



Faculty of Engineering

Performance Testing for Slope Instability due to Suffusion via Site-Specific Response Analysis with Incorporation of Electrical Resistivity Tomography

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**Doctor of Philosophy
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A thesis submitted

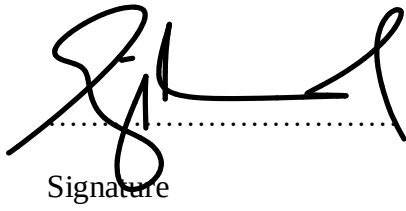
In fulfillment of the requirements for the degree of Doctor of Philosophy

(Geotechnical Earthquake Engineering)

Faculty of Engineering
UNIVERSITI MALAYSIA SARAWAK
2024

DECLARATION

I declare that the work in this thesis was carried out in accordance with the regulations of Universiti Malaysia Sarawak. Except where due acknowledgements have been made, the work is that of the author alone. The thesis has not been accepted for any degree and is not concurrently submitted in candidature of any other degree.

A handwritten signature in black ink, appearing to be 'Azrin Bin Ahmad', written over a horizontal dotted line.

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Date : 31 May 2024

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ABSTRACT

Soil instability had caused concerned in geotechnical engineering due to its consequences of failure and its effects to the structures above or surrounding it. The instability of soil is caused by either water, critical stress condition, or perilous void within the soil properties. The lack of proper consideration of critical void in soil or suffusion phenomenon causes soil defects and failures in cut-slopes that subsequently damaging to the economy and ecology. Many researches correlated internal soil instability with water or hydraulic conductivity. This research aims to investigate on internal soil instability caused by void limit state and migration of fines characterized by vibration specifically seismic soil amplification that resemble failure criterion caused by hydraulic load. Soil amplification obtained from sensible conversion of apparent resistivity (ohm.m) to shear wave velocity (m/s) captured by electrical resistivity tomography (ERT) experimental result at specific sites with cut slopes at Pan Borneo Highway, Sarawak, has been studied. Archie's law which determines porosity is used in this study, where correlation with experimental results have shown its reliability, proving that volume of void over total volume of soil (V_v/V_T) from soil phase diagram is attainable with ERT and it is equivalent to liquid limit of soil. Energy based analysis is used to actuate synthetic earthquake from underneath and simulates soil amplification, whether it exceeds design response spectra or within limit. Simplified amplitude limit of failure has been developed to alternate Fourier amplitude to propose ERT data collection and new approach to develop serviceability limit state (SLS) or design response limit. Data acquisition via ERT that sensibly converted to shear wave velocity and soil shear strength has been verified its compliance with the existing guideline for slope design from the public works department of Malaysia. Output of this research improved the reliability of design safety factor for any constructions by integrating advanced technology application. As a

result, ERT raw soil properties data that correlated well with soil liquid limit where volume of void over total void (V_v/V_T) is in equivalent and soil conductivity (σ) over soil resistivity (Ω) is consistent with liquidity index. This research also discovered that electrical conductivity against electrical resistivity, $1/\text{Sigma } (\Sigma^{-1}) \div \text{Ohm } (\Omega) = 1$ and is a threshold index of soil instability. It is also discovered that when the soil instability index higher than 1 the site-specific soil is unstable. Further research on the instability index is potential and beneficial to the geotechnical engineering industry for geo-hazard identification. In conclusion, this research is focused to give clear interpretation to ERT result and proved that it is useful for future diagnostic of soil stability and its optimum treatment required for future land use.

Keywords: Peak ground acceleration, soil amplification factor, soil instability, suffusion, design response spectra

Ujian Prestasi untuk Ketidakstabilan Cerun Akibat Suffusion melalui Analisis Tindak Balas Khusus Tapak dengan Penggabungan Tomografi Kerintangan Elektrik

ABSTRAK

Ketidakstabilan tanah telah menyebabkan kebimbangan dalam Kejuruteraan Geoteknik kerana akibat kegagalan dan kesannya kepada struktur di atas atau di sekelilingnya. Ketidakstabilan tanah disebabkan oleh sama ada air, keadaan tegasan kritikal, atau lompong berbahaya dalam sifat tanah. Kekurangan pertimbangan yang sewajarnya terhadap kekosongan kritikal dalam tanah atau fenomena suffusion menyebabkan kecacatan dan kegagalan tanah pada cerun-cerun yang kemudiannya merosakkan ekonomi dan ekologi. Banyak penyelidikan mengaitkan ketidakstabilan tanah dalaman dengan air atau kekonduksian hidraulik. Penyelidikan ini bertujuan untuk mengkaji ketidakstabilan dalaman tanah yang disebabkan oleh keadaan had lompong dan penghijrahan denda yang dicirikan oleh getaran khususnya penguatan tanah seismik yang menyerupai kriteria kegagalan yang disebabkan oleh beban hidraulik. Penguatan tanah yang diperolehi daripada penukaran sensitif kerintangan ketara (ohm.m) kepada halaju gelombang ricih (m/s) yang ditangkap melalui keputusan eksperimen tomograf kerintangan elektrik (ERT) di tapak tertentu dengan cerun potong di Lebuhraya Pan Borneo, Sarawak, akan dikaji. Hukum Archie yang menentukan keliangan dalam batuan digunakan dalam kajian ini, di mana korelasi dengan keputusan eksperimen akan menunjukkan kebolehpercayaannya, membuktikan bahawa isipadu lompong atas jumlah isipadu tanah (VV/VT) daripada rajah fasa tanah boleh dicapai dengan ERT dan ia adalah setara. kepada had cecair tanah. Analisis berasaskan tenaga digunakan untuk menggerakkan gempa bumi sintetik dari bawah dan mensimulasikan penguatan tanah, sama ada ia melebihi spektrum tindak balas reka bentuk atau dalam had. Had amplitud mudah kegagalan hendaklah dibangunkan untuk

menggantikan amplitud Fourier untuk mencadangkan pengumpulan data ERT dan pendekatan baharu untuk membangunkan keadaan had kebolegunaan (SLS) atau had tindak balas reka bentuk. Pemerolehan data melalui ERT yang secara wajar ditukar kepada halaju gelombang ricih dan kekuatan ricih tanah disahkan pematuhannya dengan garis panduan sedia ada untuk reka bentuk cerun daripada Jabatan Kerja Raya. Hasil penyelidikan ini diharapkan dapat meningkatkan kebolehpercayaan faktor keselamatan reka bentuk untuk sebarang pembinaan dengan mengintegrasikan aplikasi teknologi canggih. Hasilnya, data sifat tanah mentah ERT yang berkorelasi baik dengan had cecair tanah di mana isipadu lompong atas jumlah lompong (VV/VT) adalah setara dan kekonduksian tanah (σ) terhadap kerintangan tanah (Ω) adalah konsisten dengan indeks kecairan. Kesimpulannya, penyelidikan ini difokuskan untuk memberi tafsiran yang jelas kepada hasil ERT dan membuktikan bahawa ia berguna untuk diagnostik masa hadapan kestabilan tanah dan rawatan optimumnya yang diperlukan untuk tanah yang digunakan pada masa hadapan.

Kata kunci: *Pecutan tanah puncak, faktor penguatan tanah, ketidakstabilan tanah, penyatuan, suffusion, spektrum tindak balas reka bentuk*

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LIST OF ABBREVIATIONS

AC	Alternating Current
AF	Amplification Factor
bpf	Blows Per Feet
DC	Direct Current
DEM	Discrete Element Method
DSHA	Deterministic Seismic Hazard Analysis
e	Void Ratio
ESO	Elastic Spectral Ordinates
ERT	Electrical Resistivity Tomography
FEM	Finite Element Method
FOS	Factor of Safety
GMPE	Ground Motion Prediction Equations
G-R	Gutenberg-Richter
IP	Polarization
LL	Liquid Limit
m	cementation exponent
MASW	Multichannel Analysis of Surface Waves
n	Porosity
PGA	Peak Ground Acceleration
PGD	Peak Ground Displacement
PGV	Peak Ground Velocity
PI	Plasticity Index
PL	Plasticity Limit

PSHA	Probabilistic Seismic Hazard Analysis
RVE	Representative Volumetric Elements
SASW	Spectral Analysis of Surface Waves
SF	Suffusion Factor
SHA	Seismic Hazard Analysis
SP	Self-Potential
SPT	Standard Penetration Test
VED	Viscous Energy Dissipation
V _s	Shear Wave Velocity
W _c	Water Content

LIST OF SYMBOLS

D_G	Grain diameter
d_{10}	Grain diameter of 10% passing
d_{30}	Grain diameter of 30% passing
d_{60}	Grain diameter of 60% passing
d_{90}	Grain diameter of 90% passing
S_e	Elastic settlement
S_c	Primary settlement
S_s	Secondary settlement
ρ_o	Resistivity of rock
ρ_f	Resistivity of water
m	Cementation
ϕ	Porosity
E_n	Earthquake scenario
m_n	Magnitude
L_n	Location
r_n	Rate
η	Damping
Ω	Omega (unit of resistance)

CHAPTER 1

INTRODUCTION

1.1 Study Background

Internal erosion can take many different forms, can seriously endanger both human and animal life, and is capable of doing great harm to infrastructure (Liang et al., 2017; Masi et al., 2020; Huang et al., 2021). Piping and suffusion are two appearances of internal erosion. Different processes result in erosion, each of which is capable of causing destruction. Suffusion, also known as internal instability, is a long-term phenomenon whereby small soil particles are carried away by a soil seepage flow through spaces between larger ones (Dixon et al., 2011; Chetti et al., 2016). It indicates that a soil matrix's particle size distribution and the selective erosion of tiny particles from it do not match the requirements for self-filtering (Menad et al., 2019). Suffusion is more prone to arise in coarse, widely graded or gap-graded soils (such as some sandy gravels) (Bui et al., 2019). Internal instability is a common term used to characterize soils that are prone to suffusion. Suffusion, which is caused by seepage forces, is the mass movement of fine particles through the pore space of a coarser matrix (Yang et al., 2019) shown in Figure 1.1.

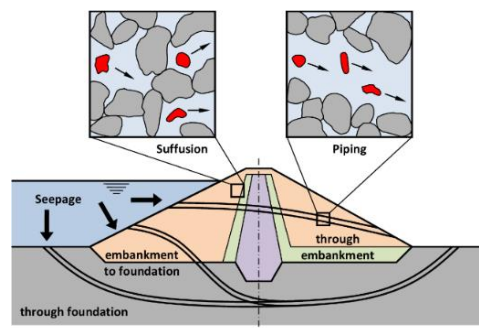


Figure 1.1: Internal Erosion Process (US Bureau of Reclamation, 2015)

Internal erosion of levees, earth dams, and foundations as well as watershed hillslopes is mostly caused by it (Feng et al., 2019). The impact of internal suffusion on a soil stratum's permeability, volumetric behaviour, and shear strength as well as the gradation are particularly concerning geo-mechanical soil parameters. Additionally, soil settlement has been connected to harm to earthen structures, buried utilities, buildings, and other structures.

The long-term impact that suffusion may have on the possibility for volumetric change to occur within a soil layer and the change in compressive strength is largely unexplored elements of geotechnical science. Additional knowledge on these internal erosion-related subjects can help with our understanding of the underlying mechanisms and processes, which will improve the way many earthen hydraulic structures are designed and protected from erosion's destructive impacts. Therefore, one of the primary mechanisms of internal erosion is suffusion, which results in selective erosion and progressive movement of tiny particles through the spaces in the soil skeleton created by coarse particles during seepage flow shown in Figure 1.2. Many hydraulic geo-structures, including embankment dams, dikes, levees, landslide dams, and natural deposits, exhibit seepage-induced suffusion (Wang, 2019; Yang et al., 2019).

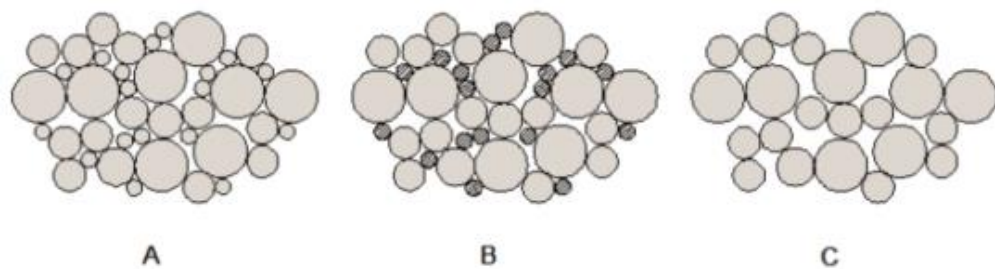


Figure 1.2: Progression of Suffusion; (A) Shows the Fine Particles Attached with Coarser Particles with a Seepage Line (B) Shows the Starting of Suffusion Influenced by Seepage and (C) Indicates the Suffusion in which the Fine Particles Flow with the Seepage Creating the Voids (Shwiyhat, 2010)

ERT is a quick and efficient non-destructive measurement technique for acquiring continuous soil subsurface resistivity profiles which is used in this research. Moisture variations and soil heterogeneities can be found using an ERT approach. ERT is becoming a prevalent tool in the field of geotechnical engineering (Masi et al., 2020). However, at this time, it only offers qualitative data. It can be difficult to determine quantitative geotechnical information about the subsurface from qualitative images, such as the moisture content, kind of soil, saturation degree, and Atterberg limits. Numerous studies have explained how pore fluid conductivity and surface conductance affect the electrical resistance of soil. To ascertain the impact of geotechnical features, electrical resistivity experiments have also been performed on commercial soils. Electrical resistivity must be associated with geotechnical parameters that can be measured in a laboratory because pore water and surface charge characterisation studies cannot be performed during a standard geotechnical investigation.

The natural disaster “earthquakes” is a set of vibrations on the surface of the earth which is caused by the generation of seismic waves due to rupture inside the earth during the release of accumulated energy (Shah et al., 2012). During an earthquake the delicate sediments may cause extensive amplification and increment in the span of ground motion, which may thus increase the seriousness of harm and devastation, this occurrence is generally termed as site effect (Laoumani et al., 2013; Sana et al., 2018). However, this study is focused on suffusion caused of failure through vibrations specifically soil amplification that has very close relation to soil shear wave velocity. The reason behind selecting seismic ground motion process that can cause soil failure instead of hydraulic loading because of the seismic hazard assessment carried out recently in Malaysia and the publish of Malaysia National Annex to Eurocode 8 by the Department of Standards Malaysia in 2017 proved that

earthquakes have been recorded in this region and it can happen again in the future. Correlation between shear wave velocity that represent soil amplification with electrical resistivity of soil within its cross-sectional spectrum that represent factor of suffusion, attest by experimental result, can give geotechnical engineer a good indicator of the limit state via soil acceleration response spectra, and suffusion factor is proposed to be a supplement to the design safety factor either in building design or slope design for roads.

Many research conducted in recent years to investigate suffusion susceptibility (Zhang et al., 1999; Kawano et al., 2018; Salahou et al., 2018; Alhasan et al., 2019; Luo et al., 2019) and its relation to hydraulic conductivity. This research however aimed to relate soil porosity and ground motion, in contrast to others, simply because the initiation of suffusion breaches or soil failures by suffusion are similar, hydrologically or seismically, and soil porosity (void ratio) is the cause of soil instability. By correlating soil porosity (void ratio) and ground motion via suffusion factor (SF) and amplification factor (AF) then alternative method of attaining soil properties via electrical resistivity test (ERT) for this study apart from boreholes or standard penetration test (SPT) to analyse suffusion can be proposed. With this correlation and proposal, ERT can be independently deploy to investigate suffusion and its potential consolidation/settlement subsequently can be used as supplementary coefficient or multiplier to factor of safety (FOS) in slope design.

The purpose of this study is to investigate the suffusion process at cut slopes in Pantu, Sarawak and determine how it affects the geo-mechanical characteristics of the soil, such as permeability, compression, and strength by using electrical resistivity tomography (ERT) and the laboratory tests of soils. Because due to the higher number of voids in the soil mass, the water is stored in the voids leading to saturation and due to increase mass of the soil, the

soil starts crumbling and slipping which leads to soil instability. In addition, this study aims to correlate the seismic soil response and suffusion i.e., to investigate the impact of earthquakes on suffusive soil.

Sarawak has a record of local origin earthquakes with low to moderate magnitude earthquakes ranging between 2.6 to 5.3. In addition, there are various active and in active fault lines in the Sarawak which are the source of moderate earthquakes. The fault lines and earthquakes in Sarawak are shown in Figure 1.3.

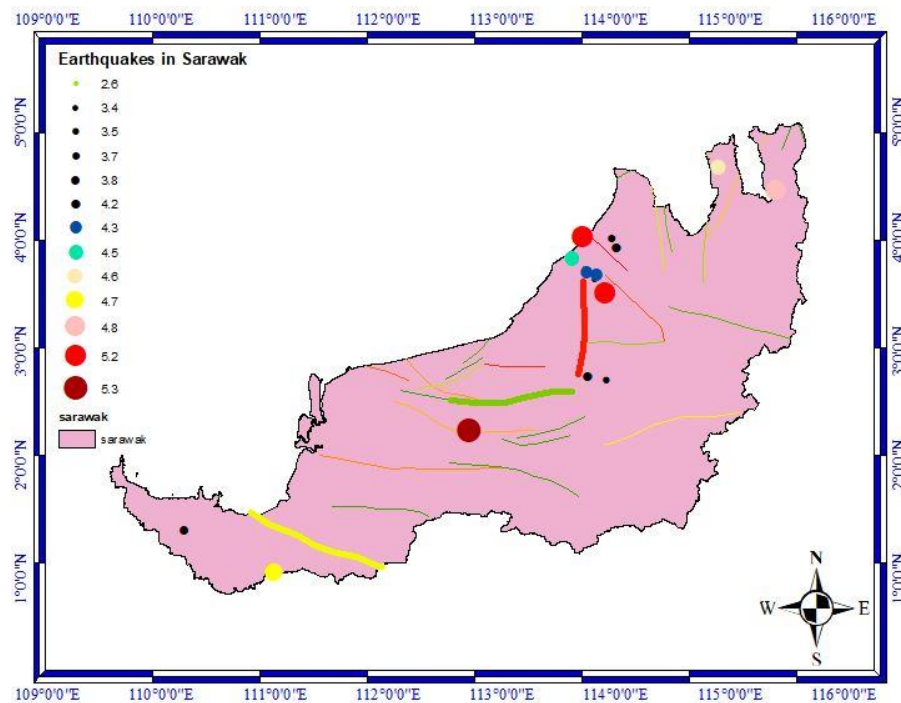


Figure 1.3: Local Earthquakes and Fault Lines in Sarawak (JMG, 2021)

The main goal of this experimental investigation is to ascertain correlations between geotechnical factors and electrical resistivity that geotechnical engineers can employ by looking at the relationship between the electrical properties of soil and the geotechnical parameters of locally available soil. In addition, to determine the impact of earthquakes on suffusion is among the motive of this research. For this study, a case study has been conducted in Sarawak state of Malaysia between Serian District and Sri Aman District to

investigate the failures demonstrated on the cut slopes in the Pantu Section (Pantu Sub-district) of Pan Borneo Highway Sarawak. ERT was conducted by using 20 electrodes at the selected locations in the Pantu section by using the Wenner-Schlumberger array in order to study the geophysical parameters of soil. In addition, standard penetration tests (SPT-N) boreholes were conducted in order to study the geotechnical parameters. From both ERT and SPT, results have been evaluated and correlations and formulations have been developed, and also the results from seismic-site response analysis is correlated with the suffusion parameters of soil, which are discussed in the results and discussion section in Chapter 4.

