SCHOOL OF MATERIALS AND MINERAL RESOURCES

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SLOPE STABILITY ANALYSIS UNDER RAPID DRAWDOWN CONDITIONS AT SUSU DAM WITH FINITE ELEMENT METHOD USING RS2

BY

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DECLARATION

I hereby declare that I have conducted, completed the research work and written the dissertation entitled “Slope Stability Analysis Under Rapid Drawdown Conditions at Susu Dam With Finite Element Method Using RS2”. I also declare that it has not been previously submitted for the award of any degree or diploma or other similar title for any examining body or University.

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Date: 21TH JUNE 2017

SLOPE STABILITY ANALYSIS UNDER RAPID DRAWDOWN CONDITIONS WITH FINITE ELEMENT METHOD USING RS2

ABSTRACT

This research focused on the natural slopes stability analysis at Susu Dam with Finite Element Method (FEM) computer program RS2. Slope stability is important in the design and construction of earth dam because exposed to dangerous conditions such as rapid drawdown which is one of the main critical factor that contributes to slope failure. The rapid drawdown condition arises when submerged slopes experience rapid reduction of the external water level. The slope stability analysis using RS2 to calculate the factor of safety of the slopes and to define the potential slip surface under steady state and rapid drawdown for the selected sections (section A1-A2 and B1-B2) with time. The minimum required factor of safety for the slopes is 1.5 for the steady state condition and 1.0 for rapid drawdown condition. The study showed the values of the safety factor decrease from steady state to rapid drawdown condition. It also showed that the higher the rate of drawdown, the larger the area affected for potential slip surface along the geometry of weaker layer.

ANALISA KESTABILAN CERUN DIBAWAH KEADAAN PENURUNAN PARAS AIR YANG PANTAS DENGAN KAEDAH FINITE ELEMENT MENGGUNAKAN RS2

ABSTRACT

Penyelidikan ini menumpukan kepada Analisa kestabilan cerun di Empangan Susu degan menggunakan kaedah Finite Element dengan program computer yang dikenali sebagai RS2. Kestabilan cerun adalah sangan penting dalam reka bentuk dan pembinaan empangan bumi kerana terdedah kepada keadaan berbahaya seperti penurunan paras air yang pantas yang merupakan salah satu faktor utama yang menyumbang kepada kegagalan cerun. Penurunan paras air yang pantas berlaku apabila cerun yang berair mengalami penurunan paras air yang pantas. Analisa kestabilan cerun menggunakan RS2 untuk mengira faktor keselamatan cerun dan menentukan permukaan slip yang berpotensi di bawah keadaan stabil dan penurunan paras air yang pantas untuk keratan rentas yang dipilih (keratan rentas A1-A2 dan B1-B2) dengan masa. Faktor keselamatan yang minima untuk cerun ialah 1.5 untuk keadaan yang stabil dan 1.0 untuk keaadaan penurunan paras air yang pantas. Kajian ini menunjukkan nilai-nilai penurunan faktor keselamatan dari keaadan stabil kepada keadaan penurunan paras air yang pantas. Ia juga menunjukkan bahawa semakin pantas penurunan paras air, semakin besar kawasan yang terjejas permukaan slip yang berpotensi pada lapisan yang lebih lemah.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Research background

Evaluating the stability of slopes in soil is an important, interesting, and challenging aspect of engineering. Slope instability is a geo-dynamic process that naturally shapes up the geo-morphology of the earth. However, they are a major concern when those unstable slopes would have an effect on the safety of people and property. Concerns with slope stability have driven some of the most important advances in our understanding of the complex behavior of soils. Extensive engineering and research studies performed over the past 70 years provide a sound set of soil mechanical principles with which to attack practical problems of slope stability.

Over the past decades, experience with the behavior of slopes, and often with their failure, has led to development of improved understanding of the changes in soil properties that can occur over time, recognition of the requirements and the limitations of laboratory and in situ testing for evaluating soil strengths, development of new and more effective types of instrumentation to observe the behavior of slopes, improved understanding of the principles of soil mechanics that connect soil behavior to slope stability, and improved analytical procedures augmented by extensive examination of the mechanics of slope stability analyses, detailed comparisons with field behavior, and use of computers to perform thorough analyses. Through these advances, the art of slope stability evaluation has entered a more mature phase, where experience and judgment, which continue to be of prime importance, have been combined with improved understanding and rational methods to improve the level of confidence that is achievable through systematic observation, testing, and analysis. This thesis is focused on the slope stability analysis at Hulu Jelai Hydroelectric project, Susu Dam using Rocscience software called RS^2.

1.2 Study Area

This study is focused on The Ulu Jelai Hydroelectric Project, Susu Dam. The Ulu Jelai Hydro Electric Project (UJHEP) is located in the State of Pahang, in the district of the Cameron Highlands about 140 km north of Kuala Lumpur and 80 km east of west coast of mainland Malaysia. The main features of the Project comprise Susu Dam, an 85m high RCC dam on Sg. Bertam, two diversion weirs on Sg. Lemoi and Sg. Telom for the diversion of flows from adjacent catchments via 7.3 km and 8 km long transfer tunnels into Sg. Bertam, a 4 km main headrace tunnel, a 372 MW Underground Power Station and the required associated water conveyance and access road systems. The hydroelectric development will generate peaking energy to the national grid. The detailed design and construction supervision of this project is being undertaken by SMEC International in association with SMEC Malaysia for the project owner Tenaga Nasional Berhad (TNB). Construction of the Project has commenced under an agreement established with TINDAKAN MEWAH SDH. BHD. (741628-U) AND SALINI COSTRUTTORI CONSORTIUM. Detailed site investigation comprised of boreholes, drill holes, test pits, field and laboratory test were carried out in the study area.

