
	P-value				H ₀ : same distribution			
	CCe	FC	LS	UM	CCe	FC	LS	UM
CCw	0.21	0.01	0.00	0.00	TRUE	FALSE	FALSE	FALSE
CCe		0.16	0.03	0.03		TRUE	FALSE	FALSE
FC			0.03	0.49			FALSE	TRUE
LS				0.04				FALSE
UM								

(b)

Table 9. Cross sectional curvature distribution analysis. (a) Population parameters (in $10^{-5} m^{-1}$) and hypothesis test for normal distribution. (b) Hypothesis test for equal distributions.

For the cross sectional curvatures of the litho-structural units, the standard deviation is the primary discriminant. In increasing order of standard deviation values, the sequence is from FC, UM/LS, CCe, to CCw. UM and LS can be distinguished from each other by the higher mean, median and mode of the latter.

3.4.1.4 Morphometric features

The morphometric feature classifications are generally consistent with the rule of “V”’s used in topographic contour interpretation. Channel pixels generally coincide with contour lines with the “V” pointing towards higher elevation; ridge pixels with contour lines with the “V” pointing towards lower elevation; and plane pixels with sub-parallel contour lines. The morphometric feature map (Fig. 61) shows mostly plane, channel, and ridge features. The number of pits, peaks and passes are negligible in relation to the total number of pixels. Since feature classes are mutually exclusive, the combined

Figure 58. *Left:* Slope curvature map (units in m^{-1}). Convex (ridges, peaks) and concave (channels, depressions) surfaces are in the red and blue ends of the spectrum, respectively. Highly curved surfaces often occur adjacent to each other, and appear as violet (mix of red and dark blue), whereas less curved surfaces appear as light green (mix of yellow and light blue). Slope gradient image basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). *Right:* map of litho-structural units for reference.

Figure 59. Cross sectional curvature population distribution expressed as cumulative (*top*) and probability (*Bottom*) density plots.

Sample Statistic	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P-value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
Mean	CCw	26.6	95.1	0.88	TRUE	0.62	0.81	0.00	0.29	TRUE	TRUE	FALSE	TRUE
	CCe	32.9	86.0	0.99	TRUE		0.77	0.00	0.56		TRUE	FALSE	TRUE
	FC	29.5	1.1	0.84	TRUE			0.00	0.36			FALSE	TRUE
	LS	77.7	1.6	0.25	TRUE				0.00				FALSE
	UM	39.8	1.4	0.98	TRUE								
Median	CCw	30.1	68.8	0.97	TRUE	0.97	0.73	0.02	0.57	TRUE	TRUE	FALSE	TRUE
	CCe	30.5	73.8	0.50	TRUE		0.71	0.02	0.55		TRUE	FALSE	TRUE
	FC	26.7	72.1	0.46	TRUE			0.01	0.84			FALSE	TRUE
	LS	55.1	77.8	0.55	TRUE				0.00				FALSE
	UM	24.7	63.1	0.94	TRUE								
Mode	CCw	29.9	85.5	0.79	TRUE	0.71	0.65	0.19	0.20	TRUE	TRUE	TRUE	TRUE
	CCe	25.2	95.4	0.89	TRUE		0.95	0.11	0.43		TRUE	TRUE	TRUE
	FC	24.4	86.4	0.98	TRUE			0.08	0.44			TRUE	TRUE
	LS	46.6	92.3	0.82	TRUE				0.01				FALSE
	UM	15.7	70.7	0.95	TRUE								
Std dev	CCw	715.9	126.5	0.78	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	611.7	86.5	0.98	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	490.6	67.7	0.91	TRUE			0.00	0.00			FALSE	FALSE
	LS	532.0	89.3	0.19	TRUE				0.09				TRUE
	UM	530.3	105.7	0.43	TRUE								

Table 10. Cross sectional curvature sampling distribution parameters (in $10^{-5} m^{-1}$), test for normality of sampling distribution, and test for equal estimates of the mean.

Figure 60. Sampling distribution plots of cross sectional curvature. *Top*: cumulative density function. *Bottom*: probability density function

proportions of ridges and channels increases as the proportion of planes decreases. The trend in the map generally shows decreasing ridges and channels, and increasing planes from north to south. This indicates a more pronounced ridge-and-channel topography for the CCE and CCw units, than in the FC, LS and UM units.

Analyzing the combined ridge and channel features (Fig. 62, Tab. 11) with variable threshold curvatures (0.0005, 0.001, 0.002, and $0.005\ m^{-1}$), there is a general decrease of ridges and channels as the threshold increases. In all threshold values, the differences in mean proportion of pairs involving FC, LS, or UM with either CCE or CCw are significant; and the difference between the mean proportions of UM and LS is not significant. CCE and CCw mean proportions become distinct at threshold curvatures $\leq 0.002\ m^{-1}$. Difference in mean proportions among LS, FC and UM pairs are not significant for threshold curvatures $\leq 0.002\ m^{-1}$.

CCE and CCw can be distinguished from the rest of the units based on the generally higher proportions of ridge and channels or high-curvature features. Distinctions within the high-curvature units and the low curvature units become evident as the threshold curvature increases. The units are arranged in order of increasing ridge and channel proportions, from FC, LS/UM, CCE, to CCw.

Analyzing channel features only (Fig. 63, Tab. 12) with variable threshold curvatures (0.0005, 0.001, 0.002, and $0.005\ m^{-1}$), there is a general decrease of channels as the threshold increases. In all threshold values, the differences in mean proportion of pairs involving LS are significant; and the difference between the mean proportions of UM-FC is not significant. CCE and CCw mean proportions become distinct at threshold curvatures $\geq 0.002\ m^{-1}$.

CCE and CCw can be distinguished from the rest of the units based on the generally higher proportions of channels. Similarly, LS is distinguished from the rest with generally lower proportions of channels. These distinctions become more evident as the threshold curvature increases. The units, arranged in order of increasing channel proportions are LS, FC/UM, CCE, and CCw.

For each feature classification (channel, ridge, plane) at various threshold curvatures (0.0005, 0.001, 0.002, and 0.005 m^{-1}), the distribution of representative morphometric parameters are analyzed (Figs. 64, Tabs. 13a and 14a). Mean curvatures are analyzed for the ridge and channel features whereas mean gradient is assessed for plane features. In this analysis, the distinctions are more evident than those for the proportions of features.

Except for at most two instance in UM-LS and LS-FC, differences in curvatures of ridges and channels for all units are significant. The units, arranged in order of increasing channel mean absolute curvature: $FC < LS < UM < CCe < CCw$. The units, arranged in order of increasing ridge mean absolute curvature: $FC < UM < LS < CCe < CCw$.

Across all threshold curvatures, differences in mean gradient of planar features for all litho-structural units are significant. The units, arranged in order of increasing mean gradient: $FC < LS < UM < CCw < CCe$.

3.4.1.5 Summary

To summarize the results of the morphometric analysis, the parameters where the maximum distinction between litho-structural units are plotted against each other. For elevation and gradient, the mean values are used, whereas the standard deviation is used for curvature. For the ridge and channel proportion, channel proportion, channel curvature, ridge curvature and plane gradient, the mean values are used. Standard errors, which come from the standard deviation of the corresponding sampling distributions, are used to plot error bars.

There are increasing trends for the plots of mean gradient and curvature standard deviation against mean elevation (Fig. 65), although the trend is less pronounced in the latter. The plot is consistent with visual assessments of the elevation, gradient and curvature maps, wherein there are decreasing values from north to south. The general trend is expected, since gradient and curvature are 1st and 2nd order derivatives of

Figure 61. *Left:* Morphometric features map of study area overlaid on hillshade relief map for emphasis, in UTM WGS84. Topographic data from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*). Ridges, channels and planes are colored yellow, blue and gray, respectively. Threshold curvature and gradient values used for classification are 0.005/m and 1 degree, respectively. *Right:* map of litho-structural units for reference.

Threshold curvature	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P-value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
0.0005	CCw	90.4	2.9	0.18	TRUE	0.09	0.00	0.00	0.00	TRUE	FALSE	FALSE	FALSE
	CCe	89.7	2.6	0.18	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	87.5	3.3	0.73	TRUE			0.57	0.83			TRUE	TRUE
	LS	87.8	3.4	0.35	TRUE				0.71				TRUE
	UM	87.6	3.0	0.21	TRUE								
0.001	CCw	81.8	3.3	0.38	TRUE	0.04	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	80.8	3.5	0.11	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	77.1	4.3	0.49	TRUE			0.66	0.73			TRUE	TRUE
	LS	77.4	4.0	0.09	TRUE				0.92				TRUE
	UM	77.3	4.0	0.20	TRUE								
0.002	CCw	67.2	4.5	0.25	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	65.1	4.4	0.39	FALSE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	59.9	4.3	0.49	TRUE			0.74	0.47			TRUE	TRUE
	LS	59.7	4.8	0.65	TRUE				0.70				TRUE
	UM	59.4	5.1	0.63	TRUE								
0.005	CCw	36.4	5.1	0.78	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	33.5	5.3	0.45	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	25.2	3.8	0.43	TRUE			0.01	0.00			FALSE	FALSE
	LS	26.9	4.6	0.49	TRUE				0.70				TRUE
	UM	27.2	5.0	0.80	TRUE								

Table 11. Channel and ridge proportion sampling distribution parameters (in *percent*), test for normality of sampling distribution, and test for equal estimates of the mean.

Threshold curvature	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P-value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
0.0005	CCw	42.1	5.1	0.84	TRUE	0.54	0.22	0.00	0.02	TRUE	TRUE	FALSE	FALSE
	CCe	41.7	4.5	0.16	TRUE		0.47	0.00	0.06		TRUE	FALSE	TRUE
	FC	41.2	5.5	0.51	TRUE			0.57	0.32			FALSE	TRUE
	LS	38.3	6.0	0.37	TRUE				0.01				FALSE
	UM	40.5	4.9	0.26	TRUE								
0.001	CCw	38.4	4.9	0.51	TRUE	0.12	0.01	0.00	0.00	TRUE	FALSE	FALSE	FALSE
	CCe	37.3	4.5	0.22	TRUE		0.16	0.00	0.01		TRUE	FALSE	FALSE
	FC	36.4	5.2	0.67	TRUE			0.00	0.33			FALSE	TRUE
	LS	33.2	5.6	0.75	TRUE				0.00				FALSE
	UM	35.7	5.0	0.48	TRUE								
0.002	CCw	31.5	4.8	0.86	TRUE	0.01	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	29.7	4.2	0.15	TRUE		0.03	0.00	0.00		FALSE	FALSE	FALSE
	FC	28.4	4.3	0.56	TRUE			0.00	0.07			FALSE	TRUE
	LS	24.7	4.8	0.30	TRUE				0.00				FALSE
	UM	27.1	5.2	0.72	TRUE								
0.005	CCw	17.3	4.2	0.28	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	15.0	3.9	0.14	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	11.7	3.0	0.21	TRUE			0.00	0.74			FALSE	TRUE
	LS	10.3	3.6	0.44	TRUE				0.00				FALSE
	UM	11.9	3.6	0.14	TRUE								

Table 12. Channel proportion sampling distribution parameters (in *percent*), test for normality of sampling distribution, and test for equal estimates of the mean.

Figure 62. Sampling distribution plots of proportion of ridges and channels, for increasing values of threshold curvature (*left to right*). *Top:* cumulative density function. *Bottom:* probability density function

Figure 63. Sampling distribution plots of proportion of channels for increasing values of threshold curvature (*left to right*). *Top*: cumulative density function. *Bottom*: probability density function

Figure 64. Sampling distribution plots of mean curvatures of channels (*Left* column) and ridges (*Middle* column), and mean gradient of planes (*Right* column) for various threshold curvatures, T_c

Feature, statistic	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P-value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
ridge, mean curvature	CCw	560	70	0.91	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	510	60	0.87	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	440	60	0.38	TRUE			0.00	0.05			FALSE	FALSE
	LS	480	60	0.88	TRUE				0.00				FALSE
	UM	450	60	1.00	TRUE								
channel, mean curvature	CCw	-560	90	0.79	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	-500	70	0.59	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	-400	40	0.71	TRUE			0.16	0.01			TRUE	FALSE
	LS	-410	50	0.69	TRUE				0.18				TRUE
	UM	-420	50	0.67	TRUE								
plane, mean gradient	CCw	23.3	1.6	0.86	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	24.4	1.8	0.99	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	17.1	1.2	0.37	TRUE			0.00	0.00			FALSE	FALSE
	LS	18.1	1.7	0.95	TRUE				0.01				FALSE
	UM	18.7	1.4	0.97	TRUE								

(a)

Feature, statistic	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P-value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
ridge, mean curvature	CCw	620	80	0.66	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	560	60	0.93	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	470	60	0.73	TRUE			0.00	0.05			FALSE	FALSE
	LS	530	60	0.99	TRUE				0.00				FALSE
	UM	490	60	0.84	TRUE								
channel, mean curvature	CCw	-610	80	0.67	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	-550	60	0.99	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	-450	50	0.46	TRUE			0.14	0.00			TRUE	FALSE
	LS	-460	50	0.27	TRUE				0.05				FALSE
	UM	-470	50	0.87	TRUE								
plane, mean gradient	CCw	22.9	1.5	0.26	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	24.4	1.8	0.55	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	17.2	1.4	0.42	TRUE			0.00	0.00			FALSE	FALSE
	LS	18.2	1.7	0.99	TRUE				0.00				FALSE
	UM	19.1	1.5	0.97	TRUE								

(b)

Table 13. Morphometric feature parameter sampling distribution parameters (gradient in *degrees*, curvature in $10^{-5} m^{-1}$), test for normality of sampling distribution, and test for equal estimates of the mean for threshold curvature values of (a) $0.0005 m^{-1}$ and (b) $0.0001 m^{-1}$.

Feature, statistic	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P-value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
ridge, mean curvature	CCw	740	70	0.87	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	640	70	0.79	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	570	70	0.18	TRUE			0.00	0.00			FALSE	FALSE
	LS	620	60	0.78	TRUE				0.03				FALSE
	UM	600	60	0.46	TRUE								
channel, mean curvature	CCw	-710	70	0.95	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	-650	60	0.37	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	-530	50	0.81	TRUE			0.00	0.00			FALSE	FALSE
	LS	-560	50	0.44	TRUE				0.05				FALSE
	UM	-570	60	0.65	TRUE								
plane, mean gradient	CCw	23.6	1.6	0.63	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	24.9	1.9	0.95	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	17.5	1.4	0.97	TRUE			0.00	0.00			FALSE	FALSE
	LS	18.7	1.5	0.97	TRUE				0.00				FALSE
	UM	19.7	1.3	0.86	TRUE								

(a)

Feature, statistic	Unit	Sampling Distribution		KS normal test		Welch's Test							
		Mean	Std dev	P-value	H ₀ : normal	P-value				H ₀ : equal means			
						CCe	FC	LS	UM	CCe	FC	LS	UM
ridge, mean curvature	CCw	1040	80	0.47	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	930	60	0.67	TRUE		0.00	0.10	0.00		FALSE	FALSE	FALSE
	FC	870	50	0.66	TRUE			0.00	0.00			FALSE	FALSE
	LS	920	60	0.79	TRUE				0.07				FALSE
	UM	900	60	0.15	TRUE								
channel, mean curvature	CCw	-1020	80	0.67	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	-950	60	0.99	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	-810	40	0.68	TRUE			0.00	0.00			FALSE	FALSE
	LS	-850	50	0.98	TRUE				0.15				TRUE
	UM	-860	50	0.19	TRUE								
plane, mean gradient	CCw	24.7	1.8	0.92	TRUE	0.00	0.00	0.00	0.00	FALSE	FALSE	FALSE	FALSE
	CCe	25.6	1.9	0.90	TRUE		0.00	0.00	0.00		FALSE	FALSE	FALSE
	FC	18.3	1.2	0.88	TRUE			0.00	0.00			FALSE	FALSE
	LS	18.9	1.6	0.97	TRUE				0.00				FALSE
	UM	19.9	1.5	0.88	TRUE								

(b)

Table 14. Morphometric feature parameter sampling distribution parameters (gradient in *degrees*, curvatures in $10^{-5}m^{-1}$), test for normality of sampling distribution, and test for equal estimates of the mean for threshold curvature values of (a) $0.002m^{-1}$ and (b) $0.005m^{-1}$.

Figure 65. *Top:* mean elevation vs mean gradient, and *Bottom:* mean elevation vs curvature standard deviation. Error bars at 2 standard errors for both plots.

elevation. Higher values and range of elevation would generally produce higher values for its derivatives.