1.2 Problem Statement

The world has experienced soil failures due to internal soil instability caused by suffusion, including dam failures in the US, railway embankment failure in Southern Italy, and damage to homes, roads, and railroads during the 2007 Noto Hanto earthquake (Santhanakrishnan et al., 2014). Also, all earthen constructions, including river levees, roads with cut slopes, dams, and retaining walls, are seriously threatened by internal erosion phenomena. The inability to witness and monitor these events since they take place within the material has hampered the study of these phenomena. Non-destructive geophysical monitoring methods like ERT are effective tools that can shed light on the underlying processes (Huang et al., 2021).

Various research has been carried out on internal erosion of soils by (Yerro et al., 2017; Jedenius, 2018; Qin & Chian, 2019; Tao, 2018; Nguyen et al., 2019; Bi et al., 2021) but earthquakes have not been incorporated in the suffusion. Our infrastructure heavily relies on earthen structures, thus geotechnical engineers need to be aware of all the dangers they offer as well as how well they can function. There is a lot of information available about

internal piping erosion processes. Suffusion and its long-term impact on the functionality of an earthen construction under the influence of earthquakes, however, are mainly unexplored. A safer design and construction of this vital piece of infrastructure may be possible with more understanding.

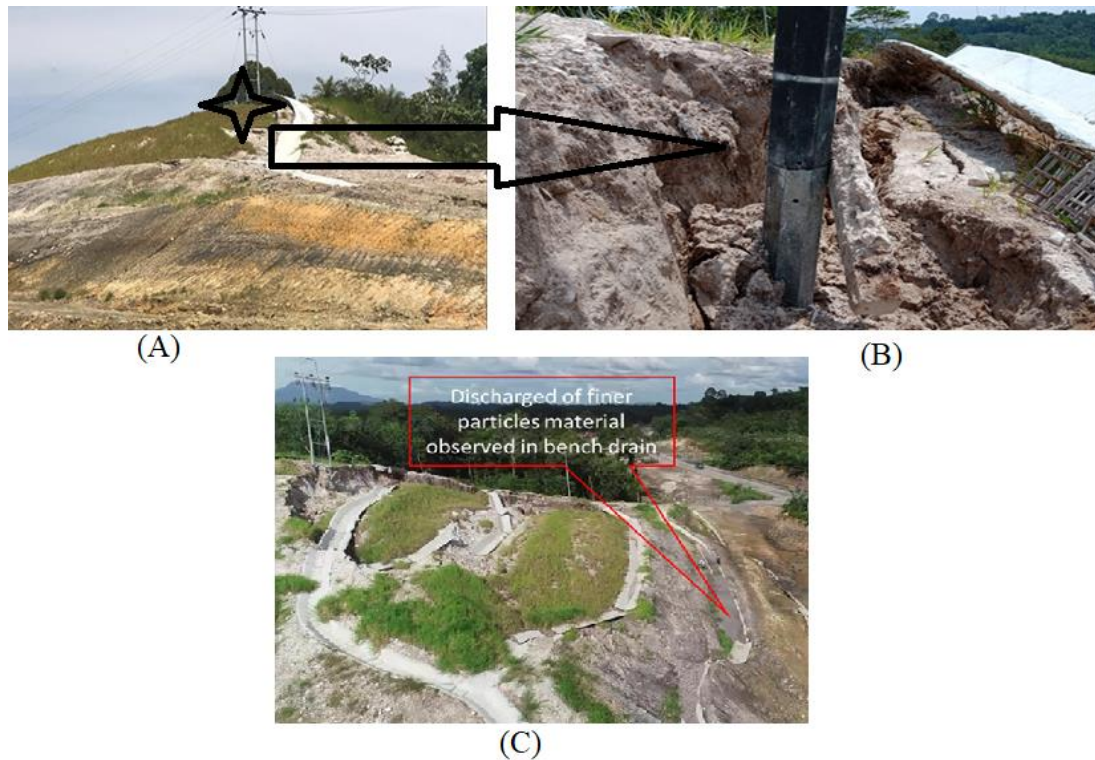


Figure 1.4: (A) Internal Erosion i.e., Suffusion had Caused Migration of Finer Particles. Suffusion Process Initiated – Fine Particles Filling the Void Internally. (B) Slope Failure or Initiation of Breach due to Suffusion Observed. Close-Up Observation of Slope Failure or Initiation of Breach. (C) Suffusion Failure Final Settlement Measured at 1.4m Deep at Top Slope.

The lack of proper consideration of this phenomenon had resulted to soil defects and failures on cut-slopes that subsequently damaging to the economy and ecology. Therefore, a case study in Sarawak between Serian District and Sri Aman District has been chosen in order to investigate the failures demonstrated on slopes in the Pantu Section (Pantu Sub-district) of Pan Borneo Highway Sarawak shown in Figure 1.4. Because Sri Aman district has many cut slopes on Pan Borneo highway that is why Sri Aman has been selected as the

case study. The requirement to identify the instability of soil is seriously needed and, in simplicity, all factors of uncertainty either material susceptibility or hydraulic conductivity or critical stress condition should be gathered in one analysis for quick front-end-engineering result to benefit mankind in general.

1.3 Aim of Research

The main aim of this research is to establish a new approach in order to determine design response limit (soil serviceability limit state) through electrical resistivity tomography and to correlate seismic site-specific response viz. soil amplification with internal instability of soil (suffusion)

1.4 Objectives

The objectives of this research are:

- i. To investigate the major uncertainty of suffusion in soil stability and apply the energy-based method i.e., ERT as an alternative assessment to detect suffusive soil
- ii. To develop a simplified equation from alternative data acquisition i.e., electrical resistivity to determine shear wave velocity, suffusion factor and limit of stable soil
- iii. To prove Archie's law porosity value is equal to V_v/V_T as per soil phase diagram

- iv. To conduct probabilistic seismic hazard assessment and investigate seismic-site response parameters viz. soil amplification, response spectra (spectral acceleration), Fourier amplitude and frequency

1.5 Hypothesis

Simplified approach using correlation between internal soil instability factors with seismic soil amplification can be achieved using soil data acquisition (electrical resistivity energy-based analysis) which is able to characterize the cause of soil failures or instability due to suffusion.

1.6 Significance of Study

This research endeavours to expose the common cause of soil failure and propose new possible data acquisition for analysis. Identification of the prime caused of soil failure is by way of literature review and establish alternative analysis to soil stability by correlating Archie's law and seismic soil response that is simpler, faster, reliable, and cheaper approach.

1.7 Scope of the Study

This research investigates soil instability in geotechnical engineering, emphasizing the consequences of failure and its impact on structures and the surrounding environment. The causes of soil instability, including water, critical stress conditions, and perilous voids, are explored. The study particularly addresses the significance of critical voids and migration of fines in relation to seismic soil amplification, with a focus on cut-slopes along the Pan Borneo Highway, Sarawak.

The findings enhance the reliability of design safety factors by integrating advanced technology. The research highlights the threshold index of soil instability and its correlation

with electrical conductivity. It concludes that ERT is a valuable tool for diagnosing soil stability, providing insights for optimal treatment in future land use planning. The research's output contributes to geo-hazard identification in the geotechnical engineering industry.

1.8 Organization of Thesis

The thesis organization is based on the various types of investigations that made up this study. It is structured as follows:

Chapter 1: Introduction; consists of background, problem statement, aim and objectives of this study, hypothesis, and significance of this research.

Chapter 2: Literature review; a review of research literature dealing with internal erosion, suffusion, electrical resistivity tomography (ERT), site response analysis with a prime focus on relationship between seismic site response and suffusion of soils, as well as concluding remarks and outline of the research program.

Chapter 3: Methodology; consists of field investigation and software modelling including the electrical resistivity tomography tests (ERT) in the field and analysis of the data provided during ERT.

Chapter 4: Results and discussion; this section consists of all the results carried out during this research and is comprising of ERT, suffusion, and seismic-site response analysis.

Chapter 5: Conclusion and recommendations; this chapter gives the overall summary of this research and highlights the carried work throughout this research and also provides the key points for the future research.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

Soil instability and failure is an on-going problem in the world, for this study in particular to review the latest available literature of the issues related to factors causing soil instability and instigating soil to fail. This chapter shall investigate, by literature, the common property shared within all causes of soil failure and/or can induced it, in addition further to investigate the all type of back-analysis that can determine the limit state of soil stability. At the end of this chapter the study will deliberately reveal gap in soil test that can determine limit state of soil stability without having to undergo laboratory procedure.

2.2 Issues Causing Soil Failure

Shan et al. (2021) re-examined the soil's failure in a lab setting using an undrained cyclic loading test to show several patterns of failure. Fine-grained soils typically perform poorly and exhibit "cyclic motion with liquefaction" in their behaviour. When compared to traditional fine-grained soils, collapsing failure is clearly visible in fine-grained soils. In actuality, the fine-grained soils' clay content, plasticity index, and water content to liquid limit ratios range from 3.0 to 20.0 percent, 0 to 9.6, and 0.85 to 1.11, respectively. There is some regularity in the particle size characteristics of the fine-grained soils with a collapse failure feature, according to the analysis of the soil indices for these soils (Shan et al., 2021).

The strain on all types of soil samples is quite low and oscillates about the x-axis in the early phases of the cyclic loading. When the axial (or shear) strain reaches a high strain level after a critical cycle, the samples reach a failure state. The excess pore water pressure

of the fine-grained soils rises significantly in the first few cycles before gradually rising (Freeze et al., 1979). However, after a critical cycle, the extra pore water pressure begins to build up and finally reaches a relatively high level. The rate of reduction in the effective stress is quite rapid at the start of the loading stage and thereafter becomes roughly constant, correlating to the excess pore water pressure. As soon as the effective cyclic route reaches the critical cycle and the samples satisfy the required failure conditions, the rate of effective stress reduction immediately increases. The characteristic of the fine-grained soils' collapse failure is shown by the hysteresis curves. Unlike the relatively little deterioration of stiffness in the early stages of the cyclic loading, the substantial loss of stiffness of the fine-grained soils arises shortly after a critical cycle. As cyclic loading cycles increase, viscous energy dissipation (VED) initially climbs significantly during the first one or two cycles before steadily declining until the minimal VED values are obtained. When the peak VED values are attained, the VED rises until the final stage of the cyclic loading, where the VED values quickly decline from the peak within a few cycles (Shan et al., 2021).

Five different types of their dynamic qualities help to explain how fine-grained soils with a collapse failure feature function. Furthermore, in the last phase of cyclic loading, every dynamic property displays a clear indication of collapse failure. Both the current classification criteria for soils, according to American Code and British Code (ASTM 2017-e1; BSI 2015), and the criteria for the liquefaction susceptibility of fine-grained soils highlight particle size characteristics as one of the soil indices.

$$\begin{aligned}
d_G &= \sqrt[n_i]{d_1^{n_1} \times d_2^{n_2} \times \dots \times d_i^{n_i}} \\
&= \sqrt[3]{d_{10} \times d_{30} \times d_{60}} \\
&= \sqrt[3]{C_u} \times \sqrt[3]{d_{10}^2 \times d_{30}} \\
&= \frac{d_{30}}{\sqrt[3]{C_c}}
\end{aligned}
\tag{Equation 2.1}$$

Therefore, D_G can be utilised to discriminate between the liquefaction behaviour of sands and the collapse failure of fine-grained soils. D_G is the grain diameter, which allows to describe the nature of the grains, the fine is the grain, and the compacted is the mass, when it comes to higher diameters i.e. coarse grains leading to higher voids, in turn increases the threat of instability. The Atterberg limits are frequently regarded as important indicators of soil strength and behaviour attributes. For instance, Boulanger and Idriss (2006) used the plasticity index (PI) to separate the behaviour of fine-grained soils that behaved like sand (PI < 7) and clay (PI > 7) for geotechnical engineering and practice. Additionally, it was suggested by Seed et al. (2003) and Bray and Sancio (2006) that one of the key criteria for determining which fine-grained soils are prone to liquefaction is a plasticity index (PI) of less than 12. Furthermore, a liquid limit (LL) of less than 32 and 37, respectively, was indicated by Andrews and Martin (2000) and Seed et al. (2003) as the crucial factor that determines the susceptibility of fine-grained soils. It is commonly acknowledged that soil strength is closely connected to its water content (Wc). Therefore, in this context, the ratio of the soil's water content to its liquid limit (Wc/LL) and the liquidity index (LI), which is defined as $(Wc - PL)/(LL - PL)$, are typically regarded as the key indicators to reflect the likelihood of the soils' strength loss (Boulanger and Idriss, 2006). Higher density and greater strength of a soil sample are represented by lower values of the w_c/LL or LI. It has been discovered that the fine-grained soil failure envelope lies within the indexes Wc, PI, and

wc/LL range, respectively, from 3 to 20 percent, 0 to 9.6, and 0.85 to 1.11 (Shan et al., 2021).

Figure 2.1 of a Venn diagram created by Garner et al. (2010) to demonstrate how soil collapse begins when one or more of the following factors occur or coincide in an unfavourable way:

- i. Material susceptibility
- ii. Stress conditions
- iii. Hydraulic load

Their research indicates that the term the relative erosion resistance of the soil, or the plasticity index of the soil, is another important factor in "material susceptibility." The hydraulic energy required to trigger an internal erosion mechanism through seepage flow through an embankment is known as the critical hydraulic load. In other words, this issue has to do with whether the embankment's or foundation's seepage gradients and velocities are sufficient to start particle movement.

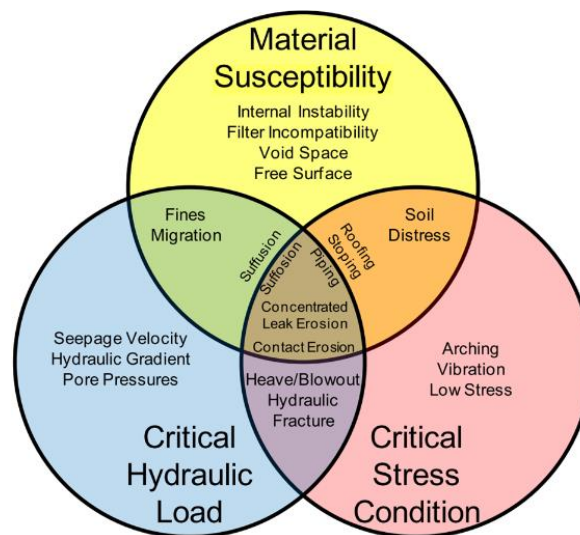


Figure 2.1: Illustration of Initiation of Soil Failure (Garner et al., 2010)

With understanding that stress changes geographically and/or temporally within an embankment, the capacity to resist internal erosion caused by the magnitude of effective stress is linked to the critical stress condition (Luo et al., 2019). Internal instability is affected by the stress condition, which can also be seen as showing the presence of "defects" in an embankment or foundation, whether these are brought on by cracking, hydraulic fracturing, arching, or other similar occurrences. The zone of the embankment that is vulnerable to all three elements is described in the centre subset. Material susceptibility, hydraulic loading, and critical stress combine to produce the release or detachment of soil grains as well as their movement.

In their investigation, Liang et al. (2021) came to the conclusion that fabric anisotropy governs the mechanical response of granular soils for a range of practical geotechnical applications. Their approach investigation looked at the interconnections between important soils grain-scale mechanisms that highlight significant macroscopic findings of the footing problem over a full loading range from peak to massive deformation regimes until its failure. They have employed MPM-DEM, which uses hierarchical coupling of the material point's technique and the discrete element method. Mesoscale ensembles of elliptical particles with certain alignments are developed to imitate bedding planes in anisotropic soils. They perform the role of representative volumetric elements (RVEs) using DEM solutions to extract nonlinear material responses in order to solve the footing as a boundary value problem that may experience considerable deformation to failure. They are incorporated within the material points of the MPM. The research backs up experimental data that show that when bedding angle α increases, the strip footing's bearing capacity declines. It shows how ignoring fabric anisotropy for soil with a horizontal/vertical deposition plane can result in a material overestimating or underestimating its bearing

capability. Every anisotropic situation, with the exception of the horizontal bedding case and the isotropic example, has final failure patterns that are primarily asymmetric and general failure modes with two major slide surfaces. The degree of asymmetry in the failure pattern and bedding angle are correlated with one another. To further support the previously published findings, microstructural studies on the evolution of several sources of fabric anisotropy in slip surface are used. In order to characterise a material, the Material Point Method uses a set of material 141 points that carry its mass, velocity, and other internal variables, including 142 stress (σ), deformation gradient (F), and void ratio (e) (Liang et al., 2021). Again, void ratio (e) and soil porosity is crucial common factor that can be delineator of failure envelope in soil mechanics. Correlation of sub-surface imaging with soil engineering properties should be extensively carried out. Adopting tomography or underground imaging like the medical industry such as x-ray, ultrasound, CT scan & MRI can provide better understand of failure mechanism in geotechnical engineering (Liang et al., 2021).