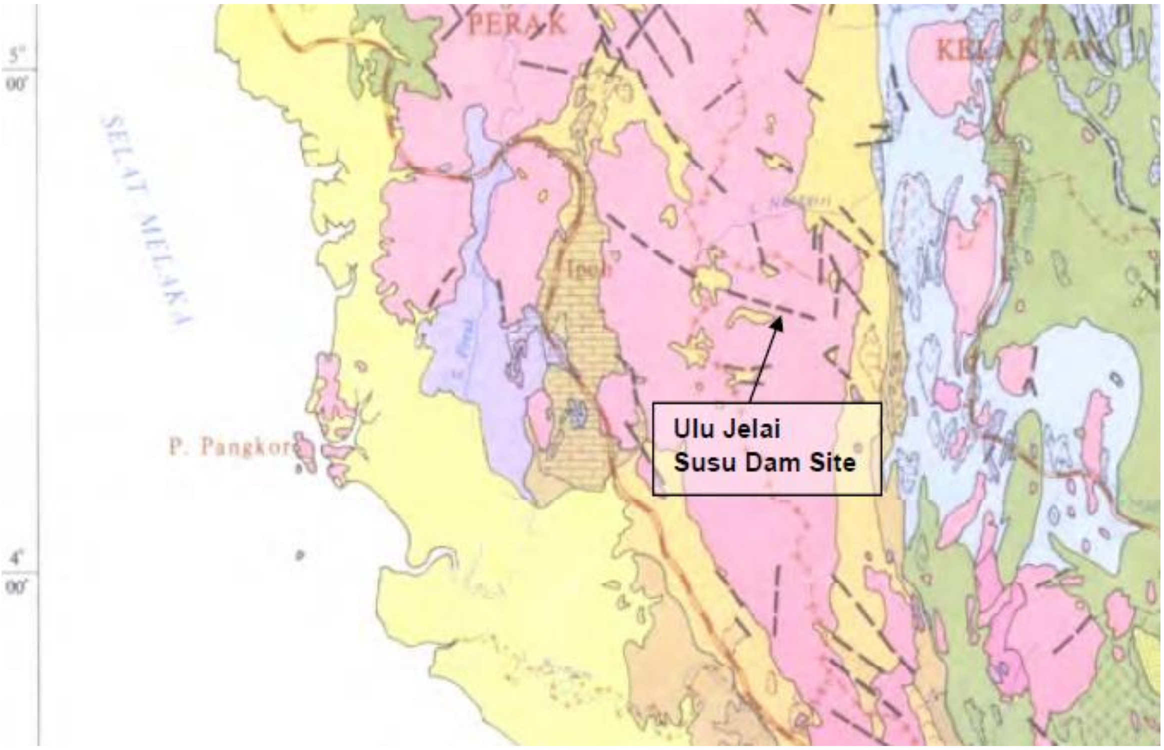

Figure 1.1: Location of Susu Dam Site.

Figure 1.2: View of Susu Dam Site

Figure 1.3: Top View of Susu Dam Site

1.3 Research Approach

The geological study on the properties of material at the project site is collected and analyzed by SMEC Malaysia Sdn. Bhd and was published in the report 'Geological Investigation of the Susu Dam Reservoir Rim Stability on the JKR Highway'. The project site contains a total of 16 slopes. This study focused on 2 most critical sections which are section A1-A2 and B1-B2 to enable in depth study of comparison of safety factor between the two sections. The method used for slope stability analysis is shear strength reduction finite element method (SSRFEM) using Rocscience software called RS2. The modelling of the slopes is based on the actual properties obtained from the geological study. Simulation of steady state and drawdown analysis is analyzed after the completion of the slope modelling to obtain the safety factors of each slope.