The plots of mean gradient against curvature standard deviation, ridge and channel mean proportion and channel proportion (Fig. 66) all show generally increasing trends. The plots are consistent with visual assessments of maps of gradient, curvature and morphometric feature, wherein high gradient is associated with high extreme absolute curvature and high proportions of ridges and channels. The plots also highlight the clustering of LS, FC and UM, and the clustering of CCe and CCw, mainly as a function of gradient. Locally, an inverse relation of gradient with curvature can be observed for CCe and CCw. Plots of the mean gradient of planar surfaces against the mean curvatures of ridges and channels (Fig. 67) follow the same general trend of the preceding plots .

Figure 66. *Top:* mean gradient vs curvature standard deviation *Middle:* mean gradient vs proportion of ridges and channels *Bottom:* mean gradient vs proportion of channels. Error bars at 2 standard errors for all plots.

Figure 67. *Top:* mean plane gradient vs mean ridge curvature *Bottom:* mean plane gradient vs mean channel curvature. Error bars at 2 standard errors for both plots.

3.4.2 Topography, slope materials and geomorphic processes

From the plots of gradient against the curvature-related values (Figs. 66, 67), the topography of the litho-structural units maybe qualitatively grouped into the following: low gradient, low absolute curvature (FS, LS, UM); high gradient, moderate absolute curvature (CCe), and high gradient, high absolute curvature (CCw).

Differences in gradient may be explained by the strength of materials with weaker materials comprising low-gradient terrain, and stronger materials comprising high-gradient terrain (Augustinus, 1995; Burbank and Pinter, 1999; Densmore et al., 1997; Kühni and Pfiffner, 2001; Moon and Selby, 1983; Roering et al., 2005; Schmidt and Montgomery, 1995). While lithology can simply represent the material strength, different lithologies can also end up having similar gradient distributions due to differential fracturing resulting to similar rock mass strengths. (Clarke and Burbank, 2010). This may be the case for the low-gradient litho-structural units LS, FC and UM. Following the above arguments, the CCe and CCw units are likely composed of materials with higher rock mass strengths than those in LS, FS and UM.

Differences in curvature may also be explained by the strength of materials, similarly as with gradient. At constant length scales, channels with higher absolute curvatures imply steeper slopes bounding both sides of the channel. Thus the highly concave channels of the CCw and CCe, relative to those of LS, FC and UM, suggest stronger slope materials.

High curvature terrains of CCw and CCe suggest sediment transport dominated by channel processes such as fluvial or debris flow incision, as discussed in Roering et al. (2005); Schneider et al. (2008); Simpson and Schlunegger (2003). In contrast, the low curvature terrains of the other units reflect dominant sediment transport by deep-seated landslides. This difference in the relative influence of channelized and hillslope processes suggests a lower frequency of landslides in CCe and CCw than in FC, LS and UM. Landslide frequency reflects landslide susceptibility, hence FC, LS and UM may be

more susceptible to landslides than CCe and CCw. This relative susceptibility rating is consistent with the gradient-based rock mass strength characterization of the different units discussed earlier.

The apparent inverse relationship between gradient and absolute curvature for CCe and CCw (Fig. 67) is worth explaining. Discussion on the stream power law, an empirical formula representing the erosive power of streams as a function of the steepness of the channel and the size of the contributing area, point to a limiting maximum channel gradient wherein the stream power decreases, and thus fluvial incision ceases (Dietrich et al., 2003; Montgomery, 2001; Tucker and Bras, 1998). In the case of the study area, further increase of mean gradient in CCe, beyond that of CCw, does not result to further channel incision. The transport regime now becomes hillslope- or landslide-dominated. More frequent or massive landslides could suppress further channelization by what is called as slope-clearing events (Densmore et al., 1997) which deliver excess amounts of debris to the channel or even erode the existing surface features downslope of the landslide. The topography of CCe, as discussed in the previous section on Geology, is characteristic of terrains which experience massive landslides. The hummocky topography at the base of the Can-abag ridge in the east likely represents the debris from such massive failures. Significant lowering of gradient, as anticipated for landslide-dominated terrain, may not happen if sliding surfaces are steeply-dipping, which is likely the case for CCe, as seen in the Guinsaugon landslide.

Alternatively, the lower gradient of CCw, compared to CCe, maybe due to gradient-lowering landslide processes facilitated by the SW-dipping beds of the CCw. However, the high-gradient, high-curvature topography of CCW does not strongly support the dominance of landslide processes in this area. Hence, the earlier explanation is adopted.

3.5 Conclusion

The five litho-structural units, CCw, CCe, FC, LS, and UM, in the study area can be characterized and distinguished by various morphometric parameters such as elevation, gradient, and curvature; by various expressions of morphometric feature classes such as the proportions of ridges and channels; and by the combination of morphometric parameters and feature classes.

LS, FC and UM share similar distributions of morphometric parameters and features. The areas underlain by these units generally have low elevation, gradient and absolute curvatures, and fewer ridges and channels. Areas underlain by CCe and CCw are generally similar, having high elevation, gradient and curvature, and more ridges and channels.

From these results, material properties and geomorphic processes are inferred. LS, FC and UM may generally have weaker materials than CCe and CCw based on gradient and on cross sectional curvature of channels. Landslides may constitute the dominant sediment transport in LS, FC and UM. Channel processes such as fluvial and debris flow incision are likely dominant in CCw. CCe may be somewhere in between, having significant channel processes, but also massive landslides at higher gradients. . These topography-based inferences will be investigated further, and validated (or rejected) in the next chapter where rock mass strength and slope stability conditions of the different litho-structural units are analyzed.

CHAPTER 4

ROCK MASS STRENGTH

4.1 Introduction

The previous chapters on Geology and Topography have shown five litho-structural units, CCw, CCE, FC, LS and UM, which all have distinct geologic and topographic characteristics. Results of topographic analyses vis-a-vis the spatial distribution of the litho-structural units reflect material control on the topography of the study area. This material control, particularly rock mass strength and slope stability, influences the dominant geomorphic processes in the area, which in turn shape and influence the landscape. The objectives of this chapter are 1) to characterize the materials comprising the slopes of the five litho-structural units in terms of rock mass strength, and 2) characterize the slope stability conditions of the litho-structural units. Since the strength of the rock masses are the focus of this work, the slope stability analysis is constrained to the deep-seated failure of the rock mass and does not include the shallow slope failures within the overlying soil or along the soil-bedrock interface. Results of the strength and stability analyses are cross-checked with a landslide inventory of the area.

4.2 Review of Literature

4.2.1 Modes of failure in rock masses

A discussion of rock mass strength typically starts off with how rock masses fail. Particularly for rock masses comprising slopes, there are, what is called, modes of failure. The simplest classification considers for modes of failure: planar, wedge, toppling and circular (Wyllie and Mah, 2004) (Fig. 68). The first two involve the sliding of a block along one and two planar structures, respectively, in a direction roughly the same as the dip direction of the slope. Toppling failure, while requiring a steep plane dipping into the slope, involves the rotation of a block or column relative to its base. These first

three modes of failure are often described as discontinuity-controlled, since the failure is strongly influenced by the geometry of the discontinuities vis-a-vis the geometry of the slope. The last one, circular failure, involves failure of rock slopes underlain by very weak or closely-fractured materials. Here, the materials behave isotropically, and there are no preferred directions of failure. Being isotropic, the shape of the failure surface is roughly circular, hence the term for this mode. Different failure modes require different types of analysis; any analysis must explicitly state the type of failure being analyzed (Wyllie and Mah, 2004).

4.2.2 Rock mass strength

Parallel to the discussion on modes of failure, a rock mass can either be characterized as isotropic or anisotropic. The former has enough “randomly” oriented joint sets such that the resulting failure is not controlled by one or two dominant discontinuity set/s (Marinos et al., 2005), and its failure roughly approximates that of a circular surface (Wyllie and Mah, 2004). The latter has one or two dominant discontinuity sets, and its failure corresponds to a planar, wedge or toppling failure (Wyllie and Mah, 2004).

The characterization of a rock mass whether it is isotropic or not is also dependent on the scale of the analysis. From the smallest to largest scale of analysis, a rock mass may be represented as an intact rock, a rock mass with one or two joint sets, or a many-to heavily-jointed rock mass (Hoek and Brown, 1997) (Fig. 69). As a general rule, when the block sizes are of the same order as the dimensions of the structure (slope bench, tunnel) being analyzed, the rock mass is considered anisotropic (Hoek and Brown, 1997). For the scale involved in the analysis of slopes, a rock mass can be represented as either isotropic or anisotropic rock masses, but rarely as an intact rock.

Consequently, the strengths of these two types of rock masses are assessed differently. The strength of an isotropic rock mass is defined by the strength of both the intact rock and the discontinuities which comprise the rock mass. For anisotropic rock masses, the strength is primarily determined by the strength of the discontinuity/ies.

Figure 68. Modes of failure in rock slopes. (a) planar, (b) wedge, (c) toppling, (d) circular. Image from Wyllie and Mah (2004).

Figure 69. Scale dependence of rock mass characterization. Image from Hoek and Brown (1997).

The most commonly-used expressions of strength for isotropic rock masses and for discontinuities are the Hoek-Brown failure criterion (Eberhardt, 2012; Hoek, 1983; Hoek et al., 2002) and the Barton-Bandis failure criterion (Barton and Bandis, 1991; Barton and Choubey, 1977), respectively.

4.2.2.1 Hoek-Brown failure criterion

The Hoek-Brown failure criterion (Eberhardt, 2012; Hoek, 1983; Hoek et al., 2002) is a strength criterion, specifically designed for jointed rock masses. A strength criterion is a mathematical relation, (expressed graphically as an envelope or curve in a 2-D plot), defining critical values of two stresses (representing the two axes of the plot) which result to failure of the rock mass. The use of a strength criterion for jointed rock masses is deemed appropriate when dealing with problems at scales equivalent to an individual slope or greater since joints will most likely be encountered within the rock mass being studied. The Hoek-Brown failure criterion takes into account the properties of the intact rock (unconfined compressive strength, material constant) and joints (Geological Strength Index) within the rock mass, and the damage to the rock mass resulting from disturbance by blasting or overburden removal (disturbance factor).

Figure 70. Proceq conversion curve from R to *UCS*.

4.2.2.1.1 Unconfined compressive strength

The unconfined compressive strength (*UCS*) is a measure of the intact rock strength. It is commonly determined in the laboratory by subjecting a rock sample to uniaxial compression until failure occurs. It has also been empirically correlated to other values that are measurable in the field using simple manual tests (Brown, 1981).

UCS can also be measured in the field using a rebound hammer, such as the Proceq Original Schmidt L-type Model (Proceq SA, 2006). This instrument produces a controlled impact on a test surface, and measures the distance of the rebound. It has been widely used in determining the uniaxial compressive strengths of rocks (Goudie, 2006).

The general operating procedures are discussed in Proceq SA (2006). For each representative test surface in the field, Barton and Choubey (1977) suggested obtaining the R-value by getting ten (10) individual rebound measurements; discarding the lowest five (5) values; and getting the mean of the remaining values. The R value and its conversion to *UCS* is sensitive to impact direction. Proceq SA (2006) provides conversion curves for downward-, upward-, and horizontally-directed impacts (Fig. 70). An analytical approach for the conversion, using the conservation of kinetic, gravitational potential, and spring potential energies, was presented by Basu and Aydin (2004), and equations (Eqs. 4.1-4.4) were formulated to facilitate the conversion :

$$R_{\theta} = \frac{x_{2(\theta)}}{x_1} \times 100 \quad (4.1)$$

$$0.5MV_{2(\theta)}^2 = 0.5kx_{2(\theta)}^2 + Mgx_{2(\theta)} \cos(90 - \theta) \quad (4.2)$$

$$V_2^2 = \frac{V_{2(\theta)}^2 (0.5kx_1^2)}{0.5kx_1^2 + Mgx_1 \cos(90 - \theta)} \quad (4.3)$$

$$R = \sqrt{\frac{V_2^2}{V_1^2}} \times 100 \quad (4.4)$$

where R is the measured rebound value defined by the ratio of the length of the spring at maximum rebound, x_2 , to the maximum extension of the spring prior to firing, x_1 . V_1, V_2 refer to initial velocities after firing and after rebound, respectively. θ is the inclination relative to the horizontal. Quantities where the impact direction is not horizontal are represented by the respective quantity with subscript θ . M, V_1, x_1 are constants for the equipment, and g is the acceleration due to gravity.

There is usually an expected limit of detection for every measuring instrument. The actual instrument reads R -values from 10 to 60, and has an accompanying conversion curve, from R to UCS , for horizontal R -values of 20 to 50 (Proceq SA, 2006). Data beyond limits of detection are often referred to as “non-detects” or “censored data” and cause some loss of information (Millard, 2013). However, statistical analysis of data sets containing non-detects may still be carried out. Millard (2013) discusses graphical methods for handling such data. One of these involves plotting the ordered data set against its empirical cumulative distribution function, $ecdf$:

$$ecdf = \left\{ \frac{1}{N}, \frac{2}{N}, \dots, \frac{N}{N} \right\} \quad (4.5)$$

where N is the number of observations. For a single detection limit or censoring level, T , all the censored values are considered equal, but they are all less than the smallest observed value. A pre-chosen or hypothesized cumulative distribution curve is fitted

to the observed values, disregarding the censored values. If the fit is satisfactory, the properties of the fitted curve is adopted for the data set. With these procedures, it is possible to estimate distribution parameters and compare data sets containing censored values.

Further reduction of the *UCS*, owing to the effect of saturation, is a known fact, and has been widely demonstrated. Reviewing earlier published works, Vásárhelyi et al. (2007) proposed tentative guidelines and factors for reducing the *UCS* of dry samples to their wet equivalents. The reduction factors (relative to dry samples) are dependent on the petrographic characteristics of the rock sample and range from 0.8-0.9 for well indurated strong rocks (marble and andesite); 0.6-0.7 for cemented, medium strength rocks (tuff, sandstone, limestone); to 0.3 for soft argillaceous rocks (shale).

4.2.2.1.2 Material constant

The material constant m_i , is a dimensionless quantity that controls the steepness of the Hoek-Brown failure envelope (Eberhardt, 2012; Hoek, 1983), and is therefore related to the friction angle of the intact rock. In the absence of triaxial tests, it is generally assumed to be dependent on the rock type. An updated table of m_i for various rock types is provided with the program RocLab (RocScience, 2007). While the m_i values are expressed as $a \pm b$, it is not explicitly stated whether b represents an absolute extremum, or a multiple of the standard deviation assuming a normal distribution, or some other indicator of range.

4.2.2.1.3 Geological Strength Index

The Geological Strength Index (*GSI*) (Marinos and Hoek, 2000) is a rock mass characterization system used to assess the discontinuities within a rock mass. It expresses rock mass discontinuity strength as a function of the geometry and surface conditions of the discontinuities (Marinos et al., 2005). It is also an expression of the freedom of intact rock blocks to slide or rotate within the rock mass (Hoek and Brown, 1997). In the *GSI* chart (Fig. 71), the geometry and surface conditions, along with their descriptions are

arranged in rows and columns, respectively. A rock mass is described based on these two conditions, and the intersection of these conditions in the chart provide a range of *GSI* values. Marinos and Hoek (2000) discuss petrographic characteristics as constraints to possible *GSI* values for particular rock types, and thus provides additional guidelines for designating *GSI*.

GSI is typically used to classify jointed rock masses, which have enough “randomly” oriented joint sets such that the resulting failure is isotropic (Marinos et al., 2005). While the use of this for rock masses with one or two dominant discontinuity sets is discouraged, one may still do so if the failure is assessed to be isotropic (meaning, not controlled by dominant structures), or when the structures significantly reduce the strength difference between rock mass and discontinuity strength (such as in the case of laminated/sheared rock masses, last row of the *GSI* chart (Fig. 71) (Marinos et al., 2005).

4.2.2.1.4 Disturbance Factor

Disturbance Factor (*D*) was introduced in the Generalized Hoek-Brown failure criterion (Hoek et al., 2002) to account for the disturbance of rock masses due to blast damage or stress relief due to removal of overburden. It expresses the disruption of the interlocking of the individual rock pieces as a result of opening of the discontinuities (Marinos et al., 2005). While the authors provided some guidelines in estimating the value of *D* (from 0.0 to 1.0), they recognized the difficulty in precisely quantifying the many factors which influence the parameter. Furthermore, they recommended adjusting its value depending on the actual performance of the slope or tunnel.