Rasti et al. (2021) stressed that one of the critical design parameters used in evaluating soil structures is the friction angle, which is obtained from the Mohr's Circle failure criterion. The soil friction angle, a measurement used in engineering, is computed in the lab to reflect the soil shear strength in geotechnical applications. Impact of size of particles on how different sandy soils behave in terms of shear strength. In order to replicate different stress regimes and determine the strength of the soil, the direct shear test device slowly imparts a lateral force to a compressed sample along a shear plane. Several direct shear tests were conducted to better understand how soil behaves at its interfaces. It is feasible to determine how soil particle size impacts the soil's shear strength behaviour by doing many symmetric direct shear tests in accordance with ASTM D3080 and assessing the

outcomes. A few of the factors that can influence the soil friction angle are the mineralogical composition, particle size and gradation, particle shape, compaction characteristics, confining pressure, roughness, moisture content, void ratio, and relative density (Vangla & Latha, 2015). The final friction angle and the thickness of the shear bands were observed to rise as particle size did. The trials showed that an increase in sand grains increased friction angle and pore pressure while decreasing void ratio and undrained shear strength (Prakasha & Chandrasekaran, 2005). Several investigations have demonstrated that the angularity impacts the sands' stress-strain response and that the peak shear strength increases as the interlocking forces increase (Vangla & Latha, 2015; Rouhanifar et al., 2020). It has been shown that the particle size has no effect on the peak friction angle if the void ratio is the same and that the void ratio has a bigger effect on the friction angle than the particle size. In the stress range examined, the results showed that the angle of friction for material with larger particle size was steeper than for other soils that include small particles. The friction angles achieved during the direct shear test ranged from 20° to 34°. The tested material's friction angle rose as the samples' uniformity coefficient and D_{50} increased. The results of the experiment demonstrated that the presence of coarse-grained material has an impact on the friction angle of soil. It was determined that friction angle and soil uniformity coefficient had a strong linear relationship with a high coefficient of determination. By increasing the uniformity coefficient, it was found that the interfacial frictional resistance recorded a larger value. Inadequate friction angle can cause soil failure and its direct correlation with void ratio is very significant for this study (Rasti et al., 2021).

Justyna et al. (2021) highlighted how each of the four isotropic failure criteria Mohr-Coulomb, Drucker-Prager, Matsuoka-Nakai, and Lade-Duncan has a distinctive parameter. According to Roscoe and Poorooshasb (1963), any two samples of a given soil will have the

same strains when exposed to stress paths with comparable geometry as long as each sample has a constant difference between the beginning void ratio and the void ratio at the critical state under the same normal stress. In the literature, soil constitutive models can be discovered. The Mohr-Coulomb condition, because of its simplicity, is the fundamental one to which all others are typically compared. It demonstrated its utility in the standard triaxial compression test, which involves applying an axisymmetric condition of stress ($\sigma_1, \sigma_2 = \sigma_3$) to a cylindrical soil sample. When genuine triaxial stress circumstances are taken into account, however, and the primary stresses $\sigma_1, \sigma_2, \sigma_3$ have different values, the issue of soil strength becomes more complicated. A gauge of soil failure state is thought to be peak soil strength. The Mohr-Coulomb condition, which causes the peak friction angle to be dependent on soil density, is used in the bulk of investigations on soil peak strength. To identify, both empirically and semi-theoretically, the parameters representing different soil failure criteria and their relationship to the internal friction angle in a specific range of initial soil densities under the condition of plane strain. Vikash and Prashant's solution is empirically tested after a study of the relationship between the intermediate principal stress σ_2 for the Drucker-Prager, Matsuoka-Nakai, and Lade-Duncan criteria that was found and noticed during the studies. A perfect match exists between the calculated and measured values. A linear relationship between the estimated and measured intermediate stress σ_2 is found for all three of the selected criteria, and Matsuoka-Nakai is extremely close to being a perfect fit. A variety of shear tests on Skarpa sand were conducted in an actual triaxial apparatus with plane strain. In trials that included a wide range of initial soil densities and confining pressures, the initial relative density index was used. The linear dependence of the friction angle is established. The main discovery for soil testing under plane strain conditions is the almost perfect fit of the Matsuoka-Nakai failure condition. The relationship between

the peak friction angle and the soil state parameter, which is determined by the difference in the void ratio between the starting and steady-state conditions, where the mean effective stress is the same, described by Been and Jefferies (1985, 1986), has been demonstrated (Sławińska et al., 2021).

Rostami et al. (2020) note in their framework research of the mechanics of failure, analysis, and re-qualification procedures of pile foundations in liquefiable soils during earthquakes that several of the most common failure mechanisms of piles during seismic liquefaction must be built into a strong conceptual and technical interpretation in order to acquire understanding of the mechanisms regulating the collapse of structures in liquefaction and identify appropriate requalification procedures. Priority is given to defining the concept of seismic design loads and important aspects of the dynamic behaviour of piles in liquefiable soils. The dynamic response commentary is offered utilising the Winkler foundation methodology together with the proposed p-y curves, finite element studies, and numerical analysis methods by critically analysing experimental studies carried out using shaking table and centrifuge modelling methodologies. Given the buckling process and the impact of lateral load, plastic hinge creation took place in a variety of places that are difficult to predict. The fact that pile foundations might have collapsed due to bending, buckling, or a combination of these actions was stressed. The dynamic behaviour of pile foundations is affected by site-specific elements, according to a number of physical modelling techniques (e.g., frequency of input motion, the amplitude, soil amplification etc.). The investigation of soil-structure interactions can benefit from complete imaging of sub-surface features, according to a geotechnical perspective. One of the main contributing factors or causes of soil and substructure collapse can be the soil's amplification factor (Rostami et al., 2021).

The soil resistance of a pile group is affected by specific soil factors and failure criteria, according to Ma et al. (2017). Under the Mohr-Coulomb failure criterion, the strength of sand differs between triaxial compression and triaxial tensile. Nonlinear static analyses were performed in three dimensions using the finite element method. The variance in sand shear strength determines how piles react to the subgrade. In sandy soil, the core of subgrade reactivity is the front piles in a pile group. While side piles in cohesive soil help to diffuse attention from the front piles. Under the Mohr-Coulomb failure criterion, the strength of sand changes between triaxial compression and triaxial tensile. The distinction determines how heaps behave below grade. Triaxial tensile strength determines the subgrade reaction deeper in the ground, whereas triaxial compression strength determines the subgrade reaction near the ground's surface. In sandy soil, the core of subgrade reactivity is the front piles in a pile group. While side piles in cohesive soil help to diffuse attention from the front piles. The concentration ratio of the subgrade reaction is examined. With increasing displacement in sandy soil, front pile concentration ratio increases while side pile concentration ratio decreases. However, in cohesive soil displacement, the concentration ratio of the front and side piles hardly ever varies. In cohesive and sandy soils alike, the subgrade reaction concentration ratios on front piles and side piles drop as they approach the ground surface. They are nearly constant values in the deeper portions of the ground. Correlation between sub-surface imaging and tomography with soil properties to be extensively carried out for better understanding of the near surface soil condition before comprehensive and sustainable design can be achieved. This includes soil porosity, compressive and tensile strength (Ma et al., 2017).

Ali (2020) discussed the causes of failure and evaluated a corrective plan by describing the slope material. The investigation has found that the seismic event's triggering

component is excessive water seepage brought on by poorly maintained runoff and sewage drains. In order to conduct a forensic geotechnical investigation of Qila Bala Hisar, both standard and unconventional testing techniques were employed. SPT, Atterberg Limit, and ERT tests on the site-specific soil were carried out. The surcharge load has grown at the site due to the construction of the additional structures. Due to an increase in the population, the fort, which was initially intended to house a limited number of residents, is today occupied by a huge number of people. The failure of the wall during the time of the earthquake, which occurred on October 27, 2015, was one of the potential causes of the wall's failure, along with the soil mass. The ERT for the site has identified pockets of waterlogged earth. These potentially saturated spots suggest that water has seeped into the ground since they are present. The following list includes some potential root causes of the water seepage: (1) seepage of water from adjacent lawns; (2) nearby damaged water supply and sewage system pipes; (3) inadequate drainage, blocked drains, or poor gradient of ground surface; and (4) infiltration/seepage of rainwater. It is conceivable that there were three saturated pockets given the concerns stated and the test results. The seepage of water from the neighbouring lawn, nearby damaged water supply and sewerage system pipes, poor drainage or drainage blockages, rat holes leading to the affected area, vibrations from generating sets situated right next to the damaged portion, and surcharge loads were some of the causes of these saturated pockets. Water seepage was the primary cause of the aforementioned elements, and an earthquake served as the catalyst. The breakdown of the slope and the collapse of the wall was caused by both of these factors. Repair of the collapsed wall and its nearby failed slope was advised as a corrective measure while taking into account the strata and condition of the area. Ground Investigation to include tomography survey likely Electrical Resistivity Imaging similar to MRI in medical field for better interpretation and proper land used and

future planning. Void and porosity are the cause underlying the failure prior to ground motion (Naveed et al., 2020).

As stated by Garner and Fannin (2010), there is no disputing the fact that the reasons of soil failure are (i) material movability, (ii) critical hydraulic load, and (iii) critical stress condition, and the mechanism originating the sources of volatility include internal instability, filter incompatibility, void space, free surface, seepage velocity, hydraulic gradient, pore pressures, arching, vibration, and low stress. Further to that, commonly shared property of vulnerability that caused soil to fail, related to all the above, can be narrowed down to single property and that is void or porosity.

2.3 Common Property Shared within all Causes of Soil Failure

Issues causing soil failure are well presented by Shan et al. (2021) research. From soil mechanics stand point via soil phase diagram, bands of soil properties contributing to instability of soil were tested, analysed and concluded. All soil properties specified i.e., clay content classified as fine-grained soil, plasticity index range that indicate soil mixture proportion and liquidity index are very much related to the volume of void (V_v) and the variation of volume of air (V_a), volume of water (V_w), combined (Benson et al., 2006). Osman et al. (2013) findings are complimentary to Shan et al. (2021) research where estimating clay content and plasticity index are attainable with electrical resistivity value. This study demands for possibility to estimate the LL, Wc and LI to complete the correlation accordingly.

According to James (2014), 3 main sources of information needed to identify an area susceptible to liquefaction:

- i. Historical information as evidence
- ii. Hydrological and geological surroundings
- iii. Geotechnical and/or geophysical analyses based on experiments that have been carried out.

Jaimes et al. (2015) stress that soil vulnerable to liquefaction has certain geology and geotechnical properties. The soil deposits most prone to liquefaction are those with minimal cohesiveness (such as fine sand and fine sandy silt) (Jaimes et al., 2015; Sato et al., 2015; US Bureau of Reclamation, 2015; Parekh, 2016; Kawano et al., 2018; Luo, 2021). Consequently, sands with very little flexibility correspond to natural soil deposits with greater sensitivity to liquefaction. Evidence suggested that gravels in saturated soils are typically loosely deposited in a silt or sand matrix that is susceptible to experiencing soil failure and was greatly impacted by liquefaction during big earthquakes (e.g. 2008 Wuhan, China earthquake) (Nazri et al., 2015).

Both natural deposits and filled embankments have been found to exhibit suffusion, an internal erosion, in large quantities. The soil skeleton is eventually left behind as fine soil particles migrate via spaces between the larger soil particles (Bowles, 1970). Due to their lack of particle size, gap-graded and broad-graded cohesionless soil is particularly susceptible to internal erosion. Internal erosion is indicated by the loss of tiny particles, which changes the void ratio and considerably increases hydraulic conductivity, lowering the soil's capacity from its initial strength (Luo et al., 2019; Luo et al., 2020; Luo, 2021)

Suffusion is described as a vast mass of dispersed and hardpan detrital species, including those that make up the structural parts of rocky massifs, and damaging and fine particles carried by a stream of underground waters, in accordance with Russian Standard

CR 116.13330.2012 (Ke & Takahashi, 2012; Ke & Takahashi, 2014; Fannin et al., 2015; Horikoshi & Takahashi, 2015; Ke et al., 2015; Sato & Kuwano, 2015; Sibille et al., 2015; US Bureau of Reclamation, 2015; Parekh, 2016; Bendahmane et al., 2017; Kawano et al., 2018; Lavrusevich & Lavrusevich, 2018; Salahou et al., 2018; Luo et al., 2019; Alhasan et al., 2019; Zhang et al., 2019).

Suffusion can develop cavities through migration and dispersal of fine particles of soil relating to various soil types can be a big threat to soil-structure interaction (Skempton et al., 2011; Ke & Takahashi, 2012, Lam et al., 2012; Ke & Takahashi, 2014; Santhanakrishnan et al., 2014; Fannin et al., 2015; Horikoshi & Takahashi, 2015; Ke et al., 2015; Sato & Kuwano, 2015; US Bureau of Reclamation, 2015; Parekh, 2016; Kawano et al., 2018; Alhasan et al., 2019; Zhang et al., 2019).

If designed facilities are not located in hazardous karst regions, coastal areas of reservoirs, or slopes that are prone to landslides, industrial and civil engineers may ignore the risk of a suffusion during construction and maintenance (Lavrusevich & Lavrusevich, 2018).

Thermal monitoring allows for the identification of the suffusion process due to distinctive changes in the dam's hydrothermal environment (Radzicki & Bonelli, 2012; Alhasan et al., 2019). Despite the fact that little research has been done on the post-erosion mechanical effects under monotonic shearing, the effects of suffusion on the cyclic resistance and liquefaction potential of internally unstable soils have received little attention (Mehdizadeh et al., 2017). Because eroded specimens with lower intergranular void ratios exhibit superior resistance during cyclic loading, understanding the intergranular void ratio

is essential for understanding the mechanical behaviour of soils post erosion (Luo et al., 2019; Mehdizadeh et al., 2019).

Individual particle suffusion under seepage tend to be influenced by a mix of stresses (Ke & Takahashi, 2012, Lam et al., 2012; Ke & Takahashi, 2014; Santhanakrishnan et al., 2014; Fannin et al., 2015; Horikoshi & Takahashi, 2015; Ke et al., 2015; Sato & Kuwano, 2015; US Bureau of Reclamation, 2015; Parekh, 2016; Kawano et al., 2018; Alhasan et al., 2019; Zhang et al., 2019).

Suffusion shared common caused to soil failures by mean of the following:

- i. The size of the tiny particles is less than the size of the spaces between the coarse particles that make up the soil's skeleton, and there are not nearly as many fine particles as there are voids to fill (Ke et al., 2012).
- ii. The fine soil particles can migrate through the spaces between the coarse particles because the hydraulic gradient (hydrologic) is reasonably big (Ke et al., 2012; Sibille et al., 2015; US Bureau of Reclamation, 2015; Parekh, 2016; Alhasan et al., 2019).
- iii. A seismic event, such as an earthquake, results in deformation (rapid migration through vibration, filling of spaces between larger particles, and/or fracture that would start internal erosion) (US Bureau of Reclamation, 2015). Granular soils' mechanical properties change when they are subjected to seismic loadings. There are two extremes in terms of soil saturation: dry soil and saturated soil, which can lead to densification and liquefaction events, respectively. The solid skeleton rearranges as the shaking goes on, leading to

densification. As a result, the initial soil volume is reduced, and the soil stiffness frequently rises.

Franz-List (1999) presented on research on employing geotextiles and geosynthetics to increase the safety against suffusion and erosion in an embankment (Gohari et al., 2018). In general, materials with $C_u < 10$ were regarded to be self-filtering, whereas those with $C_u > 20$ were deemed to be possibly unstable. A local gradient reduction that is temporally compressed can indicate instability. The primary determinants of the location of instability inside a site-specific are spatial variation of localised hydraulic gradient and vertical effective stress, with internal instability being triggered either by an increase in hydraulic gradient or by a fall in effective stress. When suffusion erosion started, permeability dropped, indicating blockage. Suffusion was examined by Bonelli and Marot (2008) as bulk erosion, where clayey sand erosion took place at the clay-water contact. Macroscopic bulk erosion is driven by the pressure gradient rather than the seepage velocity for the migration to occur (Yee et al., 2013; Parekh, 2016).

Further from the literature review, the direction of identification of common phenomenon shared within all caused of failure is drawn towards suffusion and common property in suffusion is void or porosity (Sato et al., 2015; Kondratov et al., 2017; Chung et al., 2018; Kawano et al., 2018; Kodikara et al., 2018; Sannikov et al., 2018; Sirca et al., 2018; Alhasan et al., 2019; Chang et al., 2019; Cheng et al., 2019; Maffra et al., 2019; Pachideh et al., 2019; Zhang et al., 2019; Chang et al., 2020; Deng et al., 2020; Luo et al., 2020; Luo, 2021).

2.4 Analysis for Soil to Determine Limit State of Soil Stability

2.4.1 The Verification Methodology Based on the Limit State Theory

The limit states theory or verification methodology establishes the safety by opposing a resistant variable (resisting force, strength, or bearing capacity) with a variable causative for failure either sliding force or stress (Deng et al., 2017).

$$X_{pas} > X_{act} \quad \text{Equation 2.2}$$

Where:

X_{pas} – A variable resisting the failure (resisting force, strength, capacity), and

X_{act} – A variable causing the failure (sliding force, stress)

X_{act} is in general determined from the design parameters of soils and load:

- i. soil parameters are reduced by corresponding coefficients
- ii. load (its action) is increased by corresponding coefficients

X_{pas} is determined based on the following assumptions:

- i. soil parameters are reduced by corresponding coefficients
- ii. the calculated structure resistance is reduced by a corresponding coefficient

The oldest and most used way of verifying construction safety historically is based on the "safety factor." The key benefit is how logical and simple it is. Typically, the safety factor is used to demonstrate safety:

Equation 2.3

$$FS = \frac{X_{pas}}{X_{act}} > FS_{req}$$

Where, FS – Computed safety factor, FS_{req} – A Required factor of safety, X_{pas} – A variable resisting the failure (resisting force, strength, capacity), and X_{act} – A variable causing the failure (sliding force, stress). The safety factor, abbreviated as F_s , is formally defined as the ratio of the material's actual shear strength to the mobilisation strength needed in the limit (equilibrium) condition, as shown in;

$$F_s = \frac{c + \sigma_n \tan \varphi}{c_d + \sigma_n \tan \varphi_d} = \frac{c}{c_d} = \frac{\tan \varphi}{\tan \varphi_d} \quad \text{Equation 2.4}$$

where c_d and φ_d = parameters of mobilised strength in the limit state (Qin et al., 2018). The safety factor was incorporated to kinematic analysis, allowing for the identification of the failure load in closed form in accordance with plasticity theory. As a result, by applying the strength-reduction technique to the soil slope, the analysis can reach the ultimate limit state with mobilised strength parameters.