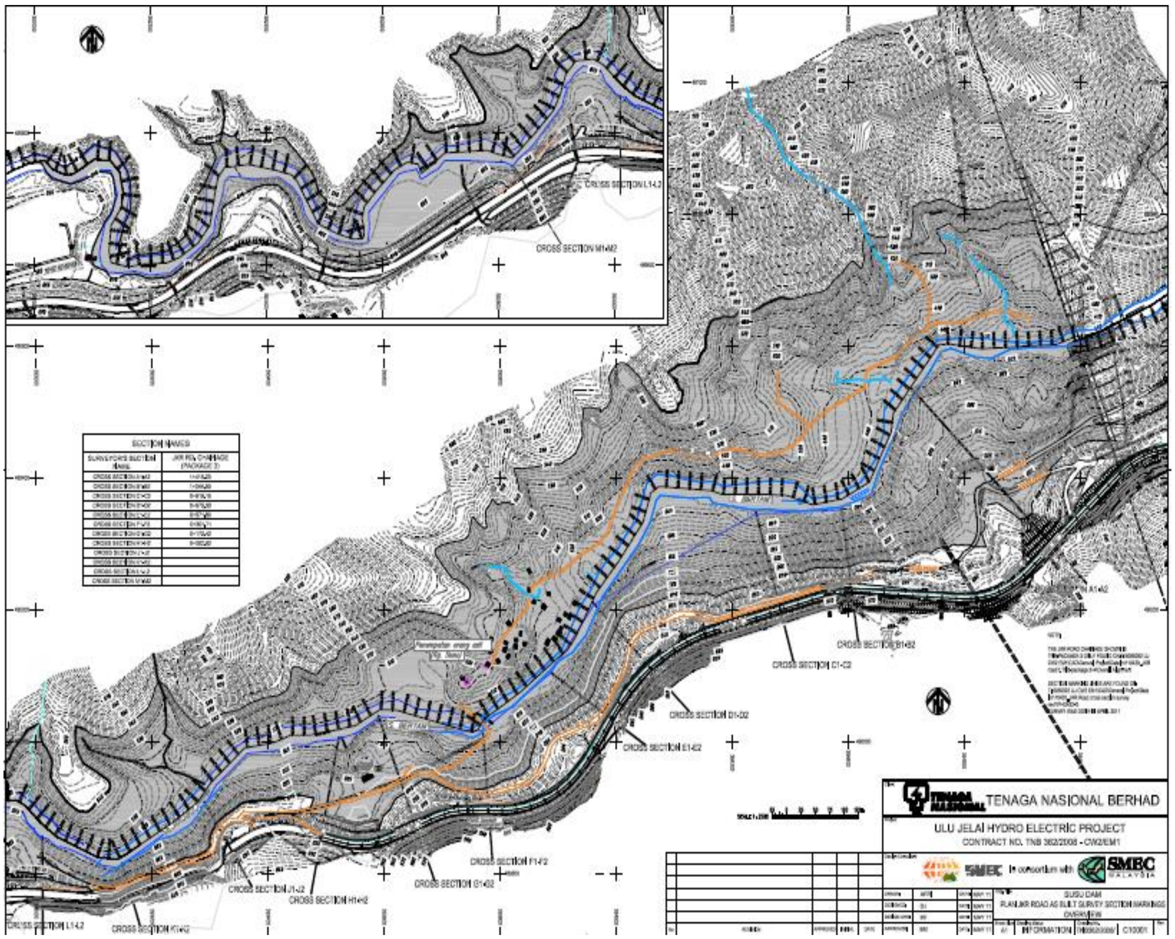

Figure 1.4: 16 Geological Cross Sections at Susu Dam

1.4 Problem Statement

If the change in external water level happens without allowing the time needed for the drainage of the slope soils, it is called sudden or rapid drawdown (RDD). Due to rapid drawdown, there will be a decrease in the slope stability, which may lead to slope failures. In the past many similar failures have been observed in natural and constructed slopes. Examples of such failures include the Pilarcitos Dam south of San Fransisco, Walter Boudin Dam in Alabama, and a number of river bank slopes along the Rio Montaro in Peru (Duncan et al. 1990) and other places. It is important to study and understand the stability of the slopes near reservoirs, rivers, lakes and seas where RDD occurs to secure the safety of people and critical infrastructure in the surrounding areas. Advanced solutions of this challenging problem will result in safe and economic treatment of problem areas that are under RDD related risks. Hence, the study focused on Hulu Jelai Hydroelectric Susu Dam to simulate and analyze the risk that RDD may cause based on the minimum factor of safety guidelines by Government of Malaysia Department of Irrigation and Drainage. There is an extensive work done on the geological study at the project site by SMEC Malaysia Sdn. Bhd. but existing organization has yet to study the safety factor under RDD occurrence of the slopes at the project site. RDD failure occurs when the pool level in the reservoir is lowered, removing the stabilizing hydrostatic pressure along the slope while simultaneously decreasing stresses on the upstream slope, thereby reducing stability of the upstream slope that can potentially cause instability.

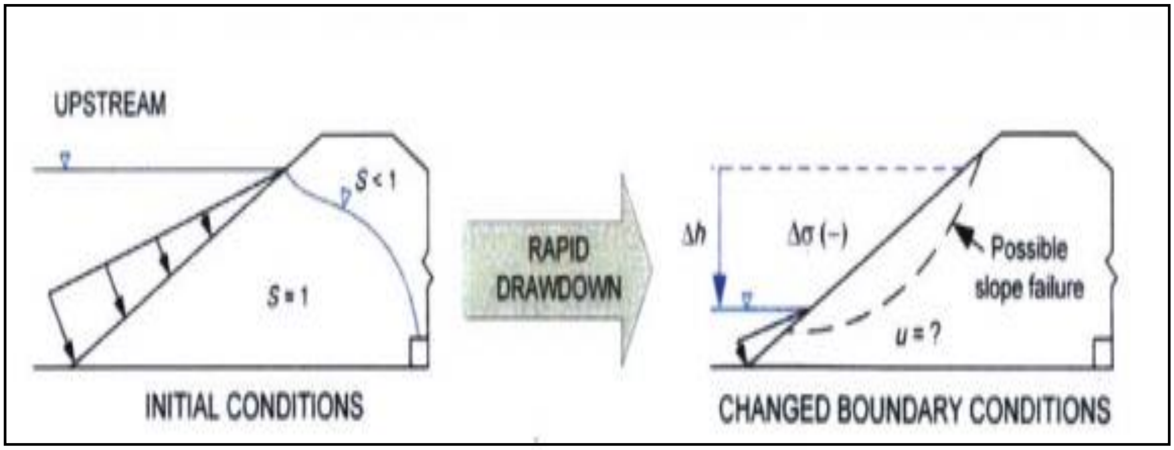

Figure 1.5: Rapid Drawdown failure mechanisms

1.5 Objectives

The objectives of this study are:

I. To study the properties of the rock and soils at Susu Dam.

II. To simulate the rapid drawdown factor of safety analysis using RS2.

III. To compare the result of factor of safety obtained using RS2.

1.6 Thesis Outline

The thesis is organized in five chapters. Chapter 1 would describe a brief background of study together with the problem statement and objectives of the research. In the following Chapter 2, a comprehensive review on the previous site geotechnical investigation of the Ulu Jelai Hydroelectric project and previous researches conducted to investigate the rapid drawdown slope analysis on other dam slopes. Chapter 2 will also detail the Finite Element Analysis, Limit Equilibrium Analysis that is involved in computing the factor of safety of the slopes studied. Chapter 3 details all the methodologies in this research study including fieldwork, modelling of the geological sections and determination of factor of safety using RS2. Chapter 4 presents the experimental results and comprehensive discussions on the factor of safety obtained for steady state analysis and rapid drawdown analysis with different drawdown rate from 0.5 m/day to 2.0 m/day. Chapter 5 will conclude the research works and recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter is devoted to describe the geological condition of the site, Finite element analysis and rapid drawdown analysis of similar studies. This attempt is to have an understanding of response of numerical simulation and learn the mathematical principles of finite element analysis and limit equilibrium method behind RS2 software.

2.2 Site Description

2.2.1 Topography

The Ulu Jelai Hydroelectric Project lies within the Cameron Highlands and extends from the Pahang/Perak State borders in the west to the Telom-Bertam-Lemoi river confluences in the east. This area is wholly contained within the Main Range of Peninsular Malaysia. This a long range of hills and low mountains extending the length of the country. The highest point of the Main Range in the Cameron Highlands is Gunung Irau at 2110 m elevation, just 2 km NW of Gunung Berincang at EL 2031m, where the hilly terrain is up to 70 km across from west to east. The highest point within the project area is Bukit Bujang at 1772 m, located between the Bertam and Lemoi River valleys. The area is characterised by deep major river systems, incised over 1000 m below the crest of the ranges. The most elevated areas are older erosion surfaces formed after the Main Range was thrust upwards. The river courses are controlled by geological structures such as faults and joints. Starting at the top of the Cameron Highlands, the river descends from the flanks of G Berincang and flows south through Berincang town to Ringlet where it has been dammed as the Sultan Abu Bakar Reservoir, at EL 1070 m. Below this level the stream strikes more easterly descending at a gentle gradient for the next 7 km through cultivated land, then steeply through a 250m change in elevation for the next 5 km before the large tributary the Sg Mansun comes in from the left. The lower reach of the Sg Bertam, over about 9 km, follows a relatively straight ENE course with steep valley sides of 30-45 degrees. Total relief is 600-800 m with the lower valley sections (up to 500-600 m altitude) covered by secondary forest and remnants of plantations. The junction with the larger Sg Telom occurs about 6 km downstream of the proposed dam site.