4.2.2.1.5 Shear strength envelope

The Hoek-Brown failure criterion defines a failure envelope in terms of the principal stresses, σ'_1 and σ'_3 , based on the following equations from (Hoek et al., 2002):

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a \quad (4.6)$$

$$m_b = m_i \exp \left(\frac{GSI - 100}{28 - 14D} \right) \quad (4.7)$$

$$s = \exp \left(\frac{GSI - 100}{9 - 3D} \right) \quad (4.8)$$

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{-GSI/15} - e^{20/3} \right) \quad (4.9)$$

Because of the convenience in dealing with slope stability problems in terms of normal (σ'_n) and shear (τ') stresses, σ'_3 and σ'_1 are usually transformed into the corresponding σ'_n and τ' values using the following equations, also from (Hoek et al., 2002):

$$\sigma'_n = \frac{\sigma'_1 + \sigma'_3}{2} - \left[\frac{\sigma'_1 - \sigma'_3}{2} \left(\frac{d\sigma'_1/d\sigma'_3 - 1}{d\sigma'_1/d\sigma'_3 + 1} \right) \right] \quad (4.10)$$

$$\tau' = (\sigma'_1 - \sigma'_3) \frac{\sqrt{d\sigma'_1/d\sigma'_3}}{d\sigma'_1/d\sigma'_3 + 1} \quad (4.11)$$

$$d\sigma'_1/d\sigma'_3 = 1 + am_b \left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^{a-1} \quad (4.12)$$

The resulting plot of σ'_n vs τ' is the equivalent Mohr-Coulomb failure envelope, from which the Mohr-Coulomb strength parameters, friction angle (ϕ') and cohesion (c'), can be computed. In the typical linear Mohr-Coulomb failure envelope, ϕ' and c' are constant, corresponding to the slope and y-intercept of the failure envelope, respectively. However, the Hoek-Brown failure criterion produces a non-linear envelope, and ϕ' and c' are now expressed as the slope and y-intercept of the tangent line to the failure envelope. As a result, ϕ' and c' will vary with σ'_n . A sample plot from RocLab (RocScience, 2007) illustrates the Hoek-Brown failure criterion and its transformation to the Mohr-Coulomb failure criterion (Fig. 72).

In the application of the Hoek-Brown model to slopes, the maximum range of the minor principal stress, σ'_{3max} , for wide ranges of slope geometry and material properties, has been expressed as a function of slope height, H (Hoek et al., 2002):

Figure 72. Hoek-Brown failure envelope plotted in $\sigma'_3 - \sigma'_1$ (Left) and $\sigma'_n - \tau'$ (Right) space. Red curves represent the actual non-linear envelope. Purple lines tangent to the envelopes represent instantaneous ϕ' and c' at a particular σ'_n .

$$\sigma'_{3max} = 0.72\sigma'_{cm} \left(\frac{\sigma'_{cm}}{\gamma H} \right)^{-0.91} \quad (4.13)$$

where γ is the unit weight of the slope material and σ'_{cm} is a “global” or averaged rock mass strength determined from the Hoek-Brown parameters (Hoek et al., 2002):

$$\sigma'_{cm} = \sigma'_{ci} \frac{(m_b + 4s - a(m_b - 8s)) (m_b/4 + s)^{a-1}}{2(1+a)(2+a)} \quad (4.14)$$

The average values of σ'_n and τ' for a particular strength envelope are influenced by the range of values of σ'_3 (Eqs. 4.10, 4.11), which in turn is determined by σ'_{3max} , and ultimately by H (Eq. 4.13). Thus, for a particular set of material properties, larger slopes are characterized by larger normal and shear stresses at failure conditions.

For a constant slope height, different sets of material properties would give different values for average normal and shear stresses at failures. These varying shear stresses at failure are indicative of the variability of shear strength across the different sets of material properties.

4.2.2.2 Barton-Bandis failure criterion

The Barton-Bandis joint model (Eq. 4.15) (Barton and Bandis, 1991; Barton and Choubey, 1977) is used to characterize the strength of rock joints. It expresses the shear stress at failure (τ_f) as a function of applied normal stress (σ_n), roughness of the joint (Joint Roughness Coefficient, JRC), strength of the joint surface (Joint Compressive Strength, JCS), and residual friction angle of the rock material (ϕ_r):

$$\tau_f = \sigma_n \tan \left[JRC \log \left(\frac{JCS}{\sigma_n} \right) + \phi_r \right] \quad (4.15)$$

$$\phi_r = (\phi_b - 20) + 20 \left(\frac{r}{R} \right) \quad (4.16)$$

Figure 73. a.) Chart for estimating *JRC*. (b) Stress-displacement tests illustrating the effect of joint length on *JRC* and *JCS*. Both images from Barton and Bandis (1991).

JRC is a unitless measure representing the roughness of a joint surface at a particular joint length. A chart for estimating *JRC* is provided by Barton and Bandis (1991) (Fig. 73a).

JCS is the unconfined compressive strength (*UCS*) of the thin layer of rock material adjacent to the joint walls, and may be equal to or less than the *UCS* of the intact rock depending on the degree of weathering in the joint walls (Barton and Choubey, 1977). In the absence of direct measures of *JCS*, a conservative lower bound value equal to $1/4$ of the *UCS* of dry intact rock is proposed (Barton and Choubey, 1977).

ϕ_r and has been empirically related to the friction angle of the dry unweathered sawn surface of rock determined from tilt or push-pull tests (ϕ_b) (Eq. 4.16), where r and R are the rebound hammer values on wet joint surfaces, and on dry unweathered sawn surface of rock, respectively (Barton and Choubey, 1977). ϕ_r is therefore a reduced value of ϕ_b , with $\frac{r}{R} \leq 1$.

JRC and JCS have been shown to be scale-dependent (Barton and Choubey, 1977), decreasing as the length of the joint increases (Fig. 73b). Standard measurements of JRC and JCS (JCR_0 and JCS_0 , respectively), performed on laboratory-sized samples, can be adjusted to the actual dimensions of the joints (JCR_n and JCS_n , respectively) with the following equations Barton and Bandis (1991):

$$JRC_n \approx JRC_0 \left[\frac{L_n}{L_0} \right]^{-0.02 JRC_0} \quad (4.17)$$

$$JCS_n \approx JCS_0 \left[\frac{L_n}{L_0} \right]^{-0.03 JRC_0} \quad (4.18)$$

where the subscripts (0) and (n) refer to the laboratory-scale (100 mm) and in-situ joint lengths. These scaled values are the ones used in Equation 4.15. The scale effect is somehow limited by the natural size of blocks in the rock mass (Barton and Choubey, 1977). The block size may be used to provide upper limits to L_n , which controls the scale effect.

The Barton-Bandis model, similar to the Hoek-Brown failure criterion, is non-linear. It can be used to compute the shear strength, s , of a discontinuity in terms of the linear Mohr-Coulomb failure criterion (Duncan and Wright, 2005; Wyllie and Mah, 2004):

$$s = c + \sigma_n \tan \phi \quad (4.19)$$

where c is cohesion, and ϕ is the friction angle. For discontinuities, $c = 0$ is assumed and that the strength of the discontinuity is practically, entirely frictional (Barton, 2013). ϕ is equivalent to the quantity inside the tangent expression in Equation 4.15. At failure:

$$s = \tau_f = \sigma_n \tan \phi \quad (4.20)$$

Thus:

$$\tau_f = \sigma_n \tan \left[JRC \log \left(\frac{JCS}{\sigma_n} \right) + \phi_r \right] = \sigma_n \tan \phi \quad (4.21)$$

$$\phi = \arctan \frac{\tau_f}{\sigma_n} \quad (4.22)$$

4.2.3 Slope stability analysis

Slope stability is commonly expressed as a factor of safety (FS), which is the ratio of the resisting and driving forces acting on a slope mass (Wyllie and Mah, 2004):

$$FS = \frac{\text{resisting forces}}{\text{driving forces}} \quad (4.23)$$

Resisting forces depend primarily on strength characteristics of the slope material. Driving forces usually consist of the weight of the potentially unstable mass, the force due to pore water pressure, and seismic loads. Unstable conditions arise when the resisting force is less than the driving force, which corresponds to $FS < 1$. As mentioned in the discussion of modes of failure, the slope stability analysis will depend on what type of failure is expected from a rock mass.

4.2.3.1 Stability analysis for circular failures

The factor of safety for circular failures can be assessed using the limit equilibrium method of slices (Fredlund et al., 1981; Krahn, 2003; Zhu et al., 2003). The procedure generally follows the following steps (Duncan and Wright, 2005; Zhu et al., 2003):

1. The potentially unstable mass is defined by a slip surface (blue circular arc in Figure 74).
2. The mass is subdivided into a finite number of vertical slices (10 slices in Figure 74).
3. Driving and resisting forces are computed for each slice.
4. Factor of safety is determined from the ratio of sum of resisting forces to the sum of driving forces.
5. The above steps are repeated for various slip surfaces to locate the surface with the least factor of safety, referred to as the critical slip surface.

The basic equation embodying the above steps is:

Figure 74. Slope stability analysis using method of slices.

$$FS = \frac{\sum s_i l_i}{\sum W_i \sin \alpha_i} \quad (4.24)$$

where s_i , l_i , W , and α_i represent the shear strength at the base, length of the base, weight, and inclination of the base of the i^{th} slice, respectively. The quantity $s_i l_i$ is the resisting force, R_i , of the slice; and $W_i \sin \alpha_i$ is the driving force, D_i , due to the component of weight parallel to the slice base.

There are many forms of the procedure (Fredlund and Barbour, 1986; Krahn, 2003; Zhu et al., 2003) and one of these is the Simplified Bishop Method (Bishop, 1955; Duncan and Wright, 2005; Fredlund and Barbour, 1986; Krahn, 2003; Wyllie and Mah, 2004; Zhu et al., 2003). This method, given its simplicity, returns values that are comparable to other more rigorous limit equilibrium procedures, and even to those computed by finite element procedures (Duncan and Wright, 2005). The basic equation (Duncan and Wright, 2005) is given by:

$$FS = \frac{\sum \left[\frac{c' l \cos \alpha + W \tan \phi'}{\cos \alpha + (\sin \alpha \tan \phi') / FS} \right]}{\sum W \sin \alpha} \quad (4.25)$$

where c' and ϕ' are the Mohr-Coulomb strength parameters cohesion and friction angle. Note that the equation has FS on both sides. This means that this method requires

iterative solving, and an initial estimate of FS to start with. The initial FS is usually solved through the Ordinary Method of Slices (Wyllie and Mah, 2004).

External forces such as those caused by pore water pressure and seismic loads can also be incorporated in the analysis (Duncan and Wright, 2005):

$$FS = \frac{\sum \left[\frac{c' l \cos \alpha + (W - ul \cos \alpha) \tan \phi'}{\cos \alpha + (\sin \alpha \tan \phi')/FS} \right]}{\sum W \sin \alpha + M_{kh}/r} \quad (4.26)$$

where u is the pore water pressure, M_{kh} is the moment due to the seismic force, and r is the radius of the circular arc. The pore water pressure can be determined if the water table or piezometric surface are known (Krahn, 2003; RocScience, ated). In the absence of such measurements, estimates of pore water pressure may be obtained using some coefficients such as r_u (Bishop and Morgenstern, 1960):

$$u = \frac{r_u}{\gamma H} \quad (4.27)$$

where u is the pore water pressure at a point along the slip surface, and γH is the vertical overburden pressure on that point due to the overlying material with unit weight γ and thickness H . Typical values of r_u encountered in practice range from 0.0 to 0.7 (Bishop and Morgenstern, 1960), and 0.3-0.45 (Fredlund and Barbour, 1986). Values of 0.35 and above are found to cause stability problems (Fredlund and Barbour, 1986). A value of 0.5 is cited as equivalent to a fully-filled tension crack (Heinrendt et al., 2003).

Seismic loads can be analyzed as pseudostatic forces acting to destabilize the slope mass, and are represented as vertical and horizontal forces, F_v and F_h , proportional to the weight, W , of the individual slice (Duncan and Wright, 2005; Krahn, 2003; RocScience, ated):

$$F_v = \frac{k_v W}{g}, \quad F_h = \frac{k_h W}{g} \quad (4.28)$$

where g is the acceleration due to gravity; and k_v, k_h are the vertical and horizontal seismic coefficients. The effect of F_v on the stability of a slope is often ambiguous, with effects on shear strength usually being offset by effects on shear stress, and may thus be excluded in the analysis (Krahn, 2003; RocScience, ated; Wyllie and Mah, 2004). Typical values for k_h are 0.1 to 0.3 (RocScience, ated). However, the choice of values appears arbitrary and subjective as seen in varying seismic coefficients used for design purposes (Melo and Sharma, 2004).

Specific steps for calculating FS using the Simplified Bishop method is outlined in (Duncan and Wright, 2005), and the particular handling of materials governed by a non-linear strength criterion is discussed in (Wyllie and Mah, 2004).

The search for the critical circular slip surface may be implemented by a grid-search or slope-search method, both of which are discussed in RocScience (ated). In the former, a grid containing the centers of potential circular slip surfaces is defined. A circular arc, centered on the grid, is drawn at a given radius such that the arc intersect the slope surface. In the latter, two points within the slope surface are assigned as the endpoints of the circular arc defining a slip surface. An angle from the horizontal, measured at the lower point, is used to generate the arc. For both methods, a large number of slip surfaces must be defined to ensure that the factor of safety is closest to the minimum value as possible. Between the two methods, the latter provides more control on where the arc intersects the slope surface.

While the Simplified Bishop Method had seen much use mainly for soil slopes, it has been adopted for use in the analysis of jointed rock masses which behave as isotropic masses (Li et al., 2008,1; Wyllie and Mah, 2004).

4.2.3.2 Stability analysis for discontinuity-controlled failures

The analysis of discontinuity-controlled failures consist of two steps. The first involves the determination of whether a potential sliding block exists by virtue of the geometric relationships between the slope face and the discontinuity planes. The second, assesses

the shear strengths and stresses along the discontinuity and solves for the factor of safety of the slope.

The first step, often referred to as kinematic analysis (Admassu and Shakoor, 2013), is a geometric assessment of the relevant planar surfaces generally carried out on a stereonet. Admassu and Shakoor (2013), summarizes the tests for the various failure types from the works of Goodman (1989); Hoek and Bray (1981); Markland (1972); Wyllie and Mah (2004).:

...plane failure may occur when a discontinuity dips in the same direction (within 20 degrees) as the slope face, at an angle gentler than the slope angle but greater than the friction angle along the failure plane...

...wedge failure may occur when the line of intersection of two discontinuities, forming a wedge-shaped block, trends in the same direction as the slope face and its plunge angle is less than the slope angle but greater than the friction angle along the discontinuities...

...toppling failure may result when discontinuities parallel to the slope face (within 30 degrees) are steeply dipping into the slope face...

The second step, which is the stability analysis for computing factor of safety, are discussed in detail by Wyllie and Mah (2004).

4.2.3.3 Probabilistic analysis

Probabilistic analysis provides a way of incorporating uncertainties into slope stability problems and is considered beneficial in decision-making in the engineering practice (El-Ramly et al., 2002). It accounts for the inherent spatial variability of properties of geologic materials, and the statistical uncertainty due to limited data. This type of analysis has also been extended to regional landslide hazard zonations through integration with GIS techniques(Zhou et al., 2003). One of the most common methods employed is

the Monte Carlo Simulation (El-Ramly et al., 2002; Metropolis and Ulam, 1949; Sobol, 1994), which produces a distribution of an output variable, U , expressed as:

$$U_i = f(R1_i, R2_i, R3_i, R4_i, \dots), i = 1, 2, 3, \dots N \quad (4.29)$$

where U is a function of random input variables $R1, R2, \dots$, and N is the number of times that the value of U was computed from a corresponding random set of input variables.

In a probabilistic slope stability analysis, the first step is to characterize relevant geotechnical data to determine the distribution of the input variables; and the second part is to conduct the stability analysis using the random variables from the input distributions (Park et al., 2005). The former corresponds to the characterization of $R1, R2, \dots$, whereas the latter corresponds to the definition of the function f , and the process of solving for U (Eq. 4.29).

The typical output for a probabilistic slope stability analysis, aside from the mean factor of safety, is a distribution of factors of safety and a probability of failure (El-Ramly et al., 2002; Tobutt, 1982). The latter is defined as:

$$P_f = \frac{N - M}{N} \quad (4.30)$$

where N is the number of iterations in the Monte Carlo simulation, also equal to the total number of factor of safety values generated; and M is the number of results where $FS > 1$ (Wyllie and Mah, 2004). Probabilistic analysis has been demonstrated for materials governed by the Hoek-Brown failure criterion (Hoek, 1998; Li et al., 2012).