2.4.2 Geophysical Survey for Ground Investigation

The geophysical surveying techniques can be broadly divided into those that utilise the earth's natural fields and those that involve injecting intentionally produced energy into the ground shown in Table 2.1 (Malehmir et al., 2013). The natural field methods make use of the Earth's gravitational, magnetic, electrical, and electromagnetic fields to search for regional perturbations in these fields that could be caused by subterranean geological structures of economic or other significance (Duffy et al., 2014).

Table 2.1: Geophysical Investigation/Survey Methods

Method	Measured parameter	Operative physical property
Seismic (MASW)	Travel times of seismic waves that are reflected or refracted	The speed at which seismic waves propagate depends on density and elastic moduli
Gravity	Fluctuations in the intensity of the Earth's gravitational field over space	Density
Magnetic	The geomagnetic field's intensity varies spatially	Magnetic remanence and susceptibility
Electrical Resistivity; Induced polarization; Self-potential; Electromagnetic Radar;	Earth resistance Polarization voltages or frequency-dependent ground resistance Electrical potentials Response to electromagnetic radiation Travel times of reflected radar pulses	Electrical conductivity Electrical capacitance Electrical conductivity E. conductivity & inductance Dielectric constant

2.4.3 Probabilistic Seismic Hazard Assessment on Sub-Surface

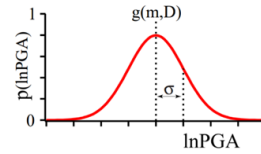
Probabilistic seismic hazard assessment (PSHA), has been used for more than 40 years as a standard input in engineering design and other industries (Hatheway, 1996). Methodologies for assessing seismic risk have developed from straightforward maximum

anticipated ground motions based on a single expert's assessment to probabilistic assessments employing data ranges and standard deviations to taking into account the statistical distribution of expert opinions (Hofmann, 1993). The process of characterizing seismic sources, which involves determining where earthquakes happen, is a component of seismic hazard assessment (Kagan et al., 2005; Beauval et al., 2006). How frequently do earthquakes happen? And how large should these earthquakes be? Seismicity catalogues of the locations of all instrumentally recorded earthquakes, which are obtained as the first step in SHA (Klügel et al., 2013; Papanikolaou et al., 2015; Prachař, 2015). The location, frequency, and magnitude of earthquakes are all determined using the fundamental data from these catalogues (Leonard et al., 2014). Depending on the historical and paleo seismic data, the catalogues may be enormous and span a thousand years or more (Mccalpin et al., 2015; Meltzner et al., 2015).

To define seismic sources, information from the catalogues, such as historical records, seismic monitoring, geodetic monitoring, which includes the positions of active fault lines and the depth or area of seismic sources, is merged (Jaimes et al., 2015; Andreev, 2016; Kotha et al., 2016). Seismic source zone is the process of identifying the source of the fault or the source of the region and delineating it accordingly. The minimum (damage threshold) and maximum earthquake magnitudes, as well as the rate of seismicity, are characterised by the Gutenberg-Richter (GR) law (Yamaguchi et al., 2011) this has the following formula: $\ln N = a - bM$, where M is the magnitude, N is the number of earthquakes of M or higher, and a and b are constants (Atkinson et al., 2015; Crampin et al., 2015; Andreev, 2016; Onur et al., 2016). Peak ground motion values, sometimes referred to as "attenuation" relationships, are used as the intensity measure in ground motion prediction equations (GMPEs), elastic spectral ordinates, and peak ground acceleration (PGA), peak

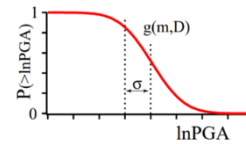
ground velocity (PGV), and peak ground displacement (PGD) (Brabbaharan et al., 2005; Lin and Lee, 2008). Standard probabilistic seismic hazard analysis (PSHA) techniques have been modified to include GMPEs empirical intensity metrics (Hassani & Atkinson, 2015; Kotha et al., 2016; Fäh et al., 2017). The probabilistic seismic hazard calculation is simple after the source model for the seismic hazard and attenuation relationships have been established. The source of seismic hazard model has provided N earthquake scenarios E_n , which has a specified related magnitude (m_n), location (L_n), and rate (r_n) (Lindh et al., 1999; Pallav et al., 2012). From the earthquake location we can determine the distance D_n to our site. From the m_n and D_n , the attenuation relationship shall interpret the distribution of possible ground-motion levels for the scenario (Carvalho et al., 2009). The following summarizes the empirical expression (Field, 2003):

$$p_n(\lnPGA) = \frac{1}{\sigma_n \sqrt{2\pi}} e^{-(\lnPGA - g(m_n, D_n))^2 / 2\sigma_n^2}$$



Equation 2.5

$$P_n(> \lnPGA) = \frac{1}{\sigma_n \sqrt{2\pi}} \int_{\lnPGA}^{\infty} e^{-(\lnPGA - g(m_n, D_n))^2 / 2\sigma_n^2} d\lnPGA$$



Equation 2.6

2.4.4 Site-Specific PSHA for Identification of Soil Response

Numerous earthquakes have effectively demonstrated the historical effect of site amplification caused by soil conditions. The extensive damage brought on by the Guerrero earthquake (1985) in Mexico City, the Spitak earthquake (1988) in Leninakan, the Loma Prieta earthquake (1989) in the San Francisco Bay area, the Kobe earthquake (1995) in Japan, the Kocaeli earthquake (1999) in Adapazari, Turkey, and the Bhuj earthquakes (2001) in India serve as crucial examples of site-specific amplification of ground motion, even at

locations 100km to 300km from the epicentre (Vucetic and Dobry, 1991; Anbazhagan et al., 2008).

For an earthquake-resistant design, the local soil conditions are crucial (Bommer and Abrahamson, 2006). Site-specific studies as shown in Figure 2.2 are only conducted in unusual circumstances due to financial constraints i.e. it is expensive. Site-specific correlations between shear wave velocity, V_s , and SPT data can be produced by altering existing correlation equations or developing new correlations based on site-specific data. The sensitivity of the synthetic ground motion and slope surface of the soil slice layers to the shear wave velocity should be taken into account (V_s) or V_{s30} (Barani et al., 2014).

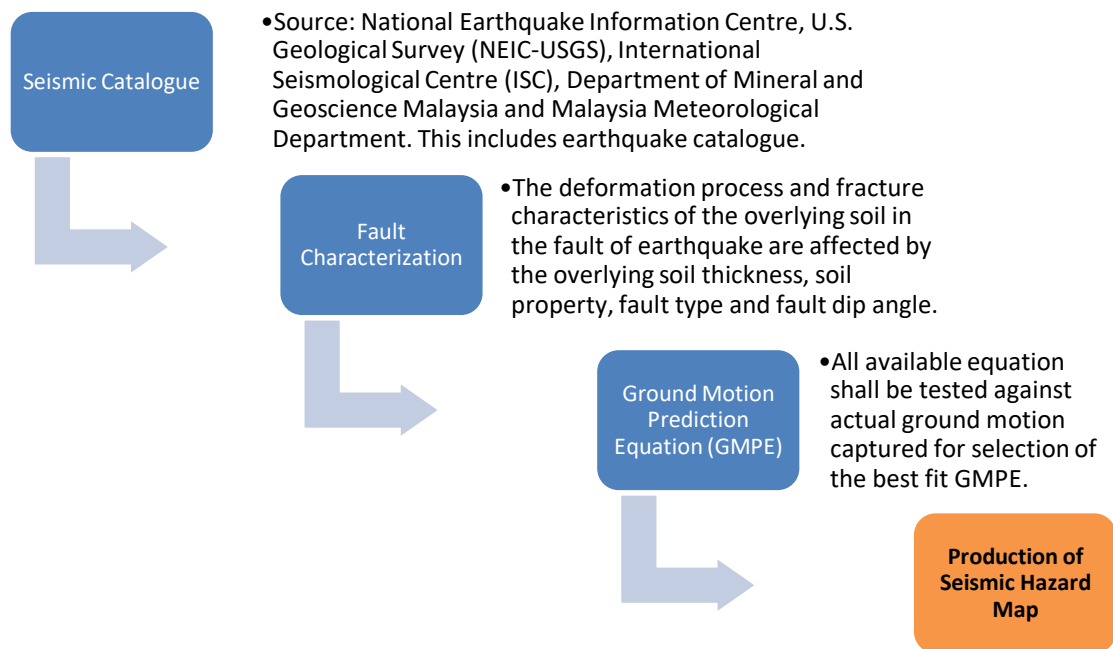


Figure 2.2: The Framework Probabilistic Seismic Hazard Assessment used Worldwide

The three most popular techniques used to estimate the response of Single Degree of Freedom (SDOF) systems and then construct the response spectra are the frequency-domain solution, the Newmark β method, and Duhamel integral solutions. For each approach used

to determine the response of SDOF systems and solve the dynamic equilibrium equation, which is defined and a brief description is provided as (Balendra and Li, 2008; Pappin et al., 2011; Hashash, 2016):

$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g \quad \text{Equation 2.7}$$

where the SDOF system's mass, viscous damping, and system stiffness are all represented by the letters m , c , and k , respectively. The nodal relative accelerations, relative velocities, and relative displacements are denoted by $\ddot{u}$, $\dot{u}$ and u correspondingly. The exciting acceleration at the base of SDOF is denoted by $\ddot{u}_g$ mentioned in Equation 2.7.

The peak of the response spectra is equal to soil amplification factor/function and represented by (JRC European Commission, Eurocode, 2004):

$$AF = 2.5S\eta \quad \text{Equation 2.8}$$

where, acceleration spectral amplification is a fixed variable = 2.5, S = soil factor and η = damping correction factor ($\eta = 1$ for 5% damping).

The mathematical technique of Fourier analysis, which looks for cycles or patterns in a set of time series data, has previously undergone standardization and normalized. It specifically aims to decompose complex or noisy data into a sequence of trigonometric or exponential functions using seismicity data collected for this study in order to simplify complex or noisy data. The observed data might then be reconstructed by combining each of these sine waves, each of which would have a unique cycle duration, amplitude, and phase relationship with the others. Then, using a time-based pattern, Fourier Transform measures each potential cycle and returns the total number of waves captured (the amplitude, offset, and rotation speed for every cycle that was found). The Fourier Transform can match any

temporal signal by determining the proper cycle rates, amplitudes, and phases. This collection of transformed wave/spectrum from various soil types and combinations can be used to create a soil-amplification limit chart or design response spectra which is covered and ready-supplied in Malaysia National Annex to Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings.

$$X_k = \frac{1}{N} \sum_{n=0}^{N-1} x_n \cdot e^{-i2\pi kn/N} \quad \text{Equation 2.9}$$

Where, to find the energy at (X) frequency (k), spin (e^{-i}) signal (x_n) around a circle (2π) at that frequency (k), and average a bunch of points along that path and (n/N).

2.4.5 Electrical Resistivity of Soil and Identification of Void Ratio/Porosity of Soil

The spatial and temporal variability of a variety of other soil physical characteristics can be substituted by the electrical resistivity of the soil (i.e., structure, water content, or fluid composition). Because it is non-destructive and exceedingly sensitive, the method offers a very appealing instrument for documenting the underlying structures without excavating (Verwer et al., 2011; Kowalczyk et al., 2017). It has already been used in a variety of contexts, including groundwater exploration, landfill and solute transfer delineation, agronomical management by locating areas of excessive compaction, determining the thickness of the soil horizon and the depth of the bedrock, or at the very least evaluating the hydrological properties of the soil (Sudha et al., 2009; Whalley et al., 2017; Bai et al., 2013; Jinmin et al., 2018; Cheng et al., 2019).

Archie's law outlined how the porosity of rocks and their electrical conductivity relate to one another. It is believed that all conduction happens only through the pore-solution and that the rock itself is non-conductive (Glover, 2016). But very little work has

been done by researchers to determine whether Archie's law applies to soils (Glover, 2016; Nassimi et al., 2016; Zhu et al., 2016; Ali, 2017; Rothfus et al., 2017). Therefore, this study application of Archie's rule for soils which takes into account their physical characteristics, volumetric moisture content, and apparent resistivity was made possible by the information gathered in locations between Serian and Sri Aman in Sarawak. To further support the legitimacy of this law, twenty-one (21) samples of soil were used, attained from an undisturbed location where no known induced compaction nor consolidation taken place before the construction of Pan Borneo Highway of Sarawak. These soils have been tested and characterized accordingly by conventional laboratory tests, and their apparent resistivity were captured via electrical resistivity test (ERT) (Samouëlian et al., 2005).

Before any soil, the complete void can be represented by its liquid limit when it experiences an elastic settlement in a natural setting outside of a lab that is brought on by the elastic deformation of both dry and moist of saturated soils without affecting the moisture content of either (Seladji et al., 2010). This is due to the fact that the soil's shift from a liquid to a plastic state occurs at the liquid limit, which is an empirically determined moisture level. When a soil is dried in an oven at a temperature no greater than 110 °C, the amount of water present in the gaps between soil grains is measured as the soil's moisture content. The amount of moisture has a significant impact on how the soil behaves (Sivakumar et al., 2015; Sivakumar-Babu, 2016; Deng et al., 2017; Shan & Ke, 2021). Therefore, the amount of the total soil volume that is occupied by the pore space, also known as soil porosity, and the moisture content from liquid limit occupy the same volume. Hence, from ERT one can determine the soil porosity which is equal to liquid limit of that particular soil. Additionally, this makes a significant contribution to the prediction of a soil's elastic settlement (S_e),

primary consolidation (S_c), and even secondary consolidation (S_s) (Purvance and Andricevic, 2000).

Archie's first law is a tool used in petroleum engineering to determine the cementation exponent of rock units (Abbas, 2011; Tucker, 2013; Berg, 2014). The volume of hydrocarbons in the rocks may then be determined using this exponent, allowing for the estimation of reserves (Archie, 1942). It is considered that Archie's Law does not apply to rocks that include a sizable amount of clay. One of the key presumptions is disproved by the conductive matrix that clay offers. Electrical conductors can also be found in graphite, native metals, and minerals with metallic clusters, however these materials are much rarer than clays. Archie's law is given by the below equation:

$$\rho_o = \rho_f \varphi^{-m} \quad \text{Equation 2.10}$$

Where ρ_o is the resistivity of a rock sample that has been completely saturated with water, ρ_f is the resistivity of the water that has saturated the rock's pores, φ is the rock's porosity, and m is the cementation exponent (Glover, 2009). At least nine out of ten times, however, reservoir engineers and petro physicists do not use Archie's first law in this manner. Instead, they employ a slightly modified version that Winsauer et al. (1952) published ten years later, and which has the form:

$$\rho_o = a \rho_f \varphi^{-m} \quad \text{Equation 2.11}$$

Where, also known as the "lithology constant" or the "tortuosity constant," is an empirical constant. In actuality, neither the new characteristic which we'll refer to as a parameter nor the rock tortuosity or lithology are related (Glover, 2015).

It is intended to account for variations in grain size, pore structure, and compaction (Archie, 1947). The path length of the current flow is obviously related to the tortuosity factor, often known as the parameter ‘a’. The value lies in the range 0.5 to 1.5 (Archie, 1947) and since the test site for this study involving undisturbed area therefore value $a = 0.5$ is used.

Almost always, the minerals that make up a rock are electrical insulators. Electrical conduction is consequently possible because of the moisture that exists in the pores of the rock and soil. Several factors affect the resistivity of rocks and soil. These include the amount of clay present, the moisture salinity, the level of pore saturation, and the quantity, dimensions, and morphology of the interconnecting pores. Porosity of soils is influenced by the degree of compaction, which is another crucial element. Archie established the empirical formula below in 1942 and it connects resistivity to porosity, saturation level, and resistivity of the saturating moisture (Ali, 2017).

$$\rho_o = a\phi^{-m}s^{-n}\rho_w \quad \text{Equation 2.12}$$

Where ϕ is the fractional pore volume (porosity), s is the percentage of the pores that contain water, ρ_w is the water's resistivity, and n is about 2. a and m are constants, changing from 0.5 to 2.5 and from 1.3 to 2.5, respectively.

Since the electrolyte in the pores of soil and rock is used to conduct current, the porosity, or void ratio, of the material and the geometry of the pores play a significant role in determining resistivity (Mukhlisin et al., 2018; Hegde et al., 2021). Intergranular voids, joint or fracture holes, and blind pores, such as bubbles or vugs, are examples of the several types of pores that can exist. Only the interconnected pores effectively contribute to

conductivity, and this contribution is further influenced by the geometry of the interconnections or the tortuosity of the current paths. Factors influencing electrical conductivity in rocks and soils are:

- i. Porosity (connected/effective - fractures or pores)
- ii. Pore saturation (% air or gas)
- iii. Hydrocarbon Fluid Saturation
- iv. Water salinity (TDS)
- v. Clay Content
- vi. Metallic Sulfide Mineral Content
- vii. Fluid temperature
- viii. Rock Matrix intrinsic resistivity

With the exception of the cementation exponent (m), all of the aforementioned parameters can be determined by testing soil samples in a lab and using ERT. But as the rock becomes more tortuous (path length), tortuosity becomes more than 1. This links the cementation exponent to the rock's permeability; as permeability increases, the cementation exponent drops. For unconsolidated sands, the exponent m was found to be close to 1.3, and it is thought that it rises with cementation. This cementation exponent typically has values of $1.8 < m < 2.0$ for consolidated sandstones. Due to strong diagenetic affinity and intricate pore architecture, carbonate rocks exhibit a larger variance in the cementation exponent. Values between 1.7 and 4.1 have been observed (Archie, 1942).