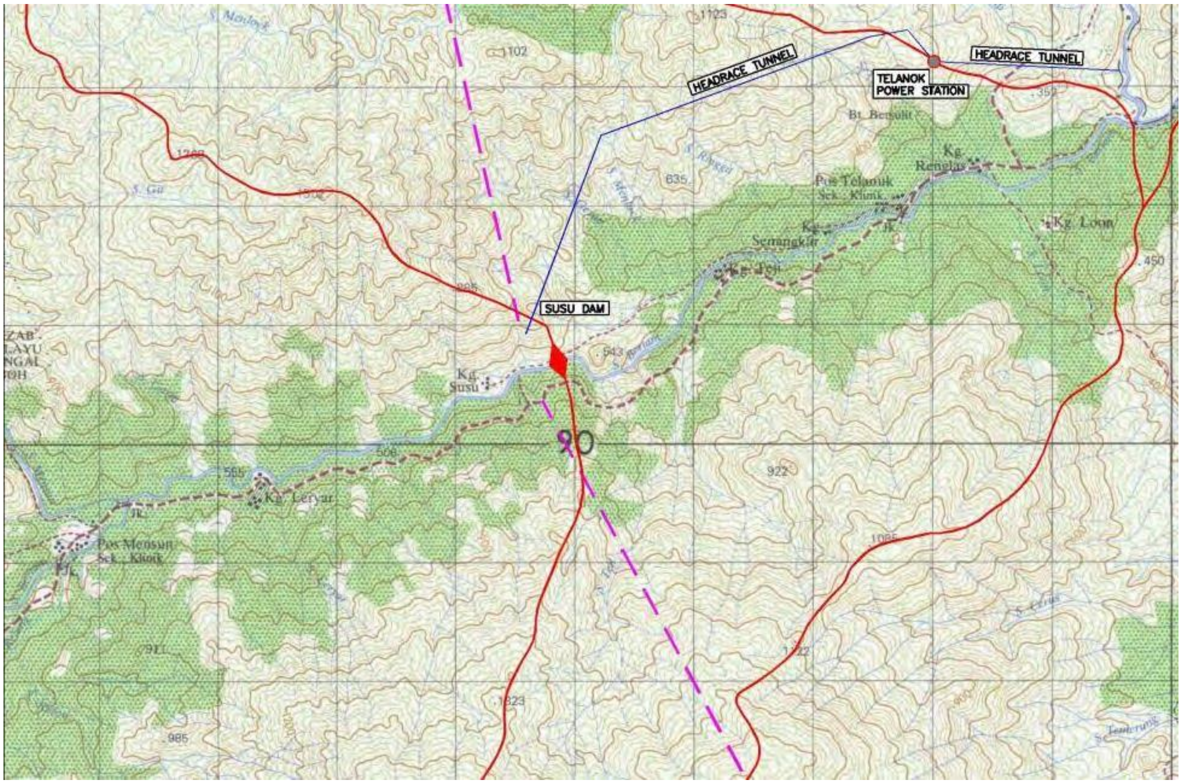

Figure 2.1: Topography of project site showing position of Susu Dam. Extract from Pos Lemoi 1:50,000 topographic map.

2.2.2 Main Range Granite

Feasibility study done by Tenaga Nasional Berhad and Tokyo Electric Power Co. Inc (2004), indicates that the biotite granite is generally medium to coarse grained leucocratic (light coloured) rock varying from equigranular to porphyritic, the latter including feldspar phenocrysts up to 50mm in length. The main mineral constituents based on petrographic examinations of thin sections are quartz, alkali feldspar, plagioclase feldspar and biotite mica. Accessory minerals include muscovite mica, apatite, zircon and corundum. The average composition by volume is 32-35% alkali feldspar, 26-28% plagioclase feldspar, 32-35% quartz and 5-8% biotite with minor accessory minerals. The large feldspar phenocrysts are often twinned pink microcline giving the rock a pinkish appearance in outcrop. The granite has also been subject to hydrothermal alteration as secondary minerals such as sericite and chlorite are seen to have formed from the feldspars and biotite. This alteration results in a more bleached appearance and reduced strength. Commonly the rock is sheared along joints. In the vicinity of these sheared zones the rock is often foliated, as expressed by parallel orientation of the feldspar crystals. With greater degrees of shearing an intermediate type of rock, such augen gneiss is formed. In extreme shearing condition the rock is a mylonite (likely in fault zones). Associated with the granite but is less frequent occurrence are pegmatite veins and aplite dykes. Both of these are observed in drill core.

In some of the earlier site investigations in the Cameron Highlands it was observed that the drill core tends to swell and deteriorate after exposure in the air. This disintegration is likely to be chemical in origin (Newbery, 1988) and is likely to be associated with the presence of the zeolite mineral, laumontite, which, on exposure to air, loses up to one third of its water and changes to a secondary mineral leonhardite. It is important to note the comments by Professor John Knill from Imperial College who wrote in the 1988 investigation report (WLPU, Annex B-Appendix) that the critical issues with the properties of the granite will be related to the swelling as the laumontite breaks downs and this phenomenon may also be related to weathering of hydrothermally altered minerals to montmorillonite. This latter mineral is well known as having a high swelling potential due to mobility of water and cations in its clay lattice structure. Additionally Prof Knill notes that there are clear indications of strain effects in the granite, and the existence of shear effects associated with the major faults. The strain and shearing has resulted in brecciation. The brecciation so formed has since been silicified by hydrothermal activity to form rocks of equal competence or better than the antecedent. The hydrothermal action has also led to voids and fractures, produced as a result of cataclasis, being infilled with vein material, generally calcite associated with zeolite minerals (refer to the swelling effect of laumontite above). It is important to note however that the 25 petrographic examinations done during the earlier investigations for the Ulu Jelai project did not indicate any zeolites, the only accessory minerals listed were opaques (oxide or sulphide minerals), zircon, sphene and carbonate). However, the occurrences of hydrothermal alteration are extensive and carbonate minerals are also evident.

2.2.3 Geological Structures

The general structure around the project area has been discerned from the regional geological and tectonic map and from a review of satellite imagery combined with a study of the topographic maps at 1:50,000 scale and shaded relief map (SMEC, 2008). Essentially the area is affected by cross cutting faults between the main N-S regional tectonic trends. The main rivers follow the structural trends of these faults:

- Bertam River: along a fault striking 065-070 degrees;
- Telom River: along a fault striking 110 degrees in upper reaches;
- Along a fault striking 160-165 degrees in a step like pattern in lower reach;
- Other common trends for faults or lineaments are 045-225 and 120-300 degrees.

Based on the earlier studies for the Ulu Jelai project (SMEC, 2005) there is an inferred pattern of conjugate faults and associated joints trending in the quadrants NW-SE and NE-SW between the more N-S regional boundary faults. The faults are expected to be mostly tensional or normal displacement, but some strike slip movements may have occurred for local adjustments to terrains between larger faults. It does not appear that there are large thrust faults through the granite. The granite structure is characterised by blocky jointing that is quite variable both in orientation and spacing, and the SMEC (2008) study assessed the joint main structures are:

- Strike 010 – parallel to regional trends;
- Strike 060 – conjugate set;
- Strike 110 – conjugate set;
- Strike 175 – parallel to regional set.