4.2.4 Landslide inventory

A landslide inventory is considered as the most important data set for evaluating landslide susceptibility, hazard and risk (van Westen et al., 2008). It describes landslide activity based on a data series of landslide events occurring over an area of interest (Lee, 2009). Remotely sensed data are valuable sources for constructing such inventories. Aerial pho-

tographs and optical satellite images allow identification of landslides based on disrupted or absent vegetation which are distinct from surrounding terrain (van Westen et al., 2008). High resolution imageries such as Quickbird, IKONOS, PRISM, CARTOSAT-1, and those generated by laser scanning provide accurate elevation data for topography-based recognition of landslides (van Westen et al., 2008).

Relative frequency analysis characterizes the effects of landslide controls such as topography and geology (Lee, 2009). By dividing the number of landslides in a subpopulation based on slope class or bedrock unit by the corresponding area, comparison among the degree of influence of these factors can be made. Frequency statistics are, however, sensitive to sample populations; and small sample sizes could provide inaccurate assessments (Lee, 2009).

The use of landslide inventories for mapping or validating susceptibility, hazard and risk must always contend with the major issue of the incompleteness of historical landslide records arising from lack of data for a specific region, or period, especially right after a major landslide-triggering event (van Westen et al., 2008). The lack of suitable time series data for most regions of the world (Lee, 2009) cannot guarantee a reliable estimate of the probability of landsliding (Lee and Jones, 2004). Hence, most estimates of landslide probability will be based not entirely on “objective” facts, but on judgement based on the available data, knowledge and experience (Lee, 2009).

4.3 Methods

4.3.1 Rock mass strength

4.3.1.1 Hoek-Brown failure criterion

For isotropic slope materials, properties relevant to the Hoek-Brown failure criterion were measured in the field, or adopted from literature; and expressed as constant values, or as distributions. These were used to construct strength envelopes and to determine the distribution of shear strength for each of the litho-structural units. The analysis under the Hoek-Brown failure criterion assumes an isotropic failure. Thus, the distinction between

CCe and CCw because of the identified anisotropy due to bedding dip direction was not considered for this case and both units were treated as one unit, CC.

4.3.1.1.1 Unconfined compressive strength

The Proceq Original Schmidt L-type Model, with an impact energy of 0.735 Nm (Proceq SA, 2006), was used in this study to measure *UCS*. The field protocol of measuring ten *R*-values per intact rock surface, and discarding the five lowest values, as suggested by Barton and Choubey (1977), was adopted. However, calibration instructions constrained the total allowable number of impacts for the instrument during the fieldwork, which resulted to some surfaces being tested with just five (5) measurements or less. In such cases, no measurements were discarded, and the *R*-value was derived from the mean of the readings. Field data consists of rebound measurements from 58, 16, 13, and 24 surfaces in CC, FC, LS and UM, respectively.

R-values were not always measured in a horizontal position. Hence, the angle of impact direction, θ , was noted, and the resulting measurement, *R*, was later converted to the equivalent horizontal value, R_h , as prescribed by equations in Basu and Aydin (2004), and implemented in a customized Python function using the Numpy library (Walt et al., 2011).

For surfaces with individual R_h readings below the limits of detection of the instrument ($R_h < 10$), an arbitrary value of 1 was assigned. Computation of the mean was done using procedures for “censored data” as discussed in Millard (2013). The empirical cumulative distribution curve was determined from the raw data, and a normal cumulative distribution curve was fitted to it using LMFIT-py (Newville et al., 2014) (Fig. 75). The fitted normal distribution, verified by the KS goodness-of-fit test (Kolmogorov, 1992; Smirnov, 1948,3), provided the data with the corresponding distribution parameters (Fig. 75). For surfaces where all values are below detectable limits, an arbitrary value of 1 was assigned for the mean.

Figure 75. Example of normal curve fitted to R_h data set, disregarding values below detectable limits ($dl = 10$). The mean of the data set is the mean of the fitted normal distribution.

The mean R -values were converted to UCS using the conversion curves provided with the rebound hammer (Proceq SA, 2006) (Fig. 70). Because of the limited values in the curves, UCS values for $R_h > 50$ were derived from the extrapolated curve, whereas those for $R_h < 20$ were considered censored data and were each assigned an arbitrary value of 1.

Field measurements were done during fair weather and the samples tested were mostly dry. Reduction factors based on Vászrhelyi et al. (2007), were used to obtain the saturated UCS values. For CC and LS, the lower (0.6) and upper (0.7) values, respectively, of reduction factors for medium strength rocks were used. For FC, the reduction factor for soft argillaceous rocks (0.3) was adopted. The lower end for well indurated strong rocks (0.8) was used for UM.

Because of the limitations of the conversion curves, some UCS values for each of the litho-structural units would have the arbitrarily-assigned value of 1. To determine the distribution of UCS , the procedure for censored values, similar to the one conducted for R_h in the field, was performed (Millard, 2013; Newville et al., 2014). A normal curve was tested for goodness-of-fit with the data points (Kolmogorov, 1992; Smirnov, 1948,3), and with positive results, the distribution parameters were adopted for the data

set. Otherwise, alternative distributions, such as the log-normal or beta distributions were explored. Distributions, means and standard deviations were tested for differences using sampling distribution analysis (Cam et al., 1986; Feller, 1945), KS test (Kolmogorov, 1992; Smirnov, 1948,3), and Welch's Test (Welch, 1947).

4.3.1.1.2 Material constant

The material constant m_i , in the absence of triaxial tests, is assumed to be dependent on the rock type. Values used for this study were based on published literature, and their representation as statistical distributions were based on some assumptions.

The range of lithologies in the study area were used to determine the corresponding values of similar rock types presented in RocScience (2007). Each litho-structural group was then defined by the range of lithologies comprising it. The overall minimum and maximum values for each set of comprising lithologies, was adopted as the minimum and maximum for the litho-structural group.

The corresponding mean and standard deviations were computed, assuming that the resulting distributions for the litho-structural groups were normal, and that the respective minimum and maximum values represent two standard deviations from their respective means.

Hypothetical data were generated from the assumed normal distributions. Differences across litho-structural groups, in terms of distributions, means and standard deviations were tested for differences using sampling distribution analysis (Cam et al., 1986; Feller, 1945), KS test (Kolmogorov, 1992; Smirnov, 1948,3), and Welch's Test (Welch, 1947).

4.3.1.1.3 Geological Strength Index

In the field, the *GSI* of slope rock masses were characterized with the aid of the *GSI* chart (Fig. 71). The geometry of discontinuities were described and classified as *massive*, *blocky*, *very blocky*, *blocky/disturbed/seamy*, *disintegrated*, or *sheared*. Surface conditions of these discontinuities were described as *very good*, *good*, *fair*, *poor*, or *very poor*.

A central value and range of GSI , corresponding to the intersection of these descriptions in the GSI chart, are assigned to the slope rock mass. Field data consists of GSI estimates from 16, 12, 9, and 20 outcrops in CC, FC, LS and UM, respectively.

To determine the distribution of GSI for each litho-structural group, a normal curve was tested for goodness-of-fit with the data points (Kolmogorov, 1992; Smirnov, 1948,3), and with positive results, the distribution parameters were adopted for the data set. Otherwise, other distributions, such as the log-normal or beta distributions were explored.

Distributions, means and standard deviations were tested for differences using sampling distribution analysis (Cam et al., 1986; Feller, 1945), KS test (Kolmogorov, 1992; Smirnov, 1948,3), and Welch's Test (Welch, 1947).

4.3.1.1.4 Disturbance Factor

Assignment of D was based on guidelines in Hoek et al. (2002). For this study, disturbance due to stress relaxation of natural slope materials was assumed, and was assigned a constant value of $D = 0.7$. This is equivalent to modest rock damage caused by controlled blasting of civil engineering slopes .

4.3.1.1.5 Shear strength

Using the distributions and values for UCS , GSI , m_i , and D , the distribution of shear stress at failure (also equal to the shear strength), τ' , was determined by probabilistic analysis using Monte Carlo simulation (El-Ramly et al., 2002; Metropolis and Ulam, 1949; Sobol, 1994). Random values ($N = 10,000$) of UCS , GSI , and m_i were generated from their respective distributions.

Since shear strength is a function of the stresses induced within the slope, which are, in turn, influenced by the size of the slope (Eqs. 4.6, 4.10, 4.11, 4.13), corresponding random values of slope heights, H , were also generated, using a uniform distribution. Different sets of height ranges ($5 - 10m$, $10 - 20m$, $20 - 50m$, and $50 - 100m$) were used to investigate the stress-dependence of the shear strength.

The shear strength, and the corresponding normal stress, σ_n , are expressed as:

$$[\tau'_k, \sigma'_{nk}] = f(H_k, UCS_k, GSI_k, m_{ik}, D, \gamma) \text{ for } k = 1, 2, 3 \dots N \quad (4.31)$$

and solved using equations provided in Hoek et al. (2002). Each unique set of inputs produces a strength envelope comprised of $[\tau'_k, \sigma'_{nk}]$ pairs. The value of τ'_k at the middle of the envelope $\left(\frac{\sigma'_{nkmin} + \sigma'_{nkmax}}{2}\right)$ was used for analyzing the distribution of shear strength.

To determine the distribution of τ' for each litho-structural group, a normal curve was tested for goodness-of-fit with the data points (Kolmogorov, 1992; Smirnov, 1948,3), and with positive results, the distribution parameters were assumed for the data set. Otherwise, other distributions, such as the log-normal or beta distributions were explored.

Distributions, means and standard deviations were tested for differences using sampling distribution analysis (Cam et al., 1986; Feller, 1945), KS test (Kolmogorov, 1992; Smirnov, 1948,3), and Welch's Test (Welch, 1947).

The analyses for the Hoek-Brown failure criterion were implemented in a customized Python 2.7 code (Python Software Foundation, 2014) using the Scipy (Jones et al., 01), Numpy (Walt et al., 2011), Matplotlib (Hunter, 2007), and Pandas (McKinney, 2010) libraries.

4.3.1.2 Barton-Bandis failure criterion

Characterization of discontinuities in the field, based on the Barton-Bandis failure criterion, was not done due to logistical constraints. The determination of ϕ_b and ϕ_r requires push-pull and/or tilt tests, whereas *JRC* requires the detailed profiling of the joint surface. Equipment for performing these tests are specialized and were not readily available at the time of the study. For the purpose of the stability analysis, uniform values of Barton-Bandis material properties were assumed and used for all litho-structural units: $L_n = 0.1 - 0.5 \text{ m}$, $\phi_r = 10 - 25 \text{ degrees}$, $JRC_0 = 0 - 20$, and $JCS_0 = 5 - 60 \text{ MPa}$.

4.3.2 Slope Stability

4.3.2.1 Stability analysis for circular failures

4.3.2.1.1 Overview

A probabilistic slope stability analysis (Fig. 77) was conducted using 1) the set of Hoek-Brown parameters for each of the litho-structural groups in the study area, 2) constant values representing external forces due to pore water pressures and seismic loads, and 3) slope gradient values of 15, 30, 45, 60 and 75 degrees. At the core of the analysis is the deterministic analysis of slope stability (Fig. 78) using the Simplified Bishop Method (Bishop, 1955; Duncan and Wright, 2005; Wyllie and Mah, 2004). Results were plotted as separate curves of slope gradient vs probability of failure and slope gradient vs mean factor of safety (Li et al., 2012) for each of the litho-structural group. Differences in the curves were analyzed and discussed.

4.3.2.1.2 Material Properties

The characterization of material properties based on the Hoek-Brown failure criterion was discussed in Section 4.3.1.1 4.3.1.1.

4.3.2.1.3 External forces

Since material properties were the main concern of this study, constant values of external forces were used in the analysis. The pore water pressure condition was represented by $r_u = 0.3$, whereas the seismic load is determined by $k_h = 0.3$. Resulting factors of safety and probabilities of failure therefore reflected conditions wherein both earthquake and hydrologic triggers are present. The above conditions used are rare in the human time scale. However, at the geomorphologic time scale, where the persistence of landscapes shaped by bedrock landslides are in the range of $10^2 - 10^7$ years (Guthrie and Evans, 2007), these triggering conditions are fairly common and are thus expected to be reflected in the topography as we see it today.

4.3.2.1.4 Slope model

For the analysis, each slope model (Fig. 76a) was defined by its gradient, β , and a base length of 30m, from which the corresponding height, H , is computed. Thus, for $\beta = \{15, 30, 45, 60, 75\}$, there is a corresponding $H = \{8, 17, 30, 52, 112\}$. This was done to make the horizontal scale of the slope comparable to the slope gradients from the DEM, which were measured over a similar horizontal scale. The toe and crest of the slope merge with horizontal surfaces. The horizontal surface at the toe extends a distance equal to the baselength of the slope, whereas the one at the crest extends to half of the length of the slope face. Such extensions were used since numerous slope stability analyses yield results in which the critical slip surfaces usually exit either beyond the slope toe and/or crest.

For each slope model, $N = 1000$ circular slip surfaces were randomly generated by an algorithm similar to the slope search method (RocScience, ated) (Fig. 76b). The lower arc exit point was allowed only at the lower horizontal surface, at the toe, and at the lower half of the slope face, whereas the upper arc exit point was allowed only at the crest and at the upper horizontal surface. The lower exit angle was constrained within the closed interval $[-30, \beta - 10]$, where a negative angle is measured below the horizontal. The mass bound by each slip surface and the slope surface was divided into 10 vertical slices with equal widths. The height and inclination of the base of the slices were computed from the geometry of the slip and slope surfaces.

4.3.2.1.5 Probabilistic analysis

The Monte Carlo simulation consisted of $K = 1000$ iterations over the distributions of the material properties of each litho-structural group (Fig. 77). Each set of material properties, M_j , was analyzed deterministically (Fig. 78) against the N slip surfaces generated for a specific slope geometry, $[H, \beta]$, and the external force variables r_u and k_h . Each simulation produced 1000 minimum factors of safety, $\min FS_{1,2,\dots,K}$, corresponding

Figure 76. Slope model for stability analysis of circular failure. (a) Slope dimensions. (b) Circular failure arcs for $\beta = 60$, $N=1000$.

to the sets of material properties. From the distribution of $\min FS$, various statistics such as the mean, $\overline{\min FS}$, and the probability of failure, P_f (Eq. 4.30), were computed.

The process was repeated for all values of β and for all litho-structural groups. Data is presented as plots of $\overline{\min FS}$ vs β , and P_f vs β for each of the litho-structural groups.

4.3.2.1.6 Deterministic analysis

The deterministic analysis involved the analysis of one set of material properties and external forces against the N failure surfaces (Fig. 78). The process is, essentially, a Monte Carlo simulation wherein the effect of the slip surface geometry on the factor of safety is being investigated. This process produced 1000 factors of safety from which $\min FS$, the minimum factor of safety with respect to the particular set of material property, was determined.

The specific process for solving the factor of safety for a particular slip surface is shown in Figure 79. It involves the computation of parameters for each slice, sl_i :

$$H_i = y_s(x_i) - y_c(x_i) \quad (4.32)$$

$$\alpha_i = \frac{\Delta y}{\Delta x}(x_i) \quad (4.33)$$

$$l_i = \frac{w_i}{\cos \alpha_i} \quad (4.34)$$

where x_i , w_i are the horizontal position and width of sl_i , determined from the extent of the failure surface and the number of slices; y_s , y_c are the vertical positions of the slope and slip surfaces, respectively; H_i , α_i , l_i are the height of the slice, and the inclination and length of the slice base, respectively.

With the material unit weight, γ , pore water pressure coefficient, r_u , and horizontal seismic coefficient, k_h , the following slice parameters are computed:

$$W_i = \gamma_i H_i w_i \quad (4.35)$$

$$SF_i = W_i \sin \alpha_i \quad (4.36)$$

$$seisF_i = k_h W_i \quad (4.37)$$

$$u_i = \frac{r_u}{\gamma H_i} \quad (4.38)$$

where W_i , SF_i , $seisF_i$, u_i are the weight of the slice, shearing force at the slice base due to the weight, seismic force acting on the center of gravity of the slice, and pore water pressure at the slice base, respectively.

The strength envelope of the material was generated from the Hoek-Brown failure criterion (Eqs. 4.10 and 4.11) and the current set of material properties. A spline function, f , was used to express the shear stress, τ' , as a function of normal stress, σ'_n :

$$\tau' = f(\sigma'_n) \quad (4.39)$$

and the corresponding Mohr-Coulomb strength parameters, ϕ' , c' are:

$$\phi' = \arctan [f'(\sigma'_n)] \quad (4.40)$$

$$c' = \tau' - [f'(\sigma'_n)] \sigma'_n \quad (4.41)$$

with ϕ' as the slope of the tangent line to the strength envelope, and c' as the y-intercept of the tangent line. With ϕ' , c' defined, the initial FS_O can be solved using the Ordinary Method of Slices (Duncan and Wright, 2005; Wyllie and Mah, 2004):

$$\sigma'_{ni} = \frac{W_i \cos \alpha_i}{l_i} - u_i \cos^2 \alpha_i \quad (4.42)$$

$$RF = \sum [(c'_i + \sigma'_{ni} \tan \phi'_i) l_i] \quad (4.43)$$

$$FS_O = \frac{RF}{\sum (SF_i + seis F_i \left(\frac{v_i}{r}\right))} \quad (4.44)$$

where σ'_n is the effective normal stress on the slice base, RF is the total resisting force, and the quantity $\sum (SF_i + seis F_i \left(\frac{v_i}{r}\right))$ is the total driving force. v_i is the vertical distance between the arc center and the center of mass of the slice, and r is the radius of the arc.