Electrical resistivity can offer continuous measurements over a wide range of scales without causing any damage. This method allows for the monitoring and quantification of temporal variables without changing the internal soil structure, such as water and plant

nutrients, which depend on it. Resistivity characteristics of geological targets (Palacky, 1987) is shown in Figure 2.3. This method can be deployed to many engineering applications such as:

- i. Identifying the soil horizon and certain heterogeneities.
- ii. Monitoring the transportation phenomena.
- iii. In a saline or waste context, monitoring of solute plume pollution.
- iv. Information regarding the physical characteristics of soil, such as its porosity and soil phases, can be obtained via electrical resistivity investigations (Table 2.2). Recent advancements in electrical technology enable quick spatial and temporal resolution.
- v. Archie's law:

$$\rho = a \Phi^{-m} f^{-n} \rho_w \quad \text{Equation 2.13}$$

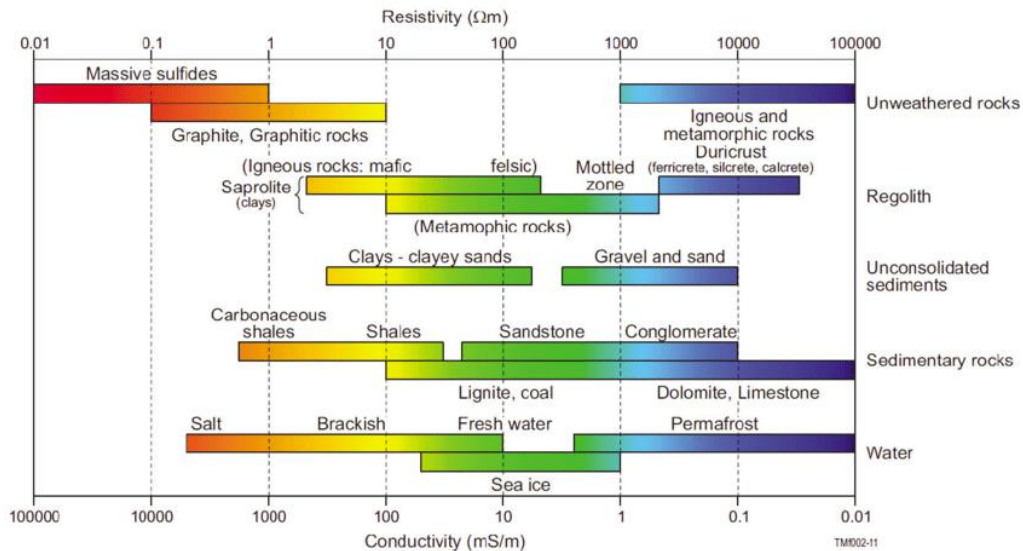


Figure 2.3: Resistivity Characteristics of Geological Targets (Palacky, 1987)

Table 2.2: Founded Subsurface Insights and Published Refined Electrical Resistivity (Palacky, 1987)

Material	Electrical Conductivity (mS/m)	Electrical Conductivity ($\mu\text{S/cm}$)	Electrical Resistivity (Ohm.m)	Comment
Surface & groundwater	5-150	50-1500	6-200	Large variability
Seawater	3000-5000	30000-50000	0.2-0.33	Mean: 3270 mS/m
Unconsolidated materials	10-1000	100-10000	1-100	saturation & mineralogy dependence
Sedimentary Rocks	1-200	10-2000	5-1000	
Igneous & Metamorphic Rocks	0.1-10	1-100	100-10000	Weathering decreases resistivity
Clays	25-250	250-2500	4-40	Claytype and moisture important
Clayey Soil (40 % clay)	125	1250	8	Clay % decrease increases resistivity

Important relationship related to site-specific resistivity value captured:

- i. Depth of study i.e. tomographic plot is equivalent to Electrical Resistivity Test (ERT) captured/reading or equivalent to $\frac{1}{2}$ of total takeout cable length (Maiti et al., 2013). Many other soil physical parameters, such its electrical resistivity, can be thought of as proxies for the geographical and temporal variability (i.e. structure, water content, or fluid composition). The technique provides a very attractive instrument for describing the subsurface features without excavating because it is non-destructive and extremely sensitive. It has already been used in a variety of situations, including groundwater research, landfill and solute transfer delineation, agronomical management by locating areas of excessive compaction or soil horizon thickness and bedrock depth, or at the very least, evaluating the soil hydrological qualities (Tan et al., 2018; Jinmin et al., 2018; Cheng et al., 2019).

- ii. Porosity = Archie's law (equation 2.13), equivalent to V_v/V theoretically equals to soil liquid limit. Upon proven, by experimental procedure, Archie's law is valid for used in soil not just rock in Table 2.4 (Glover, 2016).
- iii. Soil bearing capacity = Linear regressions equation between bearing capacity and relative compaction. $y = Ax + 87$, where x = allowable bearing capacity (kN/m²); y = relative compaction (% or n) and A = FoS
- iv. $V_s = 23 S_u^{0.475}$ (Wair et al., 2012)
- v. PGA = Seismic Hazard Map (Malaysia, 2017)
- vi. γ = Unit weight, kN/m³ (Zekkos et al., 2006)
- vii. Plasticity Index, see table 2.3 (Osman et al., 2013)

Table 2.3: Equations for Plasticity Index and Field/Laboratory Resistivity Values (<https://www.geotechdata.info>, 2021)

Sample Type	Test Location	Equation	Test Location	Equation
All soil samples	Lab.	$PI = -3.575 \ln(\rho) + 28.601$	Field	$PI = -3.056 \ln(\rho) + 24.597$
Silty sand	Lab.	$PI = 14.319e^{-0.001\rho}$	Field	$PI = 13.23e^{-0.001\rho}$
Sandy	Lab.	$PI = -1.007 \ln(\rho) + 8.6624$	Field	$PI = -0.602 \ln(\rho) + 5.572$

Table 2.4: Soil Porosity (<https://www.geotechdata.info>, 2021)

Description	Porosity	
	Low	High
Sand; Coarse	0.26	0.43
Sand; Fine	0.29	0.46
Sand/Gravel Sand; Well Graded; Little to No Fines	0.22	0.42
Sand/Gravel Sand; Poorly Graded; Little to No Fines	0.23	0.43
Silty Sand	0.25	0.49
Clayey Sand	0.15	0.37
Inorganic Silt/Silty Sand; Slight Plasticity	0.21	0.56
Gravel/Sandy Gravel; Well Graded; Little to No Fines	0.21	0.32
Gravel/Silty Sandy Gravel	0.15	0.22
Clayey Gravel/Clayey Sandy Gravel	0.17	0.27
Inorganic Silt; Uniform	0.29	0.52
Clay/Silty Clay/Sandy Clay; Low Plasticity	0.29	0.41
Organic Silt/Silty Clay; Low Plasticity	0.42	0.68
Silty Clay/Sandy Clay	0.20	0.64
Inorganic Silt; High Plasticity	0.53	0.68
Inorganic Clay; High Plasticity	0.39	0.59
Organic Clay; High Plasticity	0.50	0.75

2.5 Finding Gap and Advantages in a Simplified Analysis with ERT Data Acquisition to Determine Limit State of Soil Stability

It is possible to identify the hydrogeological conditions that are prevalent within the investigated area by correlating electrical resistivity with the available geologic information

(Rahardjo, 2014). Although porosity measurements lack the great accuracy of those performed in academic petro physics facilities, they are typically accurate and have a very good consistency to 0.5% i.e. 0.005 (Glover, 2016).

This study attempt to provide experimental evidence that porosity value, n is equal to volume of void (V_v) over total volume (V) of sample taken which is aligned with the soil phase relationship and the small percentage of uncorrelated data is due to human-error such; recording data error; disturbed sample; laboratory equipment not calibrated and uncleaned; human act of not diligent during recording captured data, predict result of test without proper test due to the familiarity and other human mistakes. So far no recorded literature of study that attempted to correlate liquid limit of soil (L_L) with soil porosity.

Additionally, it is noteworthy that electrical resistance, Ω , is equivalent to (conductivity)⁻¹ where based from Archie's law, porosity or L_L is consistent in electrical resistivity and electrical conductivity however actual reading at site-specific showing resistivity (at some points) are not the same with inverse conductivity or inverse sigma, $\Omega \neq [1/\Sigma]$. In a hypothetical situation, the volume of voids is consistent with electrical conductivity and resistivity, but tortuosity is different since it depends on the ratio of the curve's length (C) to the distance between its ends (L). And, hypothetically, electrical resistivity is equal to index volume of void in straight length (L) and electrical conductivity is equivalent to index of volume of void in curve length (C). Therefore, tortuosity (τ) value is attainable through this research where $\tau = C/L = [1/\Sigma]/\Omega$, or Σ^{-1}/Ω , or $\Omega/\Omega = 1$, when index of curve length = index of straight length. From this study, theoretically, water filling curve length void represent water content (w_c) and water filling straight length represent liquid limit (L_L), where $w_c/L_L > 1$ resultant with physical water discharged. Threshold value

of soil plastic state changing to liquid condition is a precursor to soil failure and difficult to determine at site-specific. And in simple term, where $w_c/LL > 1$, one can be identified the failure line which only inclinometer can detect when soil strata moved and will be too late.

In limit state methodology, where actual resisting force versus allowable resisting capability, is desirable in this study due to capitalization of minimized uncertainties and promoting safety. Actual data of site-specific is reliably attainable with electrical resistivity test with no human interaction nor possibility of tempering with the actual data during acquisition and/or processing. Site-specific data acquired shall be processed with seismic interaction or energy-based analysis to produce soil-amplification spectrum that comparable to Standard for identification of unstable soil.

The root cause of failure in geotechnical design is due to the information attained not site-specific, a few samples to represent all. The lowest band of weak soil layers may not represent the weakest segment of all sections. And the design to resist failure to that known segments may not be adequate for the other sections. Literature educated many of how to deal with weaknesses however the solution may not be the ideal answer because of the combination of site-specific situations and the affecting surrounding conditions. The needs to acquire underground comprehensive information is crucial to provide a better design. The literature provided us with Archie's law that measure soil porosity, resistivity, saturation exponent and cementation constant. Porosity in soil is linear to resistivity value and related to compaction degree of the soil. Resistivity value of soil is between 1 to 40 ohm.m and the correlation with degree of compaction and standard penetration test (SPT in blows) is achievable where the value is between 1 to 50 blows and a crucial criteria of strength in engineering properties, however the literature is yet to offer. The other important aspect in

failure criterion of soil is the liquidity index (LI) i.e., equivalent to water content in soil (w_c) over liquid limit of soil (LL) where the literature has provided us the experimental information of unstable soil consist value of LI of higher than 0.85 ($LI > 0.85$ is the limit of unstable soil index). The only area that the literature is yet to provide is that LL is equal to volume of void (V_v) over volume total (V_T) where it is governed by porosity that attainable via Electrical Resistivity Test (ERT) and conduction of current (conductivity) in soil is through the electrolyte contained in the pores i.e., related to w_c which also can be acquired with ERT.

This study is intended to fill the gap of knowledge that is important to ascertain by ERT with comprehensive imaging without having to interpolate cross bore-holes, data free from manual recording error, easily deployed, immediate results and tomographic image for future new findings interpretation. Better geotechnical design provision at site-specific to resist failure, counter-movement, filtering water to reduce pore pressure to the index of stable soil and identification of floating slab where borehole criteria to terminate but still layers of soil consist underneath which can induce delayed-failure. ERT, for this study, shall provide imaging for stable soil for foundation with SPT-N 50, rock layers, type of rock mass, weak soil layers and water content in soil

2.6 Chapter Summary

This chapter has summarized the previous works related to suffusion, site response PSHA, geotechnical correlations. The research gap was found from these literature on which this thesis has been compiled.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Overview

In this chapter, Electrical resistivity 2D imaging or tomography is chosen for the critical section soil data acquisition, proposed 30 meter in depth, due to its comprehensive tomography or underground mapping. Figure 3.1 shows the research methodology flowchart. Geophysical survey i.e. ERT for ground investigation should be extensive in researches, correlations, and interpretations so that incomplete investigation of soil can be avoided and other experienced geotechnical engineers and geophysicists can offer second opinion to the same tomography for better diagnostic of the site.

3.2 Methodology Flowchart

3.2.1 Collection of Data

Data for this research has been collected from various government and private agencies. The details are shown in below Table 3.1:

Table 3.1: Sources of Data for the Case Study

Data	Source
Seismic tectonic map of Malaysia (fault lines data), Geological Data	Minerals and Geoscience Department of Malaysia (DMGM) (JMG)
Borehole data	Sarawak Energy Bhd.
Earthquake time history records	PEER Ground Motion Database System.
Secondary data	Minerals and Geoscience Department of Malaysia (DMGM) (JMG)

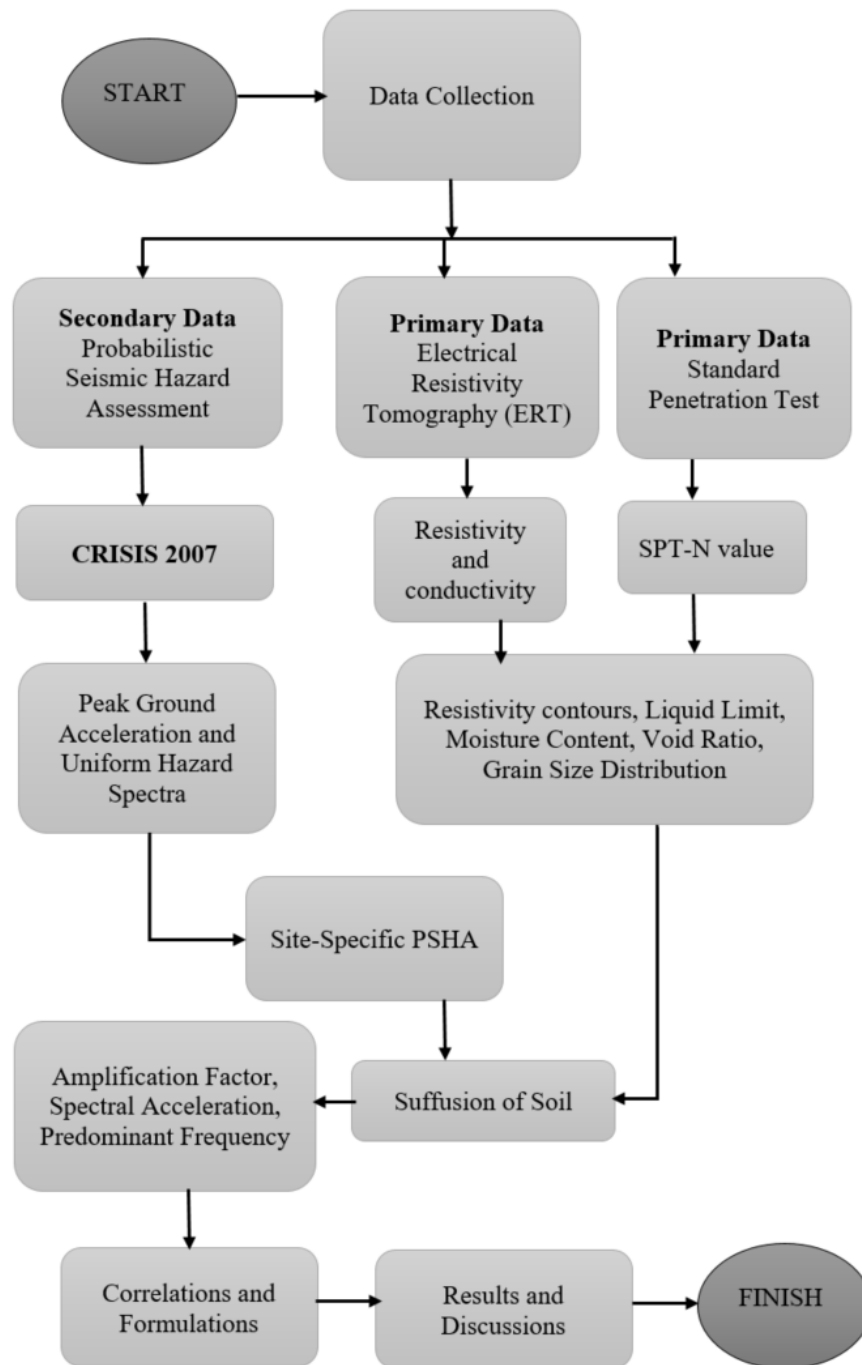


Figure 3.1: Flowchart of the Research

3.2.2 Field Investigations

Electrical resistivity tests (ERT) will be performed in the regional area of Sarawak and also at case study area which is at Pan Borneo Highway in order to develop the soil-specific relationship between resistivity of soil, moisture content, shear wave velocity and

N-value. Also, ERT will be conducted to get the resistivity of soil up to a certain depth which will be further used in order to study the suffusion of soil shown in Figure 3.2.



Figure 3.2: Field Investigation and Data Collection

The process of conducting an ERT involves the application of Ohm's law and the use of at least four electrodes. For this particular study, 20 stainless-steel electrodes were utilized, each with dimensions of 0.6 m in length and 0.012 m in diameter. These electrodes were placed at depths up to 0.45 m and were repositioned after each measurement.

The measurement process involved passing a direct current between the outer electrodes (C1 and C2), while the resulting potential drop was measured across the inner electrodes (P1 and P2). To determine the resistivity of the soil, the Wenner-Schlumberger array arrangement was employed, which involves the use of four electrodes for in-depth probing. This arrangement is symmetric and collinear, with the current electrodes placed farther apart than the potential electrodes. The arrangement enables the measurement of soil resistivity at a depth equivalent to $AB/3$ below the central electrode. The Wenner-Schlumberger array arrangement is considered to be faster and less prone to contact problems as only the current electrodes need to be repositioned after each reading. In this study, the Aimil stacked deep resistivity meter was used to conduct 6 ERTs at locations

corresponding to the SPTs, with both tests typically carried out at the same locations up to a depth of 30 m.