There are also stress relief joints that have a shallow dip, either as general sheet joints at the tops of the granite batholith structures or as valley side joints where valley deepening de-stresses the rock in the valley sides.

2.2.4 Weathering

As described above the weathering is deep and intense due to the tropical chemical weathering processes. This process may have been assisted by the presence of many micro-fractures (cataclasis) in the granitic rocks, allowing a higher microporosity than other rocks and the deep permeation by water. Locally the weathering as encountered in the drillholes, reaches depths of 50-80 m, but regionally it may be several hundred metres deep. The breakdown of biotite and feldspar is the main process of the chemical decay. The biotite breaks down first to limonite which gives the rock a brown discolouration. Kaolinization of the feldspar then follows with subsequent reduction in strength and the formation eventually of residual soil. The weathering penetrates faster along the joint planes and so a network of weathering forms with relict cores of fresh rock commonly surrounded by highly weathered rock, referred to in the logging as “core stones”. The weathering intensities become weaker with depth. Based on a review of published international literature, local practice and the Project Specification, the subsoil profile in the granite formation can be divided into different weathering grades as shown in Table 2.1 following.

Table 2.1: Weathering Classes

Weathering classification	Grade	Description
Residual Soils (RS)	VI	· All rock material is converted to soil. · The mass structure and material fabric are destroyed. · The material has not been transported.
Completely Weathered (CW)	V	· All rock material is decomposed and/or disintegrated to soil. · The original mass structure is still largely intact. · Rock which retains most of the original rock texture (fabric) but the bond between its mineral constituents is weakened by chemical weathering to the extent that the rock will disintegrate when immersed and gently shaken in water. · In engineering usage, this is a soil.
Highly Weathered (HW)	IV	· More than 50 % of the rock material is decomposed and /or disintegrated to soil. · Fresh or discoloured rock is present either as a discontinuous framework or core stones. · Rock which is weakened by chemical weathering to the extent that dry piece about the size of 50 mm diameter drill core can be broken by hand across the rock fabric. · Does not readily disintegrate when immersed in water
Moderately Weathered (MW)	III	· Less than 50 % of the rock material is decomposed and/or disintegrated to soil. · Fresh or discoloured rock is present either as a discontinuous framework or core stones. · Rock which exhibits considerable evidence of chemical weathering such as discolouration and loss of strength but

		which has sufficient remaining strength to prevent dry pieces about the size of 50 mm diameter drill core (or inherently hard rock) being broken by hand across the rock fabric. · Does not ring when struck with a hammer.
Slightly Weathered (SW)	II	· Discoloration indicates weathering of rock material and discontinuity surfaces. · All the material may be discoloured by weathering and may be somewhat weaker than in its fresh condition. · Rock which exhibits some evidence of chemical weathering, such as discolouration, but which has suffered little reduction in strength. · Except for some inherently soft rocks, slightly weathered rock rings when struck with a hammer.
Fresh with Limonite Stained Joints (Fr St)	Ib	· Some discoloration on major discontinuity surfaces. · The joint faces are coated or stained with limonite but the blocks between joints are unweathered.
Fresh Rock (Fr)	Ia	· No visible sign of rock material weathering. · Rock which exhibits no evidence of chemical weathering. · The joint faces may be clean or coated with clay, calcite, chlorite or other minerals.

At this site, it was noted that the weathering profile was quite sharp with completely weathered rock grading to slightly weathered rock over a few metres. The completely weathered rock is divided into two units:

- CW 1: completely weathered rock with no or few core stones;
- CW 2: completely weathered rock with significant proportion of core stones.

This is consistent with other published systems where weathering intensity of the rock mass can be assessed by reference to the proportion of core stones (the relict unweathered rock in the mass –often as a sub-rounded boulder appearance). A system proposed by TEPCO (2004) used the following:

- Weathering Intensity A: from intact fresh rock to 2/3 core stones;

- Weathering Intensity B: from 2/3 to 1/3 core stones;
- Weathering intensity C: from 1/3 core stones to none (mass is completely weathered or soil).

The calluses used in this report would generally lie in the two profiles shown as less than one third core-stones (CW1) and more than one third core-stones (CW2). Similar a weathering profile as that encountered at the Ulu Jelai Project Site is depicted in Figure 2.2.

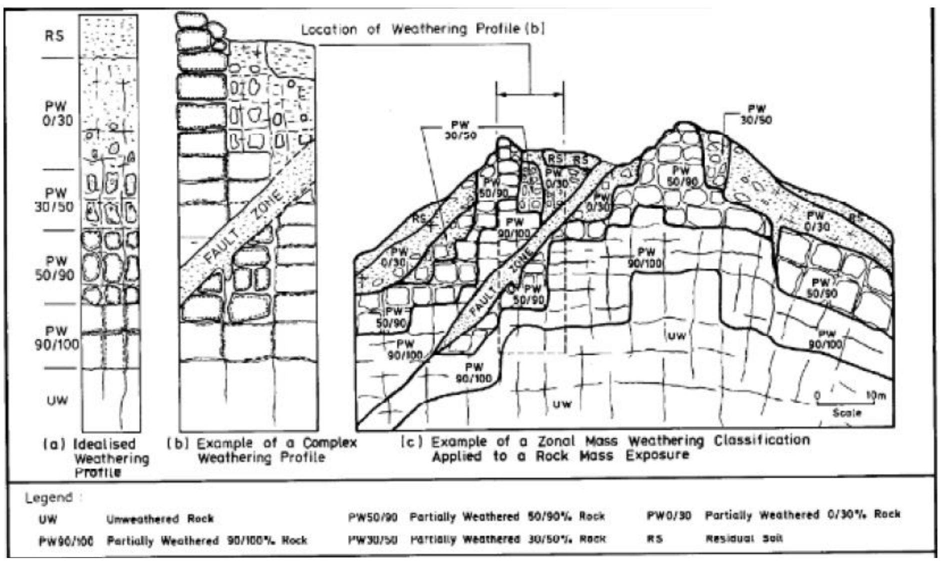

Figure 2.2: Mass weathering profiles in granitic materials (SMEC International 2014)