The succeeding procedure involved the iterative solving of FS using the Simplified Bishop Method. From the ϕ'_i , c'_i , FS_O previously computed, new sets of normal force N_i and effective normal stress σ'_{ni} were computed:

$$N_i = \left[\frac{W_i - \frac{1}{FS_O} (c'_i l_i - u_i l_i \tan \phi'_i) \sin \alpha_i}{\cos \alpha_i + \frac{(\sin \alpha_i \tan \phi'_i)}{FS_O}} \right] \quad (4.45)$$

$$\sigma'_n = \frac{N_i}{l_i} - u_i \quad (4.46)$$

A new set of ϕ'_i , c'_i is computed from the new σ'_{ni} values using Equations (4.39), (4.40), and (4.41). FS is again solved using the new values from Equations (4.43) and (4.44), with FS replacing FS_O in the latter equation and in the succeeding iterations. The iteration continues until the difference between the two succeeding FS are less than 0.005.

The generation of the slope model, and the probabilistic and deterministic analyses were implemented in a customized Python 2.7 code (Python Software Foundation, 2014) using the Scipy Stack (Jones et al., 01) core libraries. While the program Slide (RocScience, ated) can perform the basic probabilistic and deterministic slope stability

Figure 77. Probabilistic analysis of slope stability for a particular slope geometry $[H, \beta]$ and lithologic group.

Figure 78. Deterministic analysis of slope stability for a particular set of material properties, set of failure surfaces, and external force variables.

Figure 79. Simplified Bishop analysis for a given set of material properties, a failure surface, and external force variables. Dashed box indicates iterative process of solving for FS.

analyses, the nested operations would be difficult to implement in Slide, and would require a customizable program.

4.3.2.2 Stability analysis for planar failures

4.3.2.2.1 Overview

Since the shear strength of the discontinuities were not assessed in the field, the stability analysis focused on the geometric tests in the kinematic analysis, and the effect of the discontinuity plane geometry on the stability conditions of the slope. The analysis for discontinuity-controlled failures was constrained to the planar type since the wedge and toppling failure analyses are difficult to implement in 2-dimensional slope stability analysis. A simplified regional kinematic analysis for planar failure was conducted, using structural data from the field. Stability conditions of kinematically admissible configurations were assessed in detail using the same uniform distributions of Barton-Bandis material parameters for all litho-structural units. Results are presented as plots of mean FS and probability of failure, and probability of failure plotted on the litho-structural basemap.

4.3.2.2.2 Kinematic analysis

Kinematic analysis was conducted for the discontinuities mapped in the field, as discussed in the chapter on Geology. Mean orientation was used to represent bedding planes in CC (CC_beds). For the folded FC unit, various bed orientations based on the pole concentrations were used (FC_beds_LipHim1, FC_beds_LipHim2, FC_beds_LipHim3, FC_beds_IlaNai1, FC_beds_IlaNai2, FC_beds_IlaNai3). Bedding plane for LS (LS_beds) were assumed to be the same as CC. Mean orientation of thrust faults were used to represent these structures in UM and FC (UM_T). The synthetic and antithetic Riedel shears of the PFZ (PFZ_R0, PFZ_R1) were modeled based on field data and the simple shear model. While Riedel shears were modeled to be near-vertical, a conservatively lower dip (40 degrees; but still higher than the dip of other structures) was used in the analysis to ensure daylighting conditions and kinematic admissibility. Normal and thrust

faults related to the PFZ as predicted by the simple shear model were not modeled since these were not encountered in the field, and there were no strong indications of their pervasive presence in the study area.

Data for the geologic structures considered in the analysis is presented in Table 15. The analysis conservatively assumed the presence of a structure in all pixels of the appropriate litho-structural units, and thus assumed conservatively high kinematic admissibility for each of the litho-structural units.

Data for the slope face is from the gradient, β_s , and aspect, θ_s , computed from the digital elevation model of the study area, based on the equations of Wood (1996) used in the chapter on Topography (Tab. 3, Eqs. 3.2 and 3.3).

With the dip, β_d , and dip direction, θ_d , of the discontinuity, a planar failure is geometrically feasible, or kinematically admissible, if all the following relations are satisfied:

$$|\theta_s - \theta_d| \leq 20 \quad (4.47)$$

$$\beta_s - \beta_d > 10 \quad (4.48)$$

Equation 4.47 constrains the dip direction of the discontinuity to within 20 degrees from the dip direction of the slope face, whereas Equation 4.48 checks for daylighting conditions.

If the slope face - discontinuity pair is assessed as kinematically admissible, the analysis proceeds to the stability calculations. If not, the pair is assigned a zero probability of failure.

4.3.2.2.3 Material properties

For the stability analysis, the same uniform distributions of discontinuity properties, as presented in the section on the Barton-Bandis failure criterion (Sec. 4.3.1.2), were used for all litho-structural units.

Structure ID	β_d	θ_d	CCw	CCe	FC	LS	UM
PFZ_R0	40	54	1	1	1	1	1
PFZ_R1	40	177	1	1	1	1	1
CC_beds	26	247	1	1	0	0	0
UM_T	21	159	0	0	1	0	1
FC_beds.LipHim1	31	239	0	0	1	0	0
FC_beds.LipHim2	36	142	0	0	1	0	0
FC_beds.LipHim3	26	269	0	0	1	0	0
FC_beds.IlaNai1	45	250	0	0	1	0	0
FC_beds.IlaNai2	61	125	0	0	1	0	0
LS_beds	26	247	0	0	0	1	0

Table 15. Data for structures used in the planar failure analysis. Ones and zeros under the columns for the litho-structural units indicate whether or not the unit is affected by the structure in the current row. PFZ_R0-synthetic Riedel shear; PFZ_R1-antithetic Riedel shear; UM_T-thrust faults in ultramafic and fine clastic rocks. The rest are bedding planes named after the lithologic unit, and specific location, if present.

4.3.2.2.4 Slope model

Slope models were defined by $\beta_d = [20, 25, 30, 35, 40, 50]$, which is a set of planar discontinuity dips. For each value in the set of β_d , a corresponding set of slope gradients β_s is defined for the interval $[\beta_d + 10, 85]$, with a constant base length of 30m. Each slope model has a horizontal surface at the top, which extends a distance equal to half of the slope base length. The planar discontinuity is made to exit at the toe and at the upper horizontal surface or the rear lateral boundary of the slope model. The area of the polygon bound by the slope and discontinuity surfaces, represents the potentially unstable mass, and its volume per unit width (area of the polygon, in 2 dimensions) is solved geometrically from the dimensions of the slope and discontinuity surfaces.

4.3.2.2.5 External forces

Constant expressions of external forces were used in the analysis. The pore water pressure condition was represented by an assumed triangular distribution of pressure acting on the discontinuity surface, and proportional to the height of the discontinuity surface (Wyllie and Mah, 2004). The force from this pressure is:

$$U = \frac{1}{2} \frac{z_w}{\sin \beta_d} h_w \gamma_w \quad (4.49)$$

where z_w is the height of the discontinuity surface, $\gamma_w = 9.8 \times 10^{-3} \text{ MN/m}^3$ is the unit weight of water, and h_w is the depth of the water above the midpoint of the discontinuity surface, which is set to $0.25z_w$ for this analysis. The term $\frac{z_w}{\sin \beta_d}$ is the length of the discontinuity surface, and represents the base of the pore pressure triangle. The term $h_w \gamma_w$ is the magnitude of the pore water pressure at the midpoint of the discontinuity surface, and represents the height of the pore pressure triangle.

The seismic load was determined by $k_h = 0.3$, similar to the analysis for circular failures. Three sets of factor of safety analysis were carried out with respect to the external forces: 1) no pore water pressure and seismic load, 2) with pore water pressure, no seismic load, and 3) with pore water pressure and seismic load.

4.3.2.2.6 Factor of safety analysis

For each slope model defined by $[\beta_d, \beta_s]$, a distribution of FS was computed from the distributions of material properties, the geometry of the potential failure mass and the external forces. The mass was treated as a rigid block, where the forces from the weight of the block, W , pore water pressure, U , and seismic loads, $k_h W$, act on the centroid of the mass. The equation for the factor of safety is (Wyllie and Mah, 2004):

$$FS = \frac{c' l + (W \cos \beta_d - k_h W \sin \beta_d - U) \tan \phi'}{W \sin \beta_d + k_h W \cos \beta_d} \quad (4.50)$$

where c', l and ϕ' are the cohesion, length and friction angle of the sliding plane, respectively. The weight of the potential failure mass is from:

$$W = \gamma V \quad (4.51)$$

where γ and V are the unit weight and volume per unit width of the block. The FS equation was solved using the Barton-Bandis Equations (4.15), (4.21), and (4.22), and the material properties, where

$$\sigma'_n = \frac{W \cos \beta_d - k_h W \sin \beta_d - U}{l} \quad (4.52)$$

$$c' = 0 \quad (4.53)$$

For each resulting FS distribution corresponding to the current $[\beta_d, \beta_s]$, the mean FS and probability of failure was computed, similar to the analysis for isotropic failures.

The analysis of the slope models, and the actual slopes on the map were implemented in a customized Python 2.7 code (Python Software Foundation, 2014) using the Scipy (Jones et al., 01), Numpy (Walt et al., 2011), Matplotlib ((Hunter, 2007), and Pandas (McKinney, 2010) libraries.

4.3.3 Landslide inventory

A landslide inventory was constructed based on Google Earth satellite images dated 2006/03/02 and 2012/05/07 (Google, 2015 *a,b*) covering the southern and northern portions of the study area, respectively. Landslides were identified primarily from the color and textural contrast between disturbed and undisturbed vegetation, characteristic planform morphology, and in a few cases, field-based identification conducted prior to the public release of the images. Because of limited data quality and opportunity for field validation, the inventory is only a point count of individual landslides which were assumed to have occurred shortly before the images were taken. No distinction was made in terms of the type, mechanism, and size of the landslides. Each landslide was represented as a vector point, whose coordinates correspond to the approximate center of the landslide source area. Each vector point contains data on slope gradient

and lithostructural unit classification based on the corresponding raster maps of gradient and lithostructural units.

Relative landslide susceptibility of each litho-structural unit, expressed as landslide density, was computed from the number of landslides for each unit divided by the corresponding area of the unit within the region covered by the imagery. Distribution of slope gradient of landslide source areas for each lithostructural unit was expressed as mean and standard deviation. Results from the landslide inventory were compared with those from the rock mass strength and stability analysis.

4.4 Results and discussion

4.4.1 Rock mass strength

4.4.1.1 Hoek-Brown failure criterion

4.4.1.1.1 Unconfined compressive strength

The empirical cumulative distribution functions of *UCS* are normally distributed (Fig. 80) based on results of the KS tests (Tab. 16). The means and standard deviations for each of the data sets were derived from the parameters of the corresponding fitted curves (Tab. 16). Plotted together, the litho-structural groups show graphically distinct distributions (Fig. 81). Results of the KS test (Tab. 16) for randomly generated data ($N=13$, the minimum number of field data for *UCS*), show distinct distributions except for those of LS and UM. The means and standard deviations for all litho-structural groups, based on the sampling distribution analysis (sample size=13, number of samples=100) (Fig. 82), are distinct for all litho-structural groups, as confirmed by results of the KS and Welch's Test (Tab. 17).

The *UCS* of the litho-structural groups generally have mutually distinct normal distributions, means and standard deviations. FC has the lowest mean *UCS*, followed by CC, LS, and UM.

Figure 80. UCS distribution-fitting over data with non-detects.

Unit	N	N>dl	Mean	Std dev	KS normal test		
					P-value	D _n ($\alpha = 0.05$)	H ₀ : normal
CC	58	52	16.4	6.7	0.04	0.19	TRUE
FC	16	8	5.4	2.4	0.09	0.46	TRUE
LS	13	13	22.9	9.9	0.21	0.36	TRUE
UM	24	23	31.9	10.7	0.12	0.28	TRUE

(a)

Unit	KS 2-sample test					
	P-value			H ₀ : same distribution		
	FC	LS	UM	FC	LS	UM
CC	0.00	0.03	0.00	FALSE	FALSE	FALSE
FC		0.00	0.00		FALSE	FALSE
LS			0.09			TRUE

(b)

Table 16. UCS distribution analysis. (a) Population parameters and hypothesis test for normal distribution. (b) Hypothesis test for equal distributions.

Figure 81. UCS population distribution expressed as cumulative (*Top*) and probability (*Bottom*) density plots.

Sample Statistic	Unit	Sampling Distribution		KS normal test		Welch's Test					
		Mean	Std dev	P- value	Ho: normal	P-value			Ho: equal means		
						FC	LS	UM	FC	LS	UM
Mean	CC	16.5	1.8	0.81	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC	5.4	0.6	0.70	TRUE		0.00	0.00		FALSE	FALSE
	LS	23.1	2.6	0.96	TRUE			0.00			FALSE
	UM	31.9	3.0	0.96	TRUE						
Std dev	CC	6.3	1.3	0.69	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC	2.2	0.3	0.94	TRUE		0.00	0.00		FALSE	FALSE
	LS	8.5	1.6	0.55	TRUE			0.00			FALSE
	UM	10.1	2.0	0.96	TRUE						

Table 17. UCS sampling distribution parameters, test for normality of sampling distribution, and test for equal estimates of the mean.

Figure 82. Sampling distribution plots of *UCS*. *Top:* Cumulative density function. *Bottom:* Probability density function. Sample size=13, number of samples=100.

4.4.1.1.2 Material constant

Material constants for the rock types which best represent the spectrum of lithologies in the study area are presented in Table 18a. The range of lithologies, overall minimum and maximum values, and the corresponding mean and standard deviations for the litho-structural groups in the study area are shown in Table 18b.

For random data generated from the assumed normal distribution ($N=9$, representing minimum number of GSI data), the distributions for litho-structural groups (Fig. 83) generally cannot be distinguished from each other (Tab. 19). However, the means and standard deviations of the litho-structural groups, as seen from the results of the sampling distribution analysis (sample size=9, number of samples=100), are mutually distinct (Fig. 84, Tab. 20). The litho-structural groups, arranged in order of increasing mean m_i , are LS, FC, CC, and UM.

Lithology	m_i
Breccia	19 ± 5
Conglomerate	21 ± 3
Greywacke	18 ± 3
Sandstone	17 ± 4
Siltstone	7 ± 2
Crystalline limestone	12 ± 3
Sparitic limestone	10 ± 2
Micritic limestone	9 ± 2
Peridotite	25 ± 5

(a)

Litho-structural group	Range of lithologies	Min	Max	Mean	Std dev
CC	Breccia, Conglomerate, Greywacke, Sandstone	13	24	18.5	2.75
FC	Siltstones, Sandstones	5	21	13	4
LS	Crystalline, Sparitic and Micritic Limestones	7	15	11	2
UM	Peridotite	20	30	25	2.5

(b)

Table 18. m_i values for (a) various lithologies, as published in RocScience (2007) and (b) litho-structural groups in the study area.

Figure 83. m_i population distribution expressed as cumulative (*Top*) and probability (*Bottom*) density plots from the assumed normal distributions.

Unit	KS 2-sample test					
	P-value			H_0 : same distribution		
	FC	LS	UM	FC	LS	UM
CC	0.96	0.96	0.08	TRUE	TRUE	TRUE
FC		0.02	0.08		FALSE	TRUE
LS			0.25			TRUE

Table 19. m_i distribution analysis, testing for equal distributions.

Sample Statistic	Unit	Sampling Distribution		KS normal test		Welch's Test					
		Mean	Std.dev	P-value	Ho: normal	P-value			Ho: equal means		
						FC	LS	UM	FC	LS	UM
Mean	CC	18.5	0.9	0.60	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC	13.0	1.5	0.98	TRUE		0.00	0.00		FALSE	FALSE
	LS	11.0	0.8	0.98	TRUE			0.00			FALSE
	UM	25.0	1.0	0.98	TRUE						
Std dev	CC	2.6	0.6	0.71	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC	3.6	0.8	0.89	TRUE		0.00	0.00		FALSE	FALSE
	LS	1.8	0.4	0.89	TRUE			0.00			FALSE
	UM	2.2	0.5	0.89	TRUE						

Table 20. m_i sampling distribution parameters, test for normality of sampling distribution, and test for equal estimates of the mean.