Previous research has shown that soil resistivity is influenced by various factors such as mineralogy, temperature, and moisture content (Hegde & Anand, 2022). Nonetheless, this study took precautions to minimize the impact of these factors. To reduce the influence of moisture and temperature, ERTs were conducted during the dry season. Furthermore, soil moisture and temperature levels at different locations varied by less than 3% and 1%, respectively. In addition, chemical analyses were performed to determine the pH, sulfate, and chloride content at various locations. The pH ranged from 4.36 to 6.78, while sulfate and chloride content varied from 0.02% to 0.07% and 0.01% to 0.04%, respectively. In addition, the organic content varied between 0.34% to 2.50%. Dafalla & AlFouzan, (2012) have suggested that the chemical properties of soil cause a less significant change in resistivity compared to particle size distribution. It should be noted that sulfate and chloride content in soil can result in a decrease in the soil's resistivity.

3.2.3 Laboratory Investigation of soil

The laboratory investigation of soil samples were carried out at Universiti Malaysia Sarawak (UNIMAS). The soil samples were collected depth wise during the SPT and ERT.

The investigations which were carried out are listed below:

1. Moisture content
2. Specific gravity
3. Dry density

4. Bulk density

3.2.4 Site Response Analysis

The results from phase II will be further used for the modelling of suffusion soft soil. Site response analysis will be carried out by using 1-D equivalent linear method in the DEEPSOIL.

3.2.5 Correlations and Formulations

The final phase comprise of all the phases from start to end. All the correlations, results and discussion, relationships between various parameters will be addressed in final phase.

3.3 Seismic-Hazard Source of Sarawak

Earthquake catalogue, for this research, is developed based on the records attained from various international seismological databases such as National Earthquake Information Centre, U.S. Geological Survey (NEIC-USGS), International Seismological Centre (ISC), Department of Mineral and Geoscience Malaysia and Malaysia Meteorological Department.

Fault lines data were supplied by the Department of Mineral and Geoscience Malaysia as shown in Figure 3.3. These information data are added to CRISIS2007 program. Line sources or fault lines, in general, polylines defined by the 3D coordinates of their vertexes. The active source as a collection of point sources i.e. coordinates of hypocentre from the earthquake catalogues. The fault length L (in km) is related to magnitude through:

$$L=K_3e^{K_4M} \quad \text{Equation 3.1}$$

where K_3 and K_4 values are empirical constant.

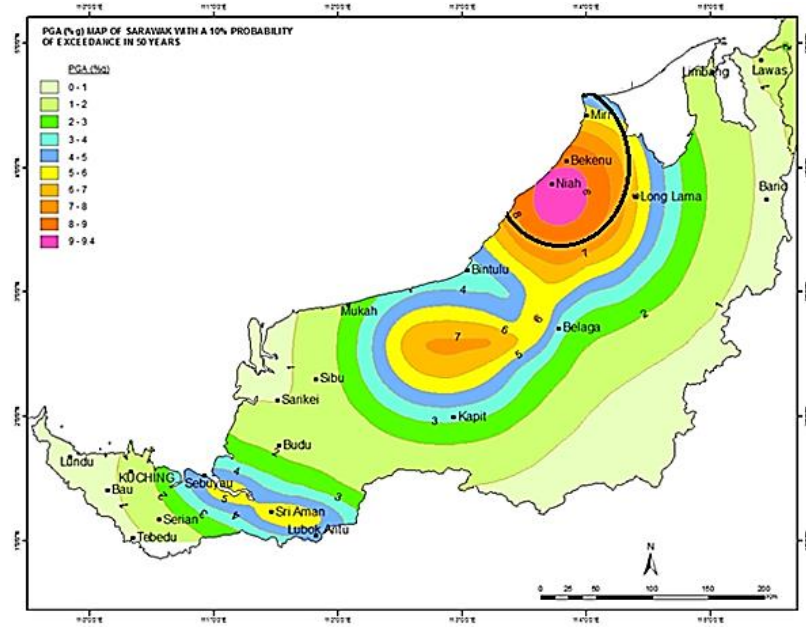


Figure 3.3: Seismic hazard map of Sarawak (JMG, 2021)

3.4 Site-Specific PSHA and Energy-Based Analysis

The site-specific study (case-study for this research), slope failure at Pantu, Sarawak (66km from Serian – Sri Aman bound of Jalan Serian-Sri Aman) is selected. This slope is within the Right of Way of the Pan Borneo Highway Sarawak with the design gradient 1:1.5 i.e. $\beta = 33.69^\circ$. Standard Penetration Test, SPT, was performed, and Imai and Tonouchi's equation was used to analyse the data as needed for this investigation (1982):

$$V_s = 97.0N^{0.314} \quad \text{Equation 3.2}$$

Peak ground acceleration at soil surface, peak ground velocity and peak ground displacement are later produced numerically from the excitation at bedrock via DEEPSOIL software with the PSHA result at the site of 0.016g (seismic hazard map produced by this study). PGA at bedrock of 0.03g that was ascertained from the seismic hazard map published by the Department of Mineral and Geoscience Malaysia, later tested for variant interaction of rock-soil or amplification factor.

Large-strain values are frequently related with failure of the soil surrounding the sampler or cone in penetration-based testing. The main factors affecting penetration-based tests, which are similar to small-strain soil properties, are the void ratio, confining stress, and stress history. The SPT has historically been the most extensively utilised in situ geotechnical test throughout the world. Since the 1960s, researchers have investigated the connection between V_s and SPT-N values. The largest dataset, which Imai and Tonouchi, (1982) analysed, contained 1654 data pairs from 386 borings at 250 sites across Japan. V_s correlation formulae based on N-value, soil type, and geologic age were created by Imai and Tonouchi. N-values were between one and nearly 400 blows per feet (bpf). In this research, Imai and Tonouchi (1982) equation for V_s and SPT relationship values is utilised for this research accordingly.

It is well acknowledged, particularly in high and moderate seismicity areas, that the underlying ground conditions have a significant impact on the earthquake vibration at the surface, and correspondingly, that the ground features have a significant impact on the seismic response of structures (Tang et al., 1999). The MS EN 1998-1 standard also takes into account the need for suitable site soil investigations to determine the ground's conditions. This will enable the classification of the soil profile with an eye toward defining the site-appropriate ground motion, enabling the selection of the pertinent spectral shape to calculate the PGA properly, and enabling the identification of potential occurrences of a soil behaviour during an earthquake, such as the peak ground displacement (PGD), and ultimately the soil amplification factor (AF). Sites are categorised according to the V_s of the top 30 metres of the soil profile by the Caltrans Seismic Design Criteria (V_{s30}). The six categories (Soil Profile Types A to F) listed in Table 3.2 are used to categorise sites. The American Society of Civil Engineers (ASCE 2006, 2010) and the National Earthquake

Hazard Reduction Program (BSSC 2003) both employ the site/soil classes developed by Caltrans, and also the Malaysia National Annex to Eurocode 8. Site Class F includes: (1) Soils vulnerable to failure or collapse under seismic loading (i.e., suffusive soils, liquefiable soils, quick and highly sensitive clays, and collapsible weakly-cemented soils). (2) Peat and/or highly organic clay layers more than 10ft (3m) thick. (3) Very high plasticity clay (PI > 75) layers more than 25 ft (8 m) thick. (4) Soft to medium clay layers more than 120 ft (36 m) thick.

The General Quadratic/Hyperbolic (GQ/H) model allows for the definition of the soil's shear strength at failure while also allowing for the representation of small-strain stiffness nonlinearity. The GQ/H model is compared to the old model for comparison, and total-stress case studies are used to illustrate the impact of accurately modelling soil shear strength. Both the initial shear stiffness (G_{max}) and the shear strength at failure (τ_{max}) of a material must be known in order to describe its behaviour under both small and large strains. These numbers represent the stress-strain behavior's linear bounds in the stress-strain domain. Because it is known that these linear bounds intersect at a particular reference shear strain, a quadratic model may be used to connect the two lines into a continuous curve. The shape of the backbone curve:-

$$\frac{\tau}{\tau_{max}} = \frac{2(\gamma/\gamma_r)}{1 + (\gamma/\gamma_r) + \sqrt{\{1 + (\gamma/\gamma_r)\}^2 - 4\theta_\tau(\gamma/\gamma_r)}} \quad \text{Equation 3.3}$$

Where τ is shear stress, τ_{max} is the shear strength at failure, γ is shear strain, γ_r is the reference shear strain, and θ_τ is a curve-fitting parameter. In this model, the reference shear strain maintains the original definition proposed by Kondner (1963) such that $\gamma_r = \tau_{max}/G_{max}$. The basic quadratic equation that forms the basis of the model is then simplified to a general

hyperbolic equation. As a result, the backbone curve is known as the GQ/H model and has a generic quadratic/hyperbolic form.

The generalised quadratic/hyperbolic (GQ/H) model can be used to depict both the soil's shear strength and small strain nonlinear behaviour. The model, which addresses shortcomings of a widely used model for nonlinear site response analysis, is a simplified one-dimensional shear stress-shear strain model. The proposed model's site response analyses show that the computed site response is quite sensitive to the soil model's inferred shear strength. Figure 3.4 show geology and shear wave velocity profiles. Using the GQ/H model and the friction angle observed by earlier researchers, the inferred shear strengths and friction angle of soil layers are adjusted.

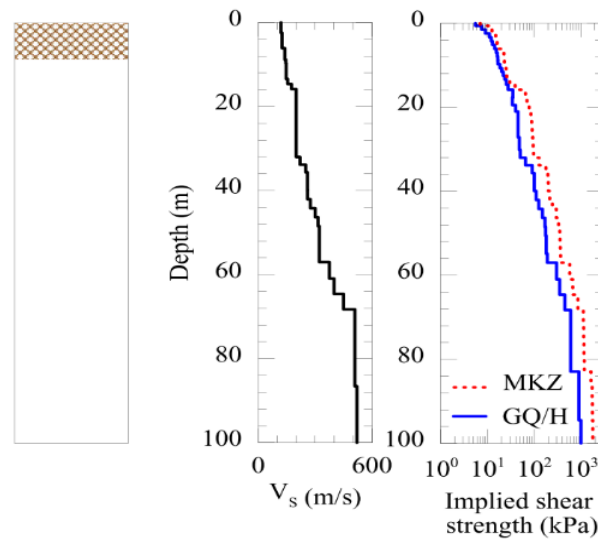


Figure 3.4: Profiles Of (A) Geology and (B) Shear Wave Velocity (V_s) of Station Apeel #2 (A02) in California, and (C) Implied Shear Strength for both MKZ and GQ/H Models

3.5 Shear Wave Velocity, V_{s30} , Via Electrical Resistivity Tomography (ERT)

Data acquisition for suffusion assessment start with electrical resistivity test (ERT). The type of equipment used for this task is from Geomative GD-10. The GD-10 is a

multipurpose direct current (DC) method instrument that is built using both digital and analogue technology. Self-potential (SP), apparent resistivity, and induced polarization (IP) surveys can all be conducted with this apparatus. Regular ERT survey setups and subsurface profiling using 2D and 3D survey arrays are supported by GD-10. The GD-10 ERT system can do in-field 2D cross-section profiling of ERT and IP. GD-10 is equipped with both centralized cabling and distributed cabling system as shown in Figure 3.5 and Figure 3.6. The soil data acquisition setup via electrical resistivity tomography is shown in Figure 3.7.

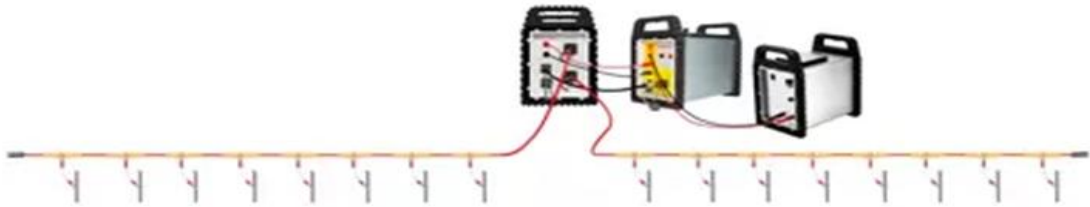


Figure 3.5: GD-10 ERT on Wenner-Schlumberger Configuration



Figure 3.6: Geomatic GD-10 Electrical Resistivity Scanning/Imaging System

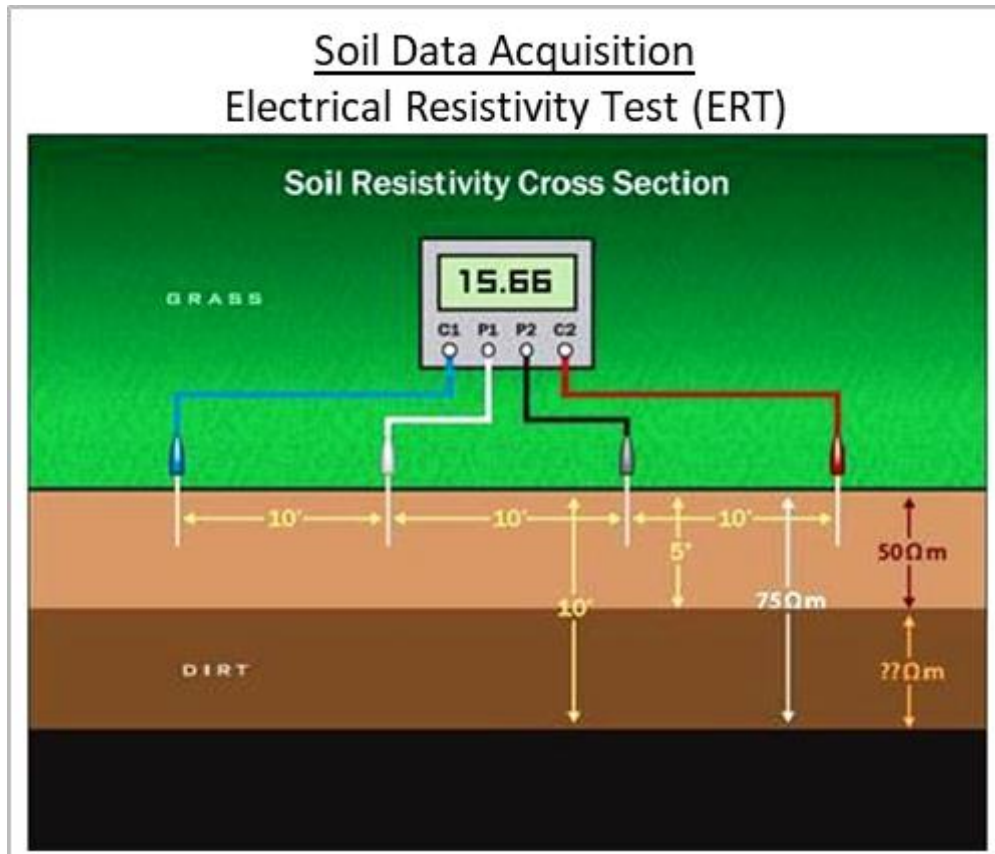


Figure 3.7: Soil Data Acquisition via Electrical Resistivity

The electrical resistivity tomography (ERT) is an investigation that measures the apparent resistivity, ρ , of a specific material by introducing electric current and voltage into the subsoil through various electrode pairs (referred to as "dipoles") driven into the ground surface. When interpreting the resistivity distribution in terms of soil lithology and saturation, it is important to keep in mind that, ρ , resistance rises as particle size, cementation, and moisture levels rise. The ERT is primarily used to characterize the layering and lithological characteristics of soil and rock deposits involved in natural geo-hazards, such as slope stability, seismic response assessments, and monitoring of waste disposal or polluted sites.



Figure 3.8: Electrical Resistivity Tomography for Site-Specific Suffusion Assessment

Geomative GD-10 Electrical System was used in this study to carry out soil data. It was created using the most recent digital and analogue circuitry techniques, multi-functional direct current (DC) method instrument. Geomative integrated both conventional centralized cabling system and distributed cabling system with infinite expand-ability of the cable system. Producing a sectional centralized ERT technique which attained both the rapidness of distributed cabling and the reliability of centralized cabling. The tomographic section produced 2 important reading; electrical resistivity (Ω), and electrical conductivity (Σ or Ω^{-1}). Correlation with borelog at site-specific was also carried out as illustrated in Table 3.2. The borehole location is shown in Figure 3.9.

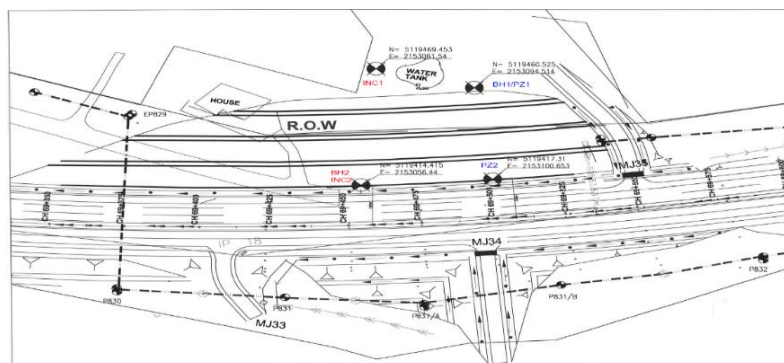


Figure 3.9: Boreholes Position and Numbering

Table 3.2: Site-Specific Data used in Correlation and Equation adopted from Guidelines for Estimation of Shear Wave Velocity Profiles by Pacific Earthquake Engineering Research Centre Headquarters at the University of California

Depth (m)	Ohm.m	ϕ	SPT-N	Soil Description	Resistivity Class	V_{s30}	S_u (kPa) $V_s = 23$ $S_u^{0.475}$
1.20	100.00	0.30	126	Clay	4 - 40	375.00	356.63
1.80	8.30	0.82	11	Clay	4 - 40	122.83	34.02
11.04	16.70	0.62	21	Clay	4 - 40	145.93	48.90
13.80	25.00	0.53	31	Clay	4 - 40	168.75	66.40
15.60	33.30	0.47	42	Clay	4 - 40	191.58	86.72
17.70	41.60	0.43	52	Clay	4 - 40	214.40	109.91
19.80	50.00	0.40	63	Clay	4 - 40	237.50	136.33
22.20	58.30	0.37	73	Clay	4 - 40	260.33	165.39
25.20	66.70	0.35	84	Clay	4 - 40	283.43	197.80
28.80	75.00	0.34	94	Clay	4 - 40	306.25	232.84

This study should be able to propose a simplified model to determine soil amplification from soil data properties captured at site-specific without having to perform finite element method and laboratory testing. This is desired for this research to produce. This simplified model is further simulated in FLAC Slope 7.0 (FEM), as illustrated in Figure

3.10 and Figure 3.11, to verify that amplification factor, f , if more than 1.0 (actual/allowable in design response spectra) equal unstable soil.