2.2.5 Hydrogeology

The groundwater regime is linked to the terrain and the extent of permeability in the weathered and fresh rock. It is expected that barriers to the flow of groundwater may be the result of faulting, joint intensity, weathering and lithological contacts. It is noted that the rivers are likely partially fed by emerging groundwater at the banks and so the groundwater table is expected to bottom out in the main flowing streams. In more elevated areas the groundwater is likely to occur at depths of 30-50 m. The infiltration rate from the surface is affected by the soil cover; if the residual soil is clayey and un-fissured the infiltration will be minimal and run-off more prevalent. There will be some seasonal fluctuations but the climate is equatorial with two prominent wet seasons and so there is expected to be year around infiltration to recharge the groundwater. Where brecciation is associated with the faulting there may be deep penetration of the groundwater to several hundred metres and this may produce high hydraulic gradients between the faults and competent surrounding rock.

2.2.6 In Situ Stress

Based on the study done by Hutchison, Charles S (2007), the region has low tectonic stresses. The most recent activity was likely the faulting that uplifted the Main Range during the Paleocene age (> 50 million years ago), since then there has been a long period of deep weathering and erosion. Stress relief along valleys is locally effective in producing shallow dipping joints. Thus, there is likely no prevailing maximum horizontal stress in this area, but a locked in stress is likely where regional faulting has occurred. The regional faulting around the Cameron Highlands is generally NS, to NW-SE along the length of Peninsular Malaysia. There may also be cross-cutting structures such as along the Telom River (strike WNW-ESE) that locally influence the stress regime.

2.2.7 Seismicity

Peninsular Malaysia lies in the western portion of the seismically stable Sunda Shelf, which also includes East Malaysia, Kalimantan, much of Indo-China and the South China Sea. Seismic activity, except for some very small events, is confined to the margins of this stable continental plate. The eastern margin lies some 1400 km from the project area and needs no further consideration. The western and southern boundaries of the Sunda Shelf Plate are marked by two vigorously active asymmetric arcuate systems, the Burmese and Indonesian Arcs. The seismic activity in the Indonesian Arc is generally confined to a northward dipping Benioff Zone. This Benioff Zone lies between the northward thrusting Australian Plate and the Asian Plate. The Sumatran section of the Indonesian Arc lies closest to the project area but apart from low felt intensities from major events such as in 2004 to 2006, there are no direct impacts to be expected on the project. A report on the earthquake hazard for Ulu Jelai was prepared by the Seismology Research Centre (2008). The estimated peak ground acceleration for this site is 0.027 g based on a 475 year return period (10% in 50 years).

2.3 Limit Equilibrium Methods

Limit Equilibrium Methods Limit equilibrium methods are the most commonly used approaches in slope stability analysis. The fundamental assumption in these methods is that failure occurs through sliding of a mass along a slip surface. The reputation of the limit equilibrium methods is principally due to their relative simplicity, the ability to evaluate the sensitivity of stability to various input parameters, and the experience geotechnical engineer have acquired over the years in calculating the factor of safety. The assumptions in the limit equilibrium methods are that the failing soil mass can be divided into slices and that forces act between the slices whereas different assumptions are made with respect to these forces in different methods. Some common features and limitation for equilibrium methods in slope stability analysis are summarized in Table 2. All methods use the same definition of the factor of safety: Shear stress required for equilibrium Shear strength of soil FOS = (1) The factor of safety is the factor by which the shear strength of the soil would have to be divided to carry the slope into a state of barely stable equilibrium.

The findings related to the accuracy of the limit equilibrium methods can be reviewed as follows: 1) For effective stress analysis of flat slopes, the ordinary method of slices is highly inaccurate. The computed factor of safety is too low. This method is accurate for $\varphi = 0$ analysis, and fairly accurate for any type of total stress analysis using circular slip surfaces. 2) For most conditions, the Bishop's modified method is reasonably accurate. Because of numerical problems, sometimes encountered, the computed factor of safety using the Bishop's modified method is different from the factor of safety for the same circle calculated using the ordinary method of slices. 3) Computed factor of safety using force equilibrium methods are sensitive to the assumption of the inclination of side forces between slices. A bad assumption concerning side force inclination will result in an inaccurate factor of safety. 4) Janbu's, Morgenstern and Prices's and Spencer's method that satisfy all conditions of equilibrium are accurate for any conditions. All of these methods have numerical problems under some conditions.