Figure 84. Sampling distribution plots of m_i . *Top:* Cumulative density function. *Bottom:* Probability density function. Sample size=9, number of samples=100.

Unit	N	Mean	Std dev	KS normal test	
				P-value	H ₀ : normal
CC	16	67.0	12.2	0.31	TRUE
FC	12	51.9	14.2	0.74	TRUE
LS	9	64.7	8.7	0.99	TRUE
UM	20	33.9	16.2	0.84	TRUE

(a)

Unit	KS 2-sample test					
	P-value			H ₀ : same distribution		
	FC	LS	UM	FC	LS	UM
CC	0.18	0.31	0.00	TRUE	TRUE	FALSE
FC		0.25	0.04		TRUE	FALSE
LS			0.00			FALSE

(b)

Table 21. *GSI* distribution analysis. (a) Population parameters and hypothesis test for normal distribution. (b) Hypothesis test for equal distributions.

Sample Statistic	Unit	Sampling Distribution		KS normal test		Welch's Test					
		Mean	Std.dev	P-value	H ₀ : normal	P-value			H ₀ : equal means		
						FC	LS	UM	FC	LS	UM
Mean	CC	66.4	3.6	0.89	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC	51.7	5.4	0.98	TRUE		0.00	0.00		FALSE	FALSE
	LS	64.6	3.4	0.98	TRUE			0.00			FALSE
	UM	33.7	6.3	0.98	TRUE						
Std dev	CC	10.7	2.6	0.65	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC	12.6	2.9	0.89	TRUE		0.00	0.00		FALSE	FALSE
	LS	7.8	1.8	0.89	TRUE			0.00			FALSE
	UM	14.5	3.4	0.89	TRUE						

Table 22. *GSI* sampling distribution parameters, test for normality of sampling distribution, and test for equal estimates of the mean.

4.4.1.1.3 Geological Strength Index

The *GSI* distributions are normally distributed (Fig. 85) based on results of the KS tests (Tab. 21a). From the actual sample data, all distributions cannot be distinguished from each other, except when compared with that of UM (Tab. 21b). However, means and standard deviations for all of the data sets, based on sampling distribution analysis (sample size=9, number of samples=100) (Fig. 86) are normally distributed and mutually distinct as confirmed by results of the KS and Welch's Test (Tab. 22). UM has the lowest mean *GSI*, followed by FC, LS, and CC.

Figure 85. *GSI* population distribution expressed as cumulative (*Top*) and probability (*Bottom*) density plots.

Figure 86. Sampling distribution plots of GSI . *Top:* Cumulative density function. *Bottom:* Probability density function. Sample size=9, number of samples=100.

4.4.1.1.4 Disturbance Factor

A constant value of $D = 0.7$ is assigned for all litho-structural groups. This factor represents disturbance due to stress relaxation in natural slopes.

4.4.1.1.5 Shear strength

The shear strength envelopes, produced by Monte Carlo simulation ($N = 10,000$), for the different litho-structural groups and for different slope height ranges are shown in Figure 87. The envelopes, at constant slope height range, generally show the highest and lowest shear strengths (equal to shear stress in the y-axis) for CC and FC, respectively. The simulation yielded distributions of shear strengths which, for all litho-structural groups and ranges of slope heights, rejected a normal distribution fit. Alternative distributions such as log-normal and beta were explored, and the former yielded the better fit (Figs. 88, Tab. 23a). Based on random samples ($N = 1,000$) all distributions are mutually distinct, except for those between LS and CC for the lowest range of slope heights (Tab. 23b).

Based on sampling distribution analysis, the means and standard deviations for all litho-structural groups, and for all slope height ranges, are generally normally distributed and mutually distinct (Figs. 89, 90 and Tab. 24).

Across all slope height ranges, FC has the lowest shear strength, followed by UM, LS, and CC.

4.4.1.1.6 Summary for Hoek-Brown failure criterion

The litho-structural groups in the study area generally have mutually distinct distributions (normal in shape), means and standard deviations for the field-measured UCS and GSI . With an assumed lithology-dependent and normally-distributed m_i , and a constant value of D , the corresponding shear strengths, for various ranges of slope heights, have mutually distinct distributions (log-normal in shape), means and standard deviations. The means and standard deviations of the different parameters are compiled in Table 25.

Figure 87. Shear strength envelopes ($N = 1000$) for the different litho-structural groups and for different slope height ranges .

Figure 88. Shear strength population distribution expressed as cumulative (*Top*) and probability (*Bottom*) density plots, derived from Monte Carlo simulation, $N = 10,000$. Dashed lines are fitted log-normal distributions.

Slope height (<i>m</i>)	Unit	Population		KS test			
		Mean	Std dev	P-value	H ₀ : normal	P-value	H ₀ : log-normal
5-10	CC	0.986	0.607	0.00	FALSE	0.03	FALSE
	FC	0.330	0.180	0.00	FALSE	0.24	TRUE
	LS	0.929	0.492	0.00	FALSE	0.01	FALSE
	UM	0.643	0.358	0.00	FALSE	0.10	TRUE
10-20	CC	1.373	0.671	0.00	FALSE	0.09	TRUE
	FC	0.476	0.249	0.00	FALSE	0.67	TRUE
	LS	1.263	0.543	0.00	FALSE	0.06	TRUE
	UM	0.958	0.512	0.00	FALSE	0.83	TRUE
20-50	CC	2.125	0.940	0.00	FALSE	0.23	TRUE
	FC	0.737	0.394	0.00	FALSE	0.91	TRUE
	LS	1.914	0.753	0.01	FALSE	0.55	TRUE
	UM	1.540	0.836	0.00	FALSE	0.79	TRUE
50-100	CC	3.200	1.342	0.01	FALSE	0.92	TRUE
	FC	1.091	0.576	0.00	FALSE	0.91	TRUE
	LS	2.839	1.058	0.01	FALSE	0.41	TRUE
	UM	2.531	1.268	0.00	FALSE	0.57	TRUE

(a)

Slope height (<i>m</i>)	Unit	KS 2-sample test					
		P-value			H ₀ : same distribution		
		FC	LS	UM	FC	LS	UM
5-10	CC	0.00	0.09	0.00	FALSE	TRUE	FALSE
	FC		0.00	0.00		FALSE	FALSE
	LS			0.00			FALSE
	UM						
10-20	CC	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC		0.00	0.00		FALSE	FALSE
	LS			0.00			FALSE
	UM						
20-50	CC	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC		0.00	0.00		FALSE	FALSE
	LS			0.00			FALSE
	UM						
50-100	CC	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC		0.00	0.00		FALSE	FALSE
	LS			0.00			FALSE
	UM						

(b)

Table 23. Shear strength distribution analysis. (a) Population parameters and hypothesis test for normal and log-normal distributions. (b) Hypothesis test for equal distributions.

Figure 89. Sampling distribution plots of shear strength (means). *Top:* Cumulative density function. *Bottom:* Probability density function.

Figure 90. Sampling distribution plots of shear strength (standard deviations). *Top:* Cumulative density function. *Bottom:* Probability density function.

Slope height (m)	Sample Statistic	Unit	Sampling distribution		KS normal test		Welch's test					
			Mean	Std dev	P-value	H ₀ : normal	P-value			H ₀ : equal means		
							FC	LS	UM	FC	LS	UM
5-10	mean	CC	0.993	0.117	0.328	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
		FC	0.329	0.034	0.970	TRUE		0.00	0.00		FALSE	FALSE
		LS	0.934	0.092	0.859	TRUE			0.00			FALSE
		UM	0.647	0.059	0.740	TRUE						
	std dev	CC	0.573	0.235	0.013	FALSE	0.00	0.00	0.00	FALSE	FALSE	FALSE
		FC	0.164	0.043	0.009	FALSE		0.00	0.00		FALSE	FALSE
		LS	0.476	0.151	0.081	TRUE			0.00			FALSE
		UM	0.349	0.120	0.001	FALSE						
10-20	mean	CC	1.378	0.130	0.786	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
		FC	0.474	0.046	0.984	TRUE		0.00	0.00		FALSE	FALSE
		LS	1.267	0.099	0.997	TRUE			0.00			FALSE
		UM	0.966	0.085	0.921	TRUE						
	std dev	CC	0.644	0.192	0.071	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
		FC	0.232	0.047	0.657	TRUE		0.00	0.00		FALSE	FALSE
		LS	0.532	0.122	0.177	TRUE			0.00			FALSE
		UM	0.506	0.125	0.015	FALSE						
20-50	mean	CC	2.130	0.182	0.963	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
		FC	0.735	0.073	0.951	TRUE		0.00	0.00		FALSE	FALSE
		LS	1.919	0.134	0.897	TRUE			0.00			FALSE
		UM	1.555	0.138	0.967	TRUE						
	std dev	CC	0.902	0.187	0.546	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
		FC	0.366	0.069	0.536	TRUE		0.00	0.00		FALSE	FALSE
		LS	0.740	0.123	1.000	TRUE			0.00			FALSE
		UM	0.826	0.181	0.124	TRUE						
50-100	mean	CC	3.207	0.262	0.865	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
		FC	1.089	0.107	0.939	TRUE		0.00	0.00		FALSE	FALSE
		LS	2.845	0.189	0.816	TRUE			0.00			FALSE
		UM	2.374	0.208	0.928	TRUE						
	std dev	CC	1.294	0.240	0.587	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
		FC	0.540	0.096	0.849	TRUE		0.00	0.00		FALSE	FALSE
		LS	1.039	0.162	0.985	TRUE			0.00			FALSE
		UM	1.246	0.259	0.090	TRUE						

Table 24. Shear strength sampling distribution parameters, test for normality of sampling distribution, and test for equal estimates of the mean for various ranges of slope height.

Shear strength envelopes using mean values of Hoek-Brown parameters are plotted for both intact rocks and rock masses (Fig. 91). For intact rocks, GSI and D are set to zero.

The litho-structural groups, arranged according to increasing intact rock shear strength are FC, CC, LS and UM. For the rock mass shear strength (discontinuities included), the order is FC, UM, LS, then CC. The rock mass shear strength of FC is limited by the very low UCS , a property of the intact rock comprising the rock mass. In contrast, the low rock mass shear strength of UM is mainly due to the low GSI , a property of the discontinuities within the rock mass. Furthermore, while CC possesses the second lowest UCS , it has the highest rock mass shear strength owing to better discontinuity conditions as reflected in the GSI . LS on the other hand, while having the second highest UCS and GSI , had relatively lower m_i , giving it rock mass shear strengths just slightly lower than CC.

Unit	UCS (MPa)	GSI	m_i	Shear strength (MPa)			
				$H = 5 - 10m$	$H = 10 - 20m$	$H = 20 - 50m$	$H = 50 - 100m$
CC	16	66	18.5	0.99	1.38	2.13	3.21
FC	5	52	13	0.33	0.47	0.74	1.09
LS	23	65	11	0.93	1.27	1.92	2.85
UM	32	34	25	0.65	0.97	1.56	2.37

Table 25. Summary of mean rock mass properties for Hoek-Brown failure criterion.

4.4.1.2 Barton-Bandis failure criterion

The assumed values for the material characterization using the Barton-Bandis failure criterion will be used in the stability analysis for planar failure.

4.4.2 Slope Stability

4.4.2.1 Stability analysis for circular failures

Results for the stability analysis are presented as plots of slope and critical failure surfaces (Figs. 92, 93), histograms (Fig. 94), and summary tables and plots of mean FS and probability of failure (Tab. 26, Fig. 95). The mean lower exit locations of the failure surfaces are also presented (Tab. 26).

Figure 91. Mean strength envelopes of intact rocks (with suffix *i*) and rock masses (with suffix *m*) for the litho-structural units.

For all litho-structural units, the slope and critical failure surface plots and histograms generally show decreasing factors of safety as the gradient increases. This is seen as the increasing proportion of red failure surfaces (Figs. 92 and 93), and increasing portion of red histogram bars (Fig. 94), as slope gradient increases. Differences across litho-structural units for constant slope gradients are not strikingly apparent in the slope and critical failure surface plots since many failure surfaces coincide, and the manner of plotting is that the failure surface with the lowest value is plotted over the rest of the coinciding surfaces; same also for the histograms, because the y-axes are not equally scaled.

The slope and critical failure surface plots also show that critical failure surfaces for $\beta = 15$ generally exit beyond the slope toe, and as the gradient increases, the exits move closer to the toe, then towards the lower part of the slope face. This trend is more apparent in the tabulated mean lateral location of failure surface lower exit points, expressed as percentage of the slope base length (Tab. 26). Negative and positive values are to the left and right of the toe, respectively. Higher absolute values indicate mean exit locations farther from the toe. Failure occurring above the toe and within the slope face (corresponding to a highly positive exit location) suggests that the slope would have already failed at a lower height at the same gradient. Thus, the current slope height is likely close to or greater than the allowable height imposed by the slope material. This is reflected in the close to 100% probability of failure for toe exits $>10\%$ at slope gradients equal to or above 60 degrees (Tab. 26).

There is a generally decreasing trend of mean FS from CC, LS, UM to FC at constant slope gradients (Tab. 26, Fig. 95). Conversely, there is a generally increasing trend in probability of failure from FC, UM, LS to CC at constant slope gradients (Tab. 26, Fig. 95). Data for CC and LS are close to each other, whereas those of FC and UM are clearly distinct from those of all the other litho-structural units.

The probability of failure may also be interpreted as a cumulative failure rate and can be fitted with a cumulative distribution. Modeling of β values approaching 0 and 90 degrees shows that the cumulative failure rates for all litho-structural units converge to 0 and 100%, respectively. The trend of data indicates a monotonically increasing, non-symmetric function. The beta distribution (Johnson et al., 1995) was chosen since its cumulative distribution function (CDF) can represent the observed data trend on the closed interval, $[0, 90]$.

Since the cumulative failure rate is approximated by the beta CDF, the incremental failure rate can be estimated from the beta probability distribution function (PDF), which is the derivative of the beta CDF (Fig. 96). The gradient corresponding to the maximum incremental failure rate may be considered as a critical or threshold gradient

	unit	slope gradient (<i>degrees</i>)				
		15	30	45	60	75
Mean FS	CC	7.14	4.40	2.62	1.17	0.17
	FC	2.20	1.33	0.74	0.22	0.02
	LS	6.67	4.10	2.38	0.91	0.04
	UM	3.46	1.98	0.91	0.09	0.01
Slip surface exit location	CC	-43.5	-1.5	0.3	1.1	7.1
	FC	-29.1	-1.0	0.8	3.3	11.7
	LS	-38.1	-2.1	0.2	0.8	6.9
	UM	-8.8	-0.6	6.4	13.3	16.7
Probability of failure	CC	1.0	1.3	10.9	55.7	94.3
	FC	12.5	40.8	78.6	95.7	99.9
	LS	1.4	2.0	7.8	64.0	99.2
	UM	2.6	13.4	71.6	98.0	100.0

Table 26. Results of probabilistic slope stability analysis (isotropic failure) of litho-structural units for various slope gradients.

wherein a large number of slopes fail as this gradient value is approached, similar to the angle of repose being exceeded as a slope is oversteepened.

For all litho-structural units, the sampling distribution of threshold gradients are normal and mutually distinct (Fig. 96, Tab. 27), based on results of the KS test (Kolmogorov, 1992; Smirnov, 1948,3), and Welch's Test (Welch, 1947).

Values for mean FS, probability of failure, and the derived threshold gradient indicate generally increasing slope stability from FC, UM, LS, to CC. While these failure rates do not necessarily represent the actual frequency of slope failures, these values express the relative stability conditions of the units. The slope stability trend is the same as the trend of rock mass shear strength (also, $FC < UM < LS < CC$), suggesting a qualitatively good correlation between rock mass shear strength and slope stability.

The probability of failure map (Fig. 97), generated from the fitted beta cumulative distribution and the corresponding actual gradient map (Fig. 55), shows generally high failure rates for FC and UM, generally low failure rates for LS, and wide range of values for CCe and CCw.

Figure 93. Probabilistic slope stability analysis results showing critical failure surfaces for various slope gradients, $\beta = 75, 60, 45, 30, 15$ (top to bottom). Reference color bar for FS values provided.

Figure 94. Normalized histogram of FS for various slope gradients, $\beta = 75, 60, 45, 30, 15$ (top to bottom), and litho-structural units. Note that y axes are not equally scaled. Bin width=0.2. Red and blue bars represent $FS < 1$ and $FS \geq 1$, respectively.