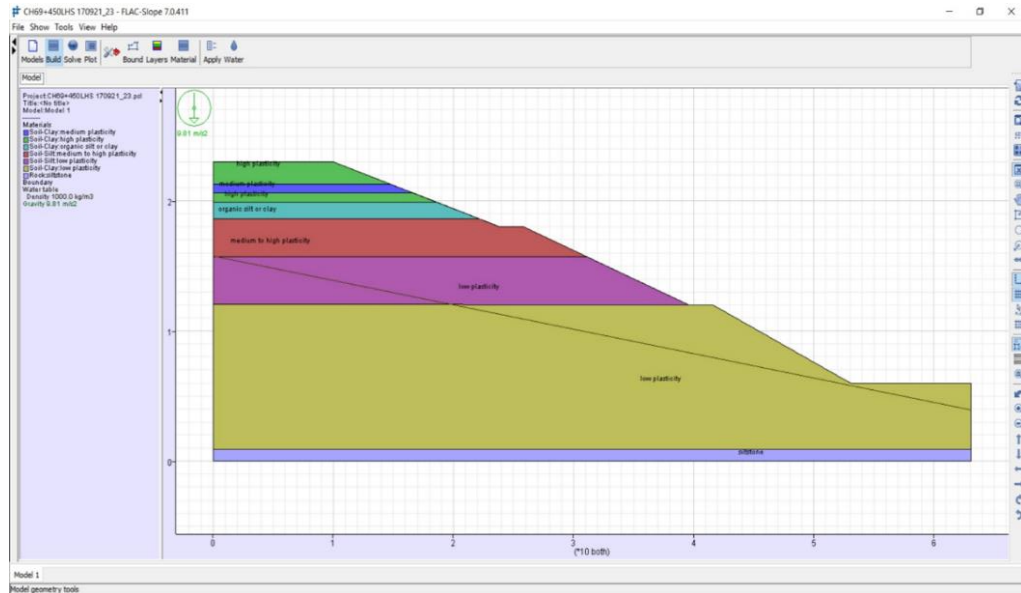


Figure 3.10: Simulation Model

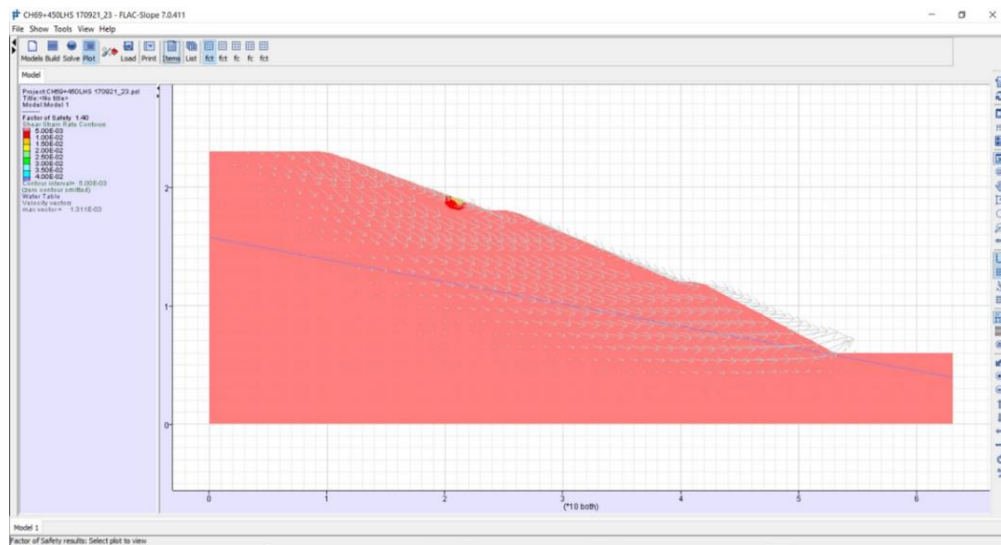


Figure 3.11: Simulation Output of Velocity Vector (Slope Gradient 1:2.5, FOS = 1.40)

3.6 Liquid Limit via Electrical Resistivity Tomography (ERT)

CH66+000 of Work Package Contract 03 of Pan Borneo Highway Sarawak in Pantu was selected to be the area of study in order to get the ERT and to determine the susceptibility

of suffusion by incorporating the LL with ERT. The detail location is shown below in Figure 3.12, Figure 3.13 and Figure 3.14.

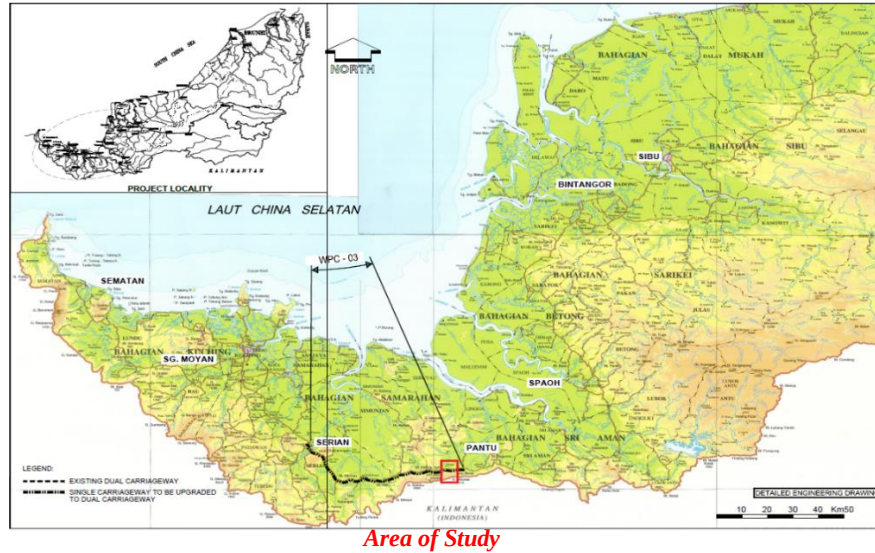


Figure 3.12: Map of Sarawak and Area of Study

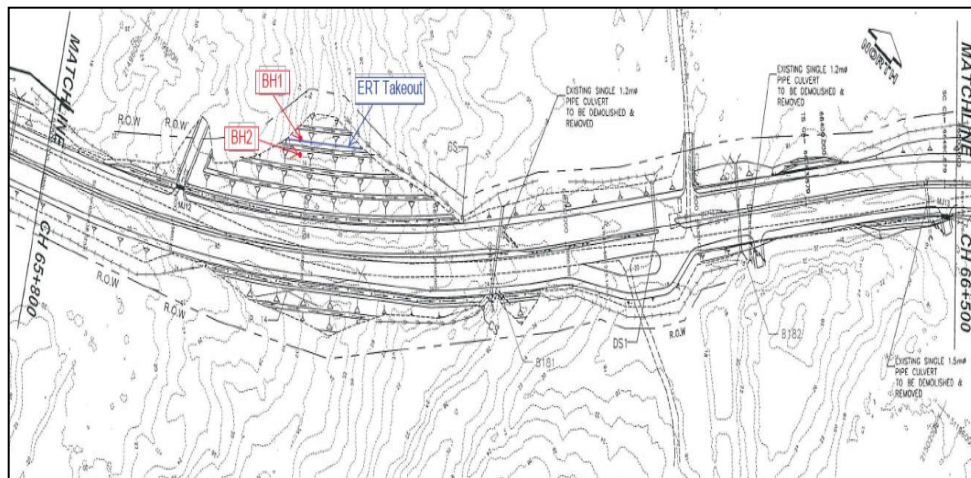


Figure 3.13: Plan and Profile of the Area of Study – Test Site 1

The liquid limit of a soil is the moisture level at which it changes from a plastic to a liquid state. Utilizing the liquid limit, cohesive fine-grained soils can be identified and categorized accordingly. Changes in a soil's moisture content can have a considerable impact on its shear strength, especially in fine-grained soils.

The moisture content of a soil is defined as the amount of water that can be removed from the pore space between soil grains by oven drying at a temperature no greater than 110 °C. The moisture content has a big impact on how the soil behaves. It is the percentage of a soil's dry mass that is composed of the water content of the soil.

To calculate moisture content of a soil, below formulation is presented as according to BS 1377: Part 2: 1990.

$$w = \left(\frac{m_2 - m_3}{m_3 - m_1} \right) \times 100 \quad \text{Equation 3.4}$$

Where, m_1 is the mass of the container (in gm), m_2 is the mass of the container with wet soil (in gm), m_3 is the mass of the container with dry soil (in gm)

The Casagrande test is mainly a static test that depends on soil shear strength, hence the cone penetrometer approach is selected. The liquid limit of a sample in its natural state or a sample from which material that was retained on a 425 mm test sieve has been removed can both be determined using this method. It is based on the measurement of a standard cone's penetration into the soil. Detail procedure of cone penetrometer is provided in BS1377: Part 2: 1990.

The value of cementation exponent (m) is said to increase with decreasing permeability (Archie, 1942) which $m = 2.5$ is strongly related to low permeability soil i.e., clay. This constant will be used to determine the liquid limit for test site 2 and 3 then later compare to the laboratory test result via cone penetrometer to confirm the correlation.

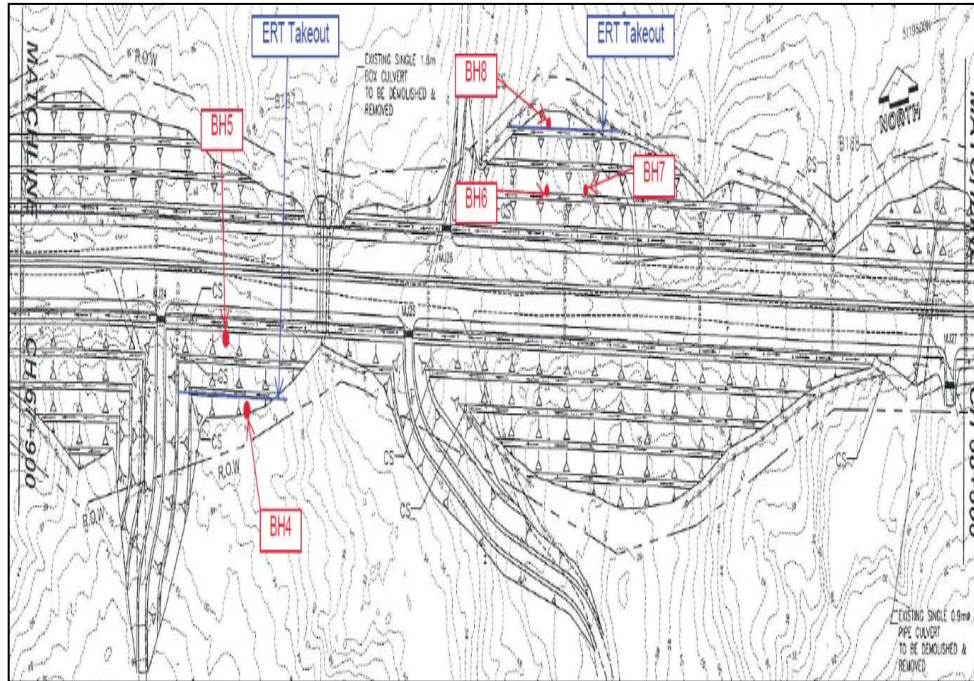


Figure 3.14: Plan and Profile of the Area of Study – Test Site 2 & 3

As a result of the water filling the pore space between the soil grains, which is a gap that can only be inhabited by air or water, the study's hypothesis states that the porosity of soil is equal to its liquid limit. From the above results it is 96.2% of soil porosity are the same with its liquid limit. The factor contributing to the variance is pointing towards the constant, a , i.e., tortuosity because of different soil particle size distribution resulting to different matrix of path length of the hydraulic conductivity. Nevertheless, this study has successfully proved the hypothesis is strongly manifest correlation between soil porosity and its liquid limit

3.7 Chapter Summary

The chapter includes research methodology and the methodology of all the objectives is explained one by one. By applying the methodology, there results were established which are explained in chapter 4 which is results and discussion.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Overview

This chapter offers a comprehensive examination of various geotechnical and geological aspects in the study area. Site-specific PSHA, suffusion, amplification, ERT have been discussed and examined. This chapter plays a pivotal role in the research, offering the key findings and interpretations of the study's results related to site response, suffusion, amplification, and ERT. It's a critical part of any geotechnical or geological research, as it allows for the discussion of how these findings contribute to a better understanding of the site's characteristics and potential risks, which can inform engineering and planning decisions

4.2 Electrical Resistivity Test Result for Test Site 1, 2 and 3

Subsequent to the soil movement detected on 3 slopes at CH66+000, CH67+800 and CH68+300 of Pan Borneo Highway Sarawak Package 3 from Serian Roundabout to Pantu Junction electrical resistivity and conductivity imaging was immediately carried out to quickly investigate and further understand the soil and rock condition under the changes. The movement was detected in a slope shown in Figure 4.1. The field work i.e. ERT was carried out and the data was interpolated in ResIPY software shown in Figure 4.2 to Figure 4.12. In addition, the SPT data was collected from different agencies and the laboratory work was carried out in UNIMAS. All the results have been tabulated in Table 4.1, Table 4.2 and Table 4.3. In addition, the summarized results of all locations have been shown in Table 4.4.



Figure 4.1: Movement Detected on Slope at Test Site 1. This Situation similarly Experienced at Test Site 2 And 3. ERT was Carried out on the Top Bench of the Slope

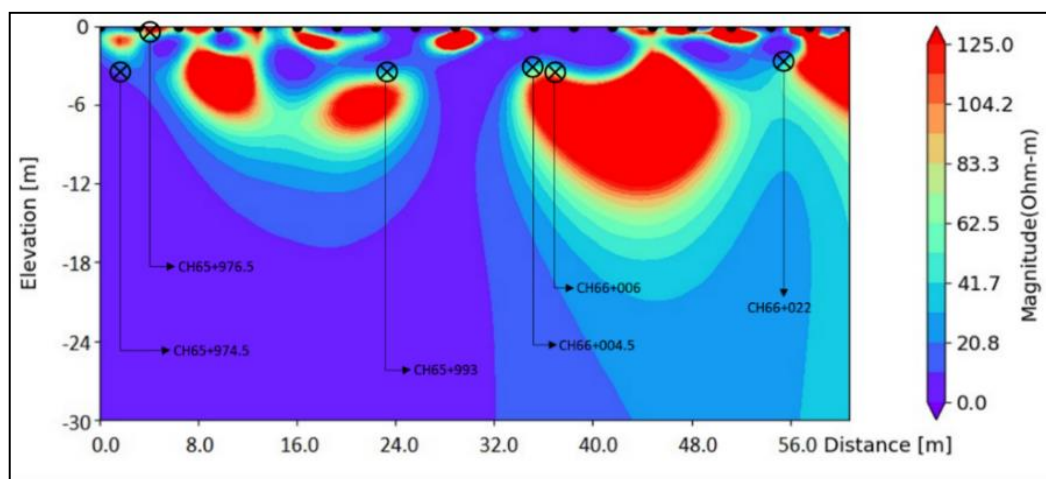


Figure 4.2: Electrical Resistivity Tomography of the Area of Study – Test Site 1

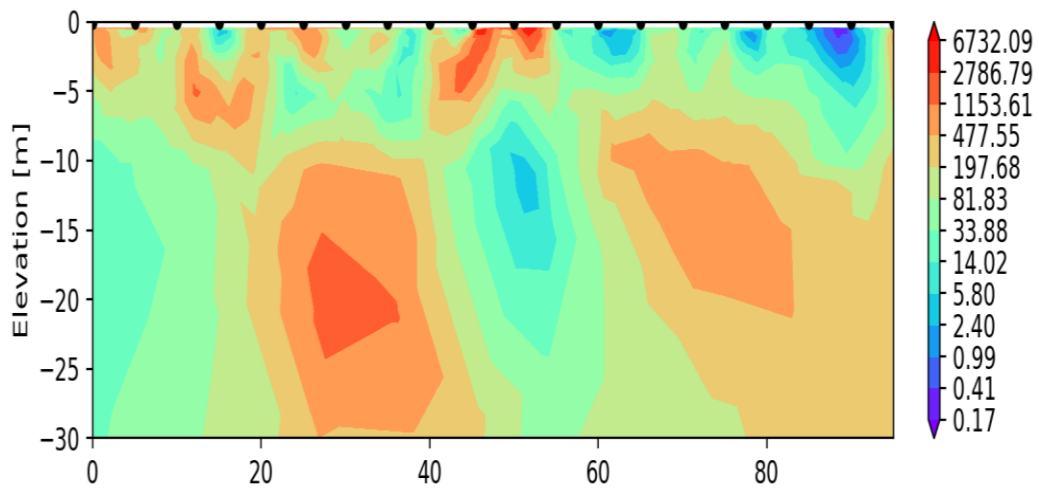


Figure 4.3: Detail Imaging of ERT CH66+800

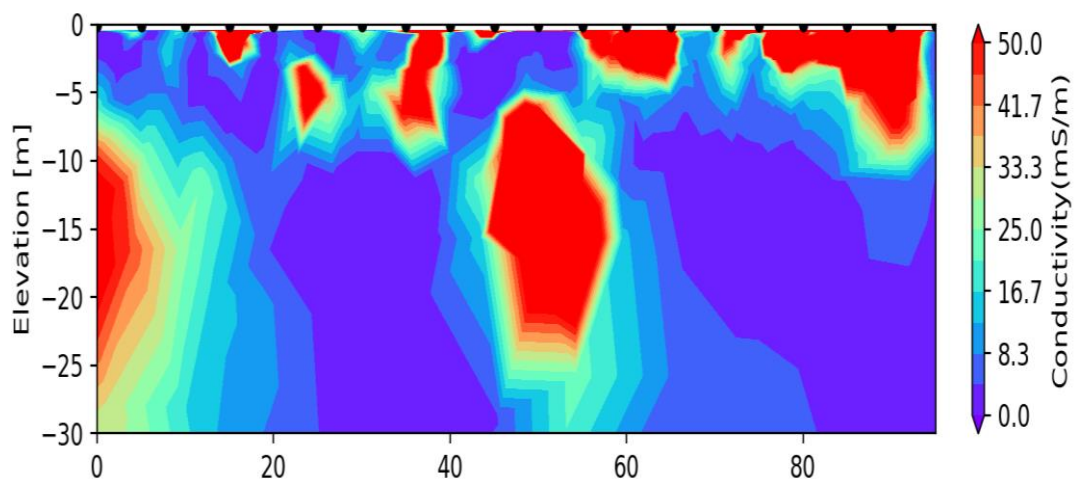


Figure 4.4: Detail Conductivity Reading/Imaging of Slope Test Site 1

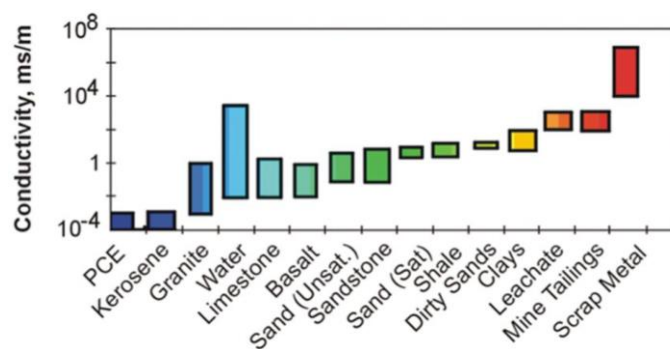


Figure 4.5: Conductivity Ranges of Various Materials (<http://www.cflhd.gov>, 2021)