Table 2.2: Features and Limitation for Traditional Equilibrium Methods in Slope Stability Analysis (Duncan and Wright, 1980)

Method	Features and Limitation
Slope Stability Charts (Janbu, 1968, Duncan et al, 1987)	- Accurate enough for many purposes. - Faster than detailed computer analysis.
Ordinary Method of Slices (Fellenius, 1927)	- Only for circular slip surfaces. - Satisfies moment equilibrium. - Does not satisfy horizontal or vertical force equilibrium.

Bishop’s Modified Method (Bishop, 1955)	- Only for circular slip surfaces. - Satisfies moment equilibrium. - Satisfies vertical force equilibrium. - Does not satisfy horizontal force equilibrium.
Force Equilibrium Methods (e.g. Lowe and Karafiath, 1960, Army Corps of Engineers, 1970)	- Any shape of slip surfaces. - Does not satisfy moment equilibrium. - Satisfies both vertical and horizontal force equilibrium.
Janbu’s Generalized Procedure of Slices (Janbu, 1968)	- Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Permit side force locations to be varied. - More frequent numerical problems than some other methods.
Morgenstern and Price’s Method (Morgenstern and Price, 1965)	- Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Permit side force orientations to be varied.
Spencer’s Method (Spencer, 1967)	- Any shape of slip surfaces. - Satisfies all conditions of equilibrium. - Side forces are assumed to be parallel.

The limitation of limit equilibrium method in slope stability analysis has been demonstrated by Krahn (2003). This limitation is caused by the absence of a stress strain relationship in the method of analysis. The limit equilibrium method lacks a suitable procedure for slope stability analysis under rapid loading condition as illustrated by Baker et al. (1993).

2.4 Finite Element Method

In the finite element method, the latter analysis, the so-called shear strength reduction (SSR) technique (Matsui & San 1992, Dawson et al. 1999) can be applied. The angle of dilatancy, soil modulus or the solution domain size are not critical parameters in this technique (Cheng, 1997). The safety factor can be obtained, assuming a Mohr-Coulomb failure criterion, by reducing the strength parameters incrementally, starting from unfactored values $\phi_{available}$ and $c_{available}$, until no equilibrium can be found in the calculations. The corresponding strength parameters can be denoted as $\phi_{failure}$ and $c_{failure}$ and the safety factor η_{fe} is defined as:

$$\eta_{fe} = \frac{\tan \varphi_{available}}{\tan \varphi_{failure}} = \frac{c_{available}}{c_{failure}}$$

There are two possibilities to arrive at the factor of safety as defined above.

Method 1: An analysis is performed with unfactored parameters modelling all construction stages required. The results represent the behaviour for working load conditions at the defined construction steps. This analysis is followed by an automatic reduction of strength parameters of the soil until equilibrium can be no longer achieved in the calculation. The procedure can be invoked at any construction step. This approach is sometimes referred to as ϕ/c-reduction technique.

Method 2: The analysis is performed with factored parameters from the outset, i.e. strength values are reduced, again in increments, but a new analysis for all construction stages is performed for each set of parameters. If sufficiently small increments are used the factor of safety is again obtained from the calculation where equilibrium could not be achieved. Both methods are straightforward to apply when using a standard Mohr-Coulomb failure criterion. In the finite element method, failure occurs naturally through the zones within the soil mass wherein the shear strength of the soil is not capable to resist the applied shear stress, so there is no need to make assumption about the shape or location of the failure surface.

2.4.1 Advantages of The Finite Element Method

The advantages of a FE approach to slope stability analysis over traditional limit equilibrium methods can be summarized as follows:

(a) No assumption needs to be made in advance about the shape or location of the failure surface. Failure occurs `naturally' through the zones within the soil mass in which the soil shear strength is unable to sustain the applied shear stresses.

(b) Since there is no concept of slices in the FE approach, there is no need for assumptions about slice side forces. The FE method preserves global equilibrium until `failure' is reached.

(c) If realistic soil compressibility data are available, the FE solutions will give information about deformations at working stress levels.

(d) The FE method is able to monitor progressive failure up to and including overall shear failure.

2.4.2 Shear Strength Reduction Theory

The safety factor of the slope stability can be defined as soil shear strength reduced degree when the slope critical failure status is just reached, and it equals the ratio of the soil shear strength and the reduced soil shear strength of critical failure status. The shear strength reduction factor is defined as the ratio of the maximum shear strength of slope soil and the actual shear stress of the slope under external loads, remaining the same external loads circumstances. The shear strength reduction coefficient is defined as the overall stability factor of slope safety, hence this safety coefficient can be considered to the strength reserve safety coefficient. Strength reduction concept can unify the strength reserve safety coefficient and the strength safety coefficient of the slope overall stability. And the finite element method may be used to calculated shear strength reduction factor without determination of the shape and position of failure surface. In elastic-plastic finite element numerical analysis based on the concept of strength reduction, for a point in domain, according to the general definition of the Bishop safety coefficient and considering the shear strength, the Mohr-Coulomb failure criteria is expressed as:

$$\tau f = C + \sigma tan\phi$$

where, C is the cohesive force of the soil; ϕ is the internal friction angle. The safety factor of the appointed shear plane of this point is

$$F = \tau f / \tau = C + \sigma tan\phi / \tau$$

Assume that the shear failure of soil does not occur, the actual shear stress in soils and the maximum shear strength are same, that is

$$\tau = \tau f_m = C + \sigma tan\phi /F = C_m + \sigma tan\phi_m$$