Sample Statistic	Unit	Sampling Distribution		KS normal test		Welch's Test					
		Mean	Std.dev	P- value	Ho: normal	P-value			Ho: equal means		
						FC	LS	UM	FC	LS	UM
Mode	CC	60.9	3.4	0.927	TRUE	0.00	0.00	0.00	FALSE	FALSE	FALSE
	FC	32.6	5.5	0.734	TRUE		0.00	0.00		FALSE	FALSE
	LS	57.8	2.5	0.827	TRUE			0.00			FALSE
	UM	40.2	2.8	0.906	TRUE						

Table 27. Threshold gradient sampling distribution parameters, test for normality of sampling distribution, and test for equal estimates of the mean. Threshold gradients derived from modes of the fitted beta PDF (Fig. 96).

Figure 95. Summary plot of probabilistic slope stability analysis results. *Top:* mean factor of safety (FS) as function of slope gradient; FS=1 indicated by gray line. *Bottom :* Probability of failure as function of slope gradient.

Figure 96. Failure rate fitted with beta distribution. *Top:* Beta CDF as cumulative failure rate. *Middle:* Beta PDF representing incremental failure rate. *Bottom:* Sampling distribution of modes of beta PDF representing the threshold gradient.

Figure 97. *Left:* Litho-structural map. *Right:* Probability of failure (%) derived from topographic gradient (Fig. 55) and the fitted beta distribution (Fig. 96).

4.4.2.2 Stability analysis for planar failures

Results of stability analysis mainly highlight the effect of the orientation of geologic structures, since the strength characterization of the discontinuities was not conducted.

The modeling of kinematically admissible failures show an overall decrease in mean FS and increase in probability of failure as the dip of the planar sliding surface increases (Fig. 98). For constant discontinuity dip values, FS values generally decrease as slope gradient increases. However, when pore water pressure and seismic load are introduced, there is a notably larger reduction of FS for smaller $|\beta_s - \beta_d|$, resulting to a non-monotonic trend (dashed and dotted lines in Fig. 98). The plots also show the kinematic restrictions wherein FS is not assessed for $\beta_s < \beta_d + 10$ since these ranges represent failure geometry that are not kinematically admissible.

With the addition of external forces, the range of mean FS and probability of failures are significantly reduced (Fig. 98). For purposes of highlighting the effect of discontinuity geometry on the stability conditions, the succeeding discussion will focus on the analysis setting where no external forces are involved.

Results of kinematic and stability analyses for the actual slopes are integrated in tabular form (Tab. 28) and shown in maps of probability of failure (Figs. 99, 100, 101, 102, and 103). Overall, the number of kinematically admissible failures, compared to the total number of slopes is small. The highest proportion, only 3%, occurs for bedding plane failures in CCw. As discontinuity dip values reach 40 degrees, only CCe and CCw exhibit admissible failures. For the stability analysis of these failures, the general trend of increasing probability of failure and decreasing mean FS as the discontinuity dip increases is observed. Maximum probability of failure is 75% for planar failures in CCe along the steeply-dipping synthetic Riedel shears.

Failures along the PFZ Riedel shears (PFZ_R0,PFZ_R1) (Fig. 99) generally have the highest probability of failure, despite having limited spatial distribution owing to the kinematic requisites of steeper slope gradients. Failures along the synthetic Riedel Shear

(PFZ_R0) have a dense linear concentration along a section of the Can-abag ridge on the side of CCe, suggesting that this part of the ridge maybe strongly controlled by the structure.

For the bedding planes in CC (CC_beds) (Fig. 100), there is a preferential distribution of admissible failures favoring CCw over CCe. This supports the earlier-discussed structural anisotropy for CC, which warranted its subdivision into CCw and CCe. The CC beds have the widest spatial distribution of failure, both in terms of absolute number, and in terms of proportion to total number of slopes. Although widely distributed, the probability of failure is low, mainly due to the gentle dip of the discontinuity.

Thrust faults affecting UM and FC (UM_T) (Fig. 100), have high kinematic admissibility, owing to the lowest dip value of the discontinuity. However, because also of the dip, the probability of failure is also the lowest.

Failure along FC beds, are spatially constrained by the kinematic requisite of slope gradients being steeper than the dip of the discontinuities. LS has a dense concentration of kinematically admissible failures at the SW-facing steep rock slope faces in Ilag. However, the failure rate is small due to the gentle dip of the discontinuity.

4.4.3 Landslide inventory

The satellite images used in the inventory correspond to extended periods of rainfall which produced numerous landslides. The 2006 image was taken less than a month after the Guinsaugon landslide (Catane et al., 2008; Evans et al., 2007; Lagmay et al., 2006; Orense and Sapuay, 2006). Figure 104 shows the identified landslides. Tables 29a and 29b present data derived from the inventories. For both images, data from units comprising < 10% of the total area (CCw and CCe in 2006-03-02; FC, LS, and UM in 2012-05-07) were disregarded to minimize skewed results due to the units' relatively smaller sample sizes. Direct comparison between the two images was not done because the data sets are mutually exclusive in time and space.

Figure 98. Mean FS (*Top*) and probability of failure (*Bottom*) for discontinuity dip, $\beta_d = [20, 25, 30, 35, 40, 50]$, as function of slope gradient. Solid lines, no pore water pressure and seismic load. Dashed lines, with pore water pressure, no seismic load. Dotted lines, with pore water pressure and seismic load.

	Structure	CC beds	FC beds IlaNai1	FC beds IlaNai2	FC beds LipHim1	FC beds LipHim2	FC beds LipHim3	LS beds	PFZ R0	PFZ R1	UM_T
	Dip	26	45	61	31	36	26	26	40	40	21
	Dip direction	247	250	125	239	142	269	247	054	177	159
Kinematically admissible (%)	CCe	0.17							1.23	0.08	
	CCw	3.00							0.04	0.16	
	FC				0.02		0.17				0.56
	LS							1.87			
	UM										1.35
Mean factor of safety	CCe	1.49							0.82	0.88	
	CCw	1.47							0.88	0.87	
	FC				1.28		1.54				1.92
	LS							1.41			
	UM										1.90
Mean probability of failure(%)	CCe	23.8							75.2	68.9	
	CCw	24.4							69.1	70.2	
	FC				35.7		22.5				11.9
	LS							26.4			
	UM										12.1

Table 28. Results of kinematic and stability analysis for planar failures

Figure 99. Probability of failure for Riedel shears, PFZ_R0 (Left) and PFZ_R1 (Right) associated with PFZ model plotted on map of litho-structural units. Stereonets show the orientation of the modeled structures; color bars indicate probability of failure.

Figure 100. Probability of failure for bedding planes in CC (*Left*), and thrust faults in UM and FC (*Right*) plotted on map of litho-structural units. Stereonets show the orientation of the modeled structures; color bars indicate probability of failure.

Figure 101. Probability of failure for bedding planes in FC plotted on map of litho-structural units. Kinematically admissible failures are absent for FC_beds_LipHim2 (*Right*), hence, no data on probability of failure. Stereonets show the orientation of the modeled structures; color bar indicates probability of failure.

Figure 102. Probability of failure for bedding planes in FC plotted on map of litho-structural units. Kinematically admissible failures are absent for FC_beds_IlaIai1 (*Right*), hence, no data on probability of failure. Stereonets show the orientation of the modeled structures; color bar indicates probability of failure.

Figure 103. Probability of failure for bedding planes in FC (*Left*) and LS (*Right*) plotted on map of litho-structural units. Kinematically admissible failures are absent for FC_beds_IlaIai2 (*Left*), hence, no data on probability of failure. Stereonets show the orientation of the modeled structures; color bar indicates probability of failure.

For the 2006-03-02 image, landslide source area gradient decreases from CCw to LS to UM to FC. This trend agrees with that of the threshold gradient (Fig. 96), with steeper landslide source areas corresponding to steeper threshold gradients. Units FC, UM and LS have areas exceeding 10% of the total area, and were considered for the landslide density analysis. Landslide densities increase from LS to UM to FC (Tab. 29a). The trend of landslide density agrees with that of rock mass strength (Figs. 89 and 91), with higher landslide densities corresponding to lower rock mass strengths.

For the 2012-05-07 image, CCe has higher landslide source area gradients than UM, consistent with the trend in threshold gradient (Fig. 96). CCe and CCw have areas exceeding 10% of the total area, and were considered for the landslide density analysis. CCe has higher landslide densities than CCw (Tab. 29b). This trend agrees with that of the probability of planar slope failures (Tab. 28), wherein the CCe slope failures along the steeply-dipping PFZ_R0 have higher probabilities of failure than any of the planar failures within CCw.

. The limited temporal and spatial coverage of the landslide inventory reduced the reliability of data used in the analysis. However, the ordering of the litho-structural units in the results of both the inventory and the strength/stability analyses point to a spatial distribution of landslides that is likely dependent on the modeled shear strength and stability conditions of the litho-structural units in the study area.

4.5 Conclusion

Rock mass shear strength and stability analysis has been characterized for isotropic rock masses and, to some extent, for anisotropic rock masses representing the litho-structural units in the study area.

For the isotropic failures, rock mass shear strengths have been shown to have an influence on the stability conditions of the litho-structural units. The units, arranged according to increasing rock mass shear strength and stability conditions, are FC, UM, LS, and CC. Very low intact rock strengths limit the stability of FC. In contrast, poor

Figure 104. Landslide inventory map showing distribution of landslides across the different litho-structural units. Inventory based on Google (2015 *a,b*). Shaded relief image basemap, in UTM WGS84, from NAMRIA digitized topographic contour maps (NAMRIA, undated *a,b*).

Unit	Number of land-slides	Gradient (<i>deg</i>)		Area (<i>km</i> ²)	% total area	Landslide density (<i>km</i> ⁻²)
		Mean	Std dev			
CCw	2	36.6	7.1	3.2	9.9	0.6
CCe	0	na	na	1.7	5.2	0.0
FC	19	20.7	8.4	7.8	24.2	2.4
LS	2	31.6	0.1	9.3	28.9	0.2
UM	15	24.3	9.4	10	31.8	1.5

(a)

Unit	Number of land-slides	Gradient (<i>deg</i>)		Area (<i>km</i> ²)	% total area	Landslide density (<i>km</i> ⁻²)
		Mean	Std dev			
CCw	0	na	na	41	61.7	0.0
CCe	13	33.1	14.1	18	27.3	0.72
FC	0	na	na	1.8	2.7	0.0
LS	0	na	na	0.31	0.5	0.0
UM	2	12.7	0.2	5.2	7.9	0.4

(b)

Table 29. Landslide inventory results. (a) 2006-03-02 image shows increasing source area gradient from FC to UM to LS, and increasing landslide density from LS to UM to FC. CCe and CCw were excluded in density analysis due to limited area representation (each at < 10% of total area coverage). (b) 2012-05-07 image shows CCe with higher source area gradient than UM, and CCe with higher landslide density than CCw. FC, LS and UM were excluded in landslide density analysis due to limited area representation (each at < 10% of total area coverage).

discontinuity conditions within the rock mass make UM unstable, despite high intact rock strengths. CC is the most stable, even with low intact rock strength, because of the good discontinuity conditions. LS, while having high discontinuity strength and *UCS*, has low values for material constant, making it slightly less stable than CC.

Plots of mean factor of safety and probability of failure provide expressions of the slope stability for a range of slope gradients and a set of material strength parameters (Fig. 95). Increasing values of slope gradient result to decreasing stability, consistent with the generally expected trend. A critical or threshold gradient, derived from the probability of failure plot, provides a single-value indicator for the shear strength and stability of the rock mass. These threshold gradients follow a similar trend as the rock mass shear strengths (Figs. 89, 96).

The analysis for planar failures focused on the effect of discontinuity orientation, rather than on the strength of the discontinuities due to logistical constraints in quantifying the latter. The kinematic criteria, determined by the orientation of the discontinuities, and the actual gradient and aspect values of the slope in the study area, provide major constraints to the spatial distribution of planar failures in the area, as seen in the sparse distribution of admissible failures. For cases wherein the failures are admissible, the probability of failure increases with the dip of the discontinuity and slope gradient.

Slopes affected by PFZ Riedel shears generally have limited spatial distribution, mostly within the CC units, but have high probability of failure. Failures along CC beds have preferential occurrence in the CCw units, but have low probability of failure due to low dip angles. Failures along thrust faults are widely distributed in UM and FC but have low probability of failure due to low dip angles. Failures along FC beds, on the other hand, have limited distribution due to steeper dip angles than the actual slope gradients. LS has planar failures concentrated along steep, SW-facing rock faces, but with low probability of failure.

Landslide inventory analysis, using spatially- and temporally-constrained Google Earth images, provide indicative validation of the strength and stability modeling results.

CHAPTER 5

ROCK MASS STRENGTH AS LINK BETWEEN TOPOGRAPHY AND GEOLOGY

5.1 Introduction

The spatial distribution of the litho-structural units, CCw, CCe, FC, LS and UM reflects the material control on the topography of the study area. Based on gradient and cross sectional curvature, the slope materials underlying the different units were characterized as weak or strong materials; based on geomorphic form and process relationships, the dominant surface processes within the units were inferred and characterized as landslide- or channel- dominated. The previous chapter on rock mass strength was able to show differences between the strengths and stability conditions of the litho-structural units. This chapter will integrate the results and discussion from the previous chapters.

5.2 Material-Process-Form

The discussion will follow a framework based on the interaction of slope materials, processes and form. The slope materials refer to the geologic materials, in particular, but not limited to, rock mass strengths. Surface processes refer to the slope stability conditions, as well as other mechanisms of erosion and transport. Surface form refers to the topography and the parameters used to characterize it.

In this framework, the slope materials are influenced by the stratigraphy and tectonic history of the area, and impose some control on the stability of slopes. The mechanism, frequency and magnitude of slope failures influence the character of material transport, which in the long term, is reflected in the topography of a landscape.

The slope materials and form have been discussed in detail in the previous chapters. However, the processes which link the two have only been inferred and discussed briefly. The next section will seek to qualify how surface processes link slope materials and topographic form in the study area.

5.3 Rock mass strength and topography

In the chapter on Topography, the litho-structural units were broadly classified into either low-gradient, low-curvature terrain, or high-gradient, high-curvature terrain. LS, UM and FC were put into the former category, and CCe and CCw into the latter. A high-gradient, moderate-curvature category was later added to accommodate the locally inverse gradient-curvature relation between CCe and CCw. CCe was transferred into this new category. The above categorizations are mainly based on the central measures of the topographic parameters representing the litho-structural units. While LS has been described in the field as forming near-vertical cliffs (and is generally known to do so), most of the LS pixels in the study area are low-gradient. These dominantly low-gradient pixels mask the presence of the steep LS slopes in the gradient distribution, hence the classification of the unit as low-gradient.

High gradient terrain was attributed to strong rocks, and the low-gradient terrain to weak rocks. Transport systems were proposed for the three categories: landslide-dominated for the low-gradient, low-curvature units; channel-dominated for the high-gradient, high curvature terrain; and channel-dominated with episodic, massive landsliding for the high-gradient, moderate curvature terrain. The above qualitative categorizations are summarized in Table 30.

In the chapter on Rock Mass Strength, probability of failure and threshold gradient curves for the different litho-structural units were derived from direct measures of rock strength parameters (Fig. 96). Both curves indicate increasing rock mass strength from FC, UM, LS to CC. The plot of threshold gradient, representing rock strength, against the mean gradient and the mean curvature of channels (Fig. 105) show the low-gradient, low-curvature LS unit having strength distributions closer to CCe and CCw, than to UM and FC. This is contrary to the strength classification of LS inferred from topographic parameters in Table 30, and will be explained later.

Unit	Topography	Material strength	Transport process
CCw	High gradient, high curvature	Strong	Mostly channelized
CCe	High gradient, moderate curvature	Strong	Mostly channelized, with episodic massive landslides
FC	Low-gradient, low-curvature	Weak	Mostly landslides
LS	Low-gradient, low-curvature	Weak	Mostly landslides
UM	Low-gradient, low-curvature	Weak	Mostly landslides

Table 30. Material strengths and transport processes inferred from topography.

Figure 105. Plot of threshold gradient vs mean gradient and channel curvature showing low-gradient, low-curvature, weak FC and UM; low-gradient, low-curvature, strong LS; high-gradient, moderate-curvature, strong CCe; and high-gradient, high-curvature, strong CCw.

The probability of failure of each litho-structural unit was integrated with the corresponding slope gradient distribution (Fig. 106). Stability conditions of the slope models (Fig. 106, middle plot) were translated into the actual slope gradient distributions in the study area (Fig. 106, top plot). Results (Fig. 106, bottom plot) show the maximum probability of failure at a given percentile of the gradient distribution. At least upto 95% cumulative frequency, FC has the highest probability of failure, followed by UM,

CC (CCe and CCw have similar distributions), and then LS. The map of probability of failure (Fig. 97) reflects this, showing the southern and northeastern parts of the study area, both underlain by FC and UM, with generally higher probabilities of failure relative to the areas underlain by CCe, CCw and LS.

Figure 106. Integrated results from topographic and slope stability analysis. *Top:* cumulative slope gradient distribution for the study area (from chapter on Topography). *Middle:* Probability of failure as function of slope gradient (from chapter on Rock Mass Strength). *Bottom:* Maximum probability of failure as function of percentile of gradient distribution.

5.3.1 Strong rock, steep slopes

The trend for FC, UM, CCe and CCw shown in Figure 106 is consistent with results of previous works (Augustinus, 1995; Burbank and Pinter, 1999; Densmore et al., 1997; Korup, 2008; Kühni and Pfiffner, 2001; Moon and Selby, 1983; Roering et al., 2005; Schmidt and Montgomery, 1995), where the weaker units (high probability of failure)

have generally lower gradient distributions. LS deviated from this, having gradient distributions more similar to the weaker FC and UM units, but with lower probabilities of failure more similar to the stronger CC units.

5.3.2 Strong rock, gentle slopes

LS represents a strong rock mass unit which exhibits low-gradient terrain, and poses a contradiction between topography-inferred and actual rock mass strengths. One explanation for this minimal relief development is limited stream dissection, resulting to lower gradient values. This reasoning is supported by the smallest proportion of channels, and second lowest absolute channel curvatures of LS compared with the rest of the units (Figs. 66 and 67) . While LS does not display a striking karstic topography, the potential for subsurface water transport in limestones, may have also played a role in further limiting the development of stream processes within the unit.

Another factor which may contribute to minimal relief development is the available vertical relief. Compared to CCw and CCE, LS has a narrower distribution of elevation (Figs. 53 and 54), suggesting a smaller total vertical relief, which would impose limits to the maximum gradient that can be developed over the terrain. Simply put, thick (vertically speaking) units have greater chances of developing higher relief and gradients than thin units.

Rock mass strength of LS may not have played a significant role in limiting the development of gradient in the unit. However, the unit's overlying stratigraphic relationship with FC and UM may have allowed these older units, in some instances, to indirectly impose the gradient-limiting effect of their low rock mass strengths on LS. For slopes where the contact between LS and FC or UM are exposed, with the latter at the bottom of the slope, slope failure may be initiated or controlled at the base by the weaker units, and may retrogress to the overlying LS unit causing it to fail.

While the overall topography of LS may be considered low-gradient, the local occurrence of cliffs within this unit attest to the unit's high rock mass strength.

5.3.3 Role of planar failures

In the above discussion, as well as in the section on isotropic rock mass strength in the Rock Mass Strength chapter, CCe and CCw are treated as units with equal strengths. However, gradient and curvature distributions for both units (Figs. 56, 57, 59 and 60) show differences suggesting differences in surface processes operating within the units. In the chapter on topography, this was explained by dominant channel processes in CCw, which tend to decrease the unit's gradient distribution, and increase the curvature of its channels, relative to CCe. The suppressed channel topography of CCe, relative to CCw, suggest the occurrence of episodic but massive landslides, referred to as slope clearing events (Densmore et al., 1997), which either fill up the existing channels with sediments or erode the existing topographic surface. With this, the assumption of similar rock mass strengths and slope stability conditions does not hold true.

Planar failures provide an explanation for the above discrepancy. For CCe and CCw, the highest probability of failure occurs along the synthetic (PFZ_R0) and antithetic (PFZ_R1) Riedel shears (Tab. 28). Failures are kinematically more common along PFZ_R0 in CCe, than along either Riedel shears in CCw (Tab. 28). PFZ_R0 failures in CCe are pronouncedly more contiguous, and are located along the main ridgeline (Fig. 99). This suggests potentially larger landslide volumes and larger slope-clearing events, for CCe than for CCw. And while such massive landslides are more common in CCe, significant lowering of the unit's gradient is not expected. The sliding plane, which dictates the post-landslide topography of the landslide source area, is already steep in the first place, and in fact, provides the minimum gradient for an area dominated by such planar failures.

Table 31 updates Table 30 and summarizes the above discussion.

The contribution of planar failures to landslide processes varies with the isotropic rock mass strengths of slope materials. Weak isotropic rock mass strengths reduce the chances of developing steeper slopes which could provide daylighting conditions

Unit	Material strength	Transport process	Topography
CCw	strong isotropic rock mass, with steep potential failure planes (kinematically less frequent than in CCe)	mostly channelized transport	high gradient, high curvature
CCe	strong isotropic rock mass, with steep potential failure planes (kinematically more frequent than in CCw)	channelized transport, with episodic massive landslides	high gradient, moderate curvature
FC	weak isotropic rock mass	mostly landslides (isotropic failures)	low gradient, low curvature
LS	strong isotropic rock mass	limited role of landslides and channelized transport	low gradient, low curvature
UM	weak isotropic rock mass	mostly landslides (isotropic failures)	low gradient, low curvature

Table 31. Material strengths linked to topography by transport processes

for potential planar failures. In contrast, high isotropic rock mass strengths allow the development of steep slopes, and thus increase the kinematic admissibility of planar failures. Anisotropic failures are thus, less likely to occur within the gentle slopes underlain by UM and FC. However, the discontinuities associated with these unlikely planar failures contribute to the whole set of discontinuities defining the isotropic rock masses. Anisotropic failures may have significant direct contributions to landslide processes within the steep slopes underlain by CCe and CCw in instances where the isotropic rock mass is stronger than the planar discontinuity, with the latter providing the critical strength at failure conditions.

The above discussion has possible implications to the frequency and magnitude of landslide processes in the study area. Isotropic failures are likely to be more frequent in FC and UM, compared to CCe and CCw, based on the probabilities of failure. However the individual failures, occurring at generally low gradients, would tend to have smaller volumes. In contrast, both the isotropic and anisotropic failures in CCe and CCw are likely to be less frequent based on the probability of failure and kinematic constraints.

The strong rock masses and infrequent lowering of gradient by landslides in areas underlain by CCE and CCw allow the development of steep slopes and large potential landslide volumes which could be mobilized when critical conditions are reached. Previous discussion on CCE and CCw, however, has pointed to a more channel-dominated processes for CCw, leaving the massive landslide-dominated processes to CCE. This discussion thus highlights the role of anisotropic, planar failures in facilitating large landslides in areas underlain by materials with high isotropic strengths.

5.3.4 Landslide occurrence

The landslide inventories in Chapter 4 provide a snapshot of landslide susceptibility and frequency in the study area. Compared to the topography which was shaped over thousands to millions of years, the inventories provide a very limited view of long term landslide processes. However constrained this observation window may be, the results somehow indicate similar landslide rates as those predicted by the strength and stability analyses. The landslide densities derived from the inventory (Tabs. 29a and 29b) support the idea of relatively less dominant landslide processes in CCw relative to CCE; and in LS relative to UM and FC (Tab. 31).

5.4 Rock mass strength and geology

This section discusses how rock mass strength is influenced by the geologic conditions in the area.

5.4.1 Influence of lithology

Finer grains and clastic textures have been associated with weaker rocks; and coarser, crystalline textures are associated with stronger rocks (Korup, 2008; Kühni and Pfiffner, 2001; RocScience, 2007; Roering et al., 2005; Schmidt and Montgomery, 1995). The mean shear strength envelopes (Fig. 91) for the intact rocks, as a function of UCS and m_i , show increasing values from FC, CC, LS, to UM. For the same sequence, the

general textural characteristic trends are from clastic to crystalline, and from finer- to coarser-grained. Thus, the texture of lithologies in the study area may have a significant influence on the intact rock shear strength.

The control of rock type is quite significant for FC, where the intact rock shear strength is the lowest among the litho-structural units in the area, mainly because of the lowest and second to the lowest values of UCS and m_i for FC, respectively (Figs. 80-84, Tab. 25). This influence extends into rock mass shear strength and stability conditions, where FC also has the lowest value.

For the case of LS, the high UCS , despite the lowest m_i , resulted to higher values of intact rock and rock mass shear strengths, and better stability conditions. The high intact rock strength of LS could be attributed to the crystalline texture of the rock, and the abundant carbonate minerals which can act as cementing material.

5.4.2 Influence of structures

5.4.2.1 Isotropic failures

The plot of intact rock mean shear strength envelopes and their correspondingly reduced rock mass counterparts (Fig. 91) clearly illustrate the effect of discontinuities on the strength of rock masses. Despite having the highest intact rock shear strength (UCS and m_i ; Figs. 80-84, Tab. 25) among the litho-structural units, the rock mass shear strength of UM is significantly reduced by the low discontinuity strength as reflected in its GSI values (Figs. 85-86, Tab. 25).

UM outcrops exhibit, at best, blocky to very blocky structures defined by at least three to four intersecting discontinuity sets; and at worst, highly sheared rock masses. This characteristic structure may be due to the age of the unit and the tectonic processes it has undergone. Clarke and Burbank (2010) discusses two modes of bedrock fracturing which may have contributed strongly to the structure of the UM unit. The first mode, tectonic fracturing, involves fracturing of bedrock at depth due to thrust faults following the “tectonic rock crusher” model of Molnar et al. (2007). This mode may correspond to

the emplacement of the ophiolitic UM during the Late Cretaceous (Metal Mining Agency of Japan and Japan International Cooperation Agency (MMAJ-JICA), 1990; Mines and Geosciences Bureau (MGB), 2010), which implies an already deeply-fractured rock mass even prior to surface exposure. The second mode, geomorphic fracturing, involves fracturing of surface to near-surface bedrock by surface processes, as well as by stress relief due to various topographic configurations. This second mode, while likely common to all litho-structural units in the area, serves to exacerbate the existing fracturing of UM obtained from the first mode. The long and complex fracturing history of UM, including the post-emplacement, pre-Philippine Fault Zone compressional and tensional events (Aurelio et al., 1991; Metal Mining Agency of Japan and Japan International Cooperation Agency (MMAJ-JICA), 1990; Yumul et al., 2008), and the present stress regime from the PFZ (Aurelio, 2000; Besana and Ando, 2005), contributed to the highly fractured to sheared structure of the UM rock masses.

In contrast to UM, the CCe and CCw units have higher rock mass shear strengths, despite low *UCS* (Figs. 80-82, Tab. 25), because of better structure conditions as reflected in the higher *GSI* values (Figs. 85-86, Tab. 25). Discounting the bedding planes, the units are, at best, massive or intact, as seen from outcrops and car- to house-sized, intact sandstone and breccia boulders along the path of the Guinsaugon landslide. Even close to major fault planes, rock masses can still be described as massive, as seen in the source area of the smaller Guinsaugon-like landslide in Magkasag. Being younger, these units have experienced less tectonic deformation than UM, possibly explaining their better discontinuity conditions.

The lithology, to a certain degree, influences the discontinuity conditions, such that the coarser-grained rocks produce rough discontinuity surfaces when fractured, and the finer-grained rocks produce smooth discontinuity surfaces. LS and CC, being coarser grained, have surfaces that are more rough than FC. UM, while crystalline, generally has smooth serpentized surfaces, as a result of weathering at the surface. For the LS unit, fractures may be filled by secondary carbonate minerals from dissolution

and recrystallization processes within the the rock mass, which improves discontinuity strengths.

5.4.2.2 Anisotropic failures

Regional geologic structures provide conditions for potential planar slope failures. Kinematic analysis and slope stability demonstrate how the orientation of these structures affect their potential to cause instabilities. The orientations of these structures are ultimately determined by the stratigraphy and tectonic history of the area. Deposition of sedimentary rocks generally resulted to the formation of bedding planes, which were tilted by later deformation events. Tectonic regimes, such as that of the Philippine Fault Zone, impose their structural signatures on the existing rock materials.

While the strengths of the discontinuities were not explored in detail in this study, the type of discontinuity affects the strength. Faults are inherently weak because of the displacements which significantly reduced the strength along the discontinuities. Further displacements tend to smoothen the discontinuities and further reduce the friction angle along the surface. In contrast, bedding planes have not necessarily undergone displacements, and thus would have preserved their original strengths. With this line of reasoning, the modeled probabilities of failure could be much higher for the Riedel shears and thrust faults if actual discontinuity strengths had been factored in the analysis; conversely, probabilities of failure could be much lower for the bedding planes.

5.5 Conclusion

This study explains the empirical relationship between geology and topography in terms of physically-based, and quantifiable concepts such as rock mass strength and slope stability. Weak rock masses, either due to weak intact rocks or poor structure conditions, exhibit low-gradient, low-curvature topography as a result of dominant landsliding processes. This is the case for FC and UM. Strong rock masses, depending on the orientation of large scale structures, may either have channel-dominated processes

Unit	Geologic property influencing strength	Material strength	Transport Process	Topography
CCw	Good structures due to few deformations; gently dipping, daylighting, bedding planes	Strong isotropic rock mass	Mostly channelized transport	High gradient, high curvature
CCe	Good structures due to few deformations; gently dipping, non-daylighting bedding planes; steeply-dipping, daylighting fault planes	Strong isotropic rock mass, with steep potential failure planes	Channelized transport, with episodic massive landslides	High gradient, moderate curvature
FC	Weak lithology	Weak isotropic rock mass	Mostly landslides (isotropic failures)	Low-gradient, low-curvature
LS	Strong lithology; good structures	Strong isotropic rock mass	Limited role of landslides and channelized transport	Low-gradient, low-curvature
UM	Poor structures due to numerous deformations	Weak isotropic rock mass	Mostly landslides (isotropic failures)	low-gradient, low-curvature

Table 32. Geologic conditions linked to topography by rock mass strength and surface processes.

creating high-gradient, high-curvature terrain such as in CCw; or episodic massive landslides creating high-gradient, moderate curvature terrain such as in CCe. When relief development is limited and a rock mass is strong, such as in the case of LS, the topography may be similar as the topography of weak rock masses. The above relationships are summarized in Table 32. Landslide inventory analyses provide indicative validation to the inferred process rates.

The study demonstrated the use of gradient distributions to infer the relative rock mass strengths of areas underlain by different geologic materials. Differences in the gradient distribution indicated differences in rock mass strengths, which were verified by field-based strength measurements and modeled stability conditions. While

the results show generally similar trends for the gradient distributions and rock mass strengths, a deviation from this pattern was also highlighted in the case of the LS unit. This deviation points to a general case wherein the statistical representation of topography may not be reflective of the rock mass condition of the underlying rock unit. The use of gradient distributions to interpret relative rock mass strength must therefore be complemented by field-based characterization of topography, thorough analysis of topographic maps, or at the least, by some general prior knowledge of topographic expressions of various geologic materials. Furthermore, the interpretation of geomorphic processes from topography must be checked with field-based characterization of rock mass strengths.

While the study did not focus on prescribing a new landslide susceptibility classification, some remarks on this matter can be made based on the results. Results are consistent with the generally accepted concept of steeper slopes being more unstable and more susceptible to landslides for a constant set of material conditions. This has been demonstrated by the probability of failures for both the isotropic and anisotropic cases. However, a comparison of the high-gradient CC with the low-gradient FC, shows greater probability of failure for the latter which is contrary to the “steeper,-thus-more-unstable-slopes” concept. This highlights the need for caution in applying uniform gradient-based ratings in landslide susceptibility analysis in an area with highly variable rock mass conditions. An alternative approach would be to subdivide the area based on the geomechanical properties of the underlying units, and use a specific gradient classification for each of the subareas.

The role of rock mass strength in the frequency-magnitude relations of landslide processes has been discussed. Low isotropic rock mass strengths tend to restrict the occurrence of anisotropic failures, but facilitate frequent and small slope failures. In contrast, high isotropic rock mass strengths with steeply-dipping structures acting as potential sliding planes, facilitate large but infrequent slope failures. Both cases present different landslide frequency-magnitude scenarios which must be factored in landslide

risk analysis. The existence of both cases in the study area highlights the need to further qualify the concept of landslide hazards as expressed in geohazard information provided to communities and decision-makers.

At the core of this study is the division of the area of concern into distinct geologic units. Topographic, rock mass strength, and slope stability analyses were conducted on the basis of this geology-based categorization. This highlights the significance of geologic information in the conduct of studies such as this, and affirms the indispensable role of basic geologic knowledge in the practice of engineering geology and geomorphology.

The geologic environment - lithology, structures, stratigraphy, tectonic history - provides the material base on which surface processes act on to create the present landform. Rock mass strength, as a geomechanical manifestation of the geologic environment, provided a good framework for understanding and explaining the spatial variability of topographic characteristics in relation to geologic materials in the Can-abag mountain range in Southern Leyte, Philippines.

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