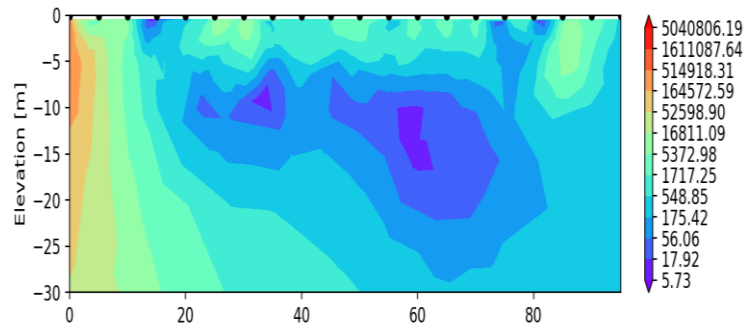


Figure 4.6: Detail Imaging of ERT at CH67+800

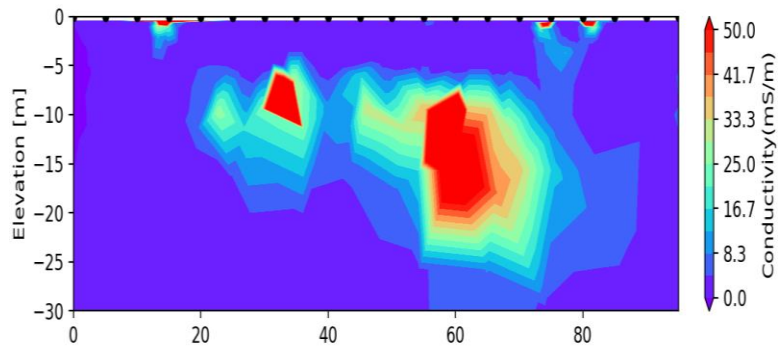


Figure 4.7: Detail Conductivity Reading/Imaging of Slope CH67+800

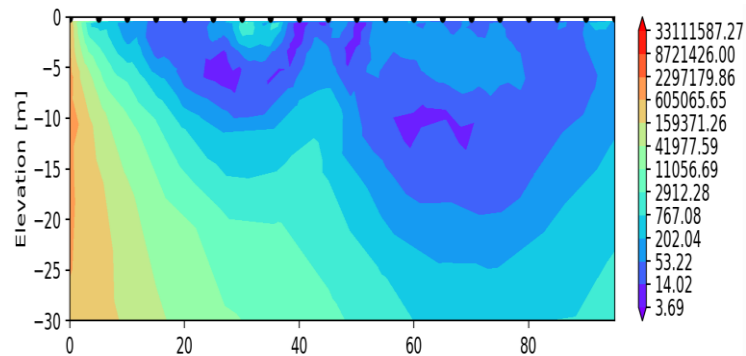


Figure 4.8: Detail Imaging of ERT at CH68+300

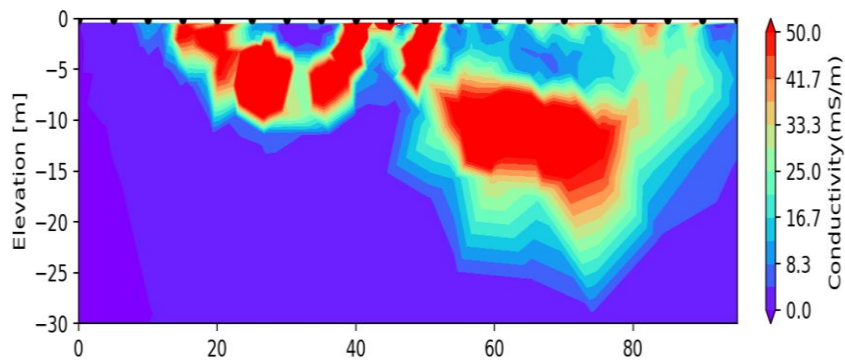


Figure 4.9: Detail Conductivity Reading/Imaging of Slope CH68+300

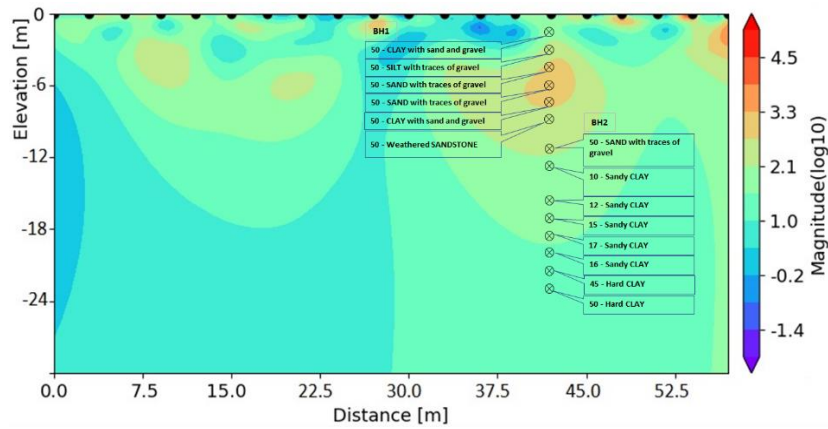


Figure 4.10: Electrical Resistivity Tomography (ERT) for Borehole 1 & 2

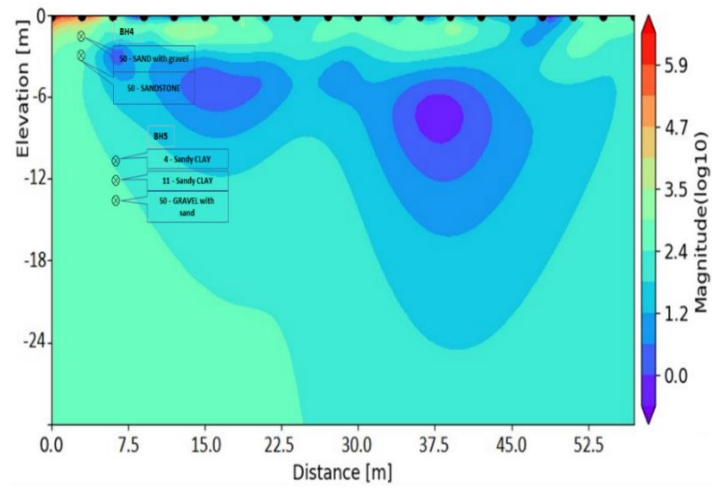


Figure 4.11: Electrical Resistivity Tomography (ERT) for Borehole 4 & 5

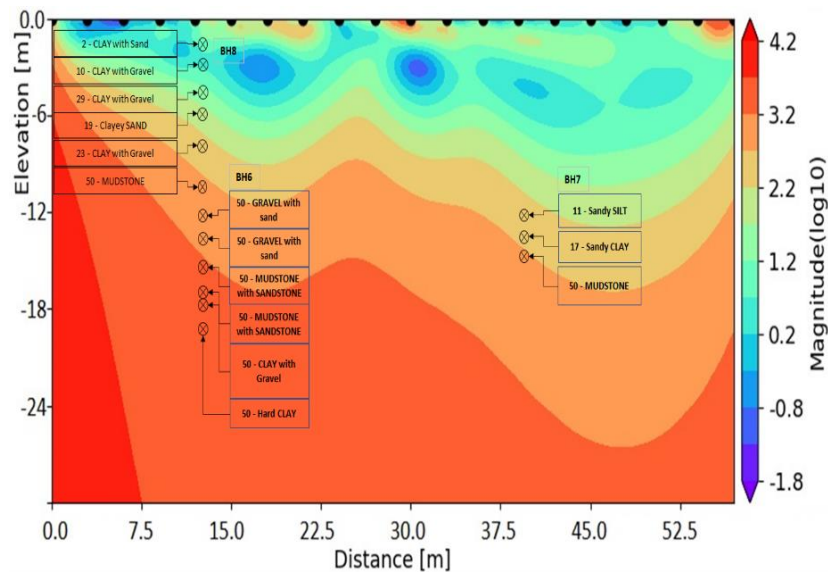


Figure 4.12: Electrical Resistivity Tomography (ERT) for Borehole 6, 7 & 8

Table 4.1: Results of Soil Properties Data of BH1 and BH2

Depth	BH1	BH1								
0.20	Vs	SPT-N	$\Omega.m$	$\Omega.m.m$	σ	$\phi, [V_u/V_t]$	$e, [V_u/V_s]$	e_{min}	$\Delta e/e_{min}$	
2.00	340.50	55	79.43	142.98	12.59	0.33	0.49	0.28	179	
3.40	338.58	54	501.19	701.66	2.00	0.16	0.19	0.19	100	
5.00	336.70	53	1,995.26	3,192.42	0.50	0.09	0.10	0.10	100	
6.40	336.70	53	1,995.26	2,793.37	0.50	0.09	0.10	0.10	100	
8.00	340.50	55	1,995.26	3,192.42	0.50	0.09	0.10	0.10	100	
9.00	350.84	60	501.19	501.19	2.00	0.16	0.19	0.19	100	
30.00	360.00		39.81	836.03	25.12	0.44	0.77	0.46	169	
	356.24		1,015.34	378.67			$(\sum \Delta e/e_{min})_{30}^{10}$		107	
Depth	BH2	BH2								
0.20	Vs	SPT-N	$\Omega.m$	$\Omega.m.m$	σ	$\phi, [V_u/V_t]$	$e, [V_u/V_s]$	e_{min}	$\Delta e/e_{min}$	
1.80	436.15	120	6,309.57	10,095.32	0.16	0.06	0.06	0.06	100	
5.00	199.88	10	199.53	638.48	5.01	0.23	0.30	0.15	202	
6.40	211.66	12	100.00	140.00	10.00	0.30	0.43	0.24	184	
8.00	227.02	15	63.10	100.95	15.85	0.36	0.57	0.23	244	
9.40	236.12	17	63.10	88.33	15.85	0.36	0.57	0.23	244	
11.00	231.67	16	50.12	80.19	19.95	0.40	0.66	0.30	220	
12.40	320.54	45	39.81	55.74	25.12	0.44	0.77	0.38	202	
14.00	334.87	52	39.81	63.70	25.12	0.44	0.77	0.38	202	
15.40	342.46	56	39.81	55.74	25.12	0.44	0.77	0.38	202	
16.80	357.72	64	39.81	55.74	25.12	0.44	0.77	0.38	202	
18.40	357.72	64	39.81	63.70	25.12	0.44	0.77	0.38	202	
19.80	362.64	67	39.81	55.74	25.12	0.44	0.77	0.38	202	
30.00	362.65		39.81	406.07	25.12	0.44	0.77	0.38	202	
	300.65		543.39	396.66			$(\sum \Delta e/e_{min})_{30}^{10}$		111	

Table 4.2: Results of Soil Properties Data of BH4 and BH5

Depth		BH4								
0.20	Vs	SPT-N	$\Omega.m$	$\Omega.m.m$	σ	$\phi, [V_u/V_t]$	$e, [V_u/V_s]$	e_{min}	$\Delta e/e_{min}$	
2.00	340.50	55	3,162.28	5,692.10	0.32	0.08	0.08	0.08	1.00	
3.00	350.84	60	1,000.00	1,000.00	1.00	0.12	0.14	0.14	1.00	
30.00	360.00		1,000.00	27,000.00	1.00	0.12	0.14	0.14	1.00	
	360.85		1,720.76	1,123.07				$(\sum \Delta e/e_{min})_{30}^{10}$	1.04	
Depth		BH5								
0.20	Vs	SPT-N	$\Omega.m$	$\Omega.m.m$	σ	$\phi, [V_u/V_t]$	$e, [V_u/V_s]$	e_{min}	$\Delta e/e_{min}$	
2.00	149.91	4	25.12	45.21	39.81	0.52	1.10	0.67	1.64	
3.40	205.95	11	100.00	140.00	10.00	0.30	0.43	0.24	1.84	
5.00	342.46	56	251.19	401.90	3.98	0.21	0.26	0.13	2.10	
30.00	360.00		125.89	3,147.31	7.94	0.28	0.38	0.20	1.89	
	322.85		125.55	124.48				$(\sum \Delta e/e_{min})_{30}^{10}$	1.07	

Table 4.3: Results of Soil Properties Data of BH6, BH7 and BH8

BH6									
Depth									
0.20	Vs	SPT-N	Ω_m	$\Omega_{m,m}$	σ	$\phi, [V_v/V_v]$	$e, [V_v/V_v]$	e_{min}	$\Delta e/e_{min}$
2.00	331.32	50	50.12	90.21	19.95	0.40	0.66	0.38	1.72
3.40	331.32	50	501.19	701.66	2.00	0.16	0.19	0.08	2.48
5.20	331.32	50	5,011.87	9,021.37	0.20	0.06	0.07	0.07	1.00
6.80	331.32	50	5,011.87	8,019.00	0.20	0.06	0.07	0.07	1.00
7.60	331.32	50	5,011.87	4,009.50	0.20	0.06	0.07	0.07	1.00
9.20	331.32	50	5,011.87	8,019.00	0.20	0.06	0.07	0.07	1.00
30.00	360.00		15,848.93	329,657.78	0.06	0.04	0.04	0.04	1.00
	353.18		3,000.00	3,000.00				$(\sum \Delta e/e_{min})_{S30}^{1/3}$	1.08
BH7									
Depth									
0.20	Vs	SPT-N	Ω_m	$\Omega_{m,m}$	σ	$\phi, [V_v/V_v]$	$e, [V_v/V_v]$	e_{min}	$\Delta e/e_{min}$
2.00	205.95	11	15.85	28.53	63.10	0.63	1.71	1.05	1.63
3.40	236.12	17	50.12	70.17	19.95	0.40	0.66	0.38	1.72
4.60	331.32	50	158.49	190.19	6.31	0.25	0.34	0.17	1.95
30.00	360.00		5,011.87	127,301.56	0.20	0.06	0.07	0.07	1.00
	337.66		3,000.00	3,000.00				$(\sum \Delta e/e_{min})_{S30}^{1/3}$	1.06
BH8									
Depth									
0.20	Vs	SPT-N	Ω_m	$\Omega_{m,m}$	σ	$\phi, [V_v/V_v]$	$e, [V_v/V_v]$	e_{min}	$\Delta e/e_{min}$
2.00	120.59	2	158.49	285.28	6.31	0.25	0.34	0.17	0.95
3.40	199.88	10	15.85	22.19	63.10	0.63	1.71	1.05	1.63
5.00	279.23	29	50.12	80.19	19.95	0.40	0.66	0.38	1.72
6.40	244.51	19	158.49	221.89	6.31	0.25	0.34	0.17	1.95
8.00	259.63	23	501.19	801.90	2.00	0.16	0.19	0.08	2.48
11.00	331.32	50	1,584.89	4,754.68	0.63	0.10	0.11	0.11	1.00
30.00	360.00		5,011.87	95,225.57	0.20	0.06	0.07	0.07	1.00
	295.91		3,000.00	3,000.00				$(\sum \Delta e/e_{min})_{S30}^{1/3}$	1.08

Table 4.4: Summary of Field Data and Analysis

BH		1	2	4	5	6	7	8
S_a/PGA		4.00	4.63	3.50	4.25	4.13	4.75	3.70
$AF, (S_a(T_0))$		1.60	1.85	1.40	1.70	1.65	1.90	1.48
$(\sum \Delta e/e_{min})_{S30}^{1/3}$		1.07	1.11	1.04	1.07	1.08	1.06	1.08
V_{S30}		356.24	300.65	360.85	322.85	353.18	337.66	295.91
$SF, (a = 1.6)$		2.48	2.69	2.21	2.57	2.42	2.67	2.30
$a_s SF$		3.97	4.31	3.54	4.12	3.87	4.28	3.68

4.3 Site-Specific PSHA

Case-study area of WPC03 of the development and upgrading of the proposed Pan Borneo highway in the state of Sarawak, Malaysia is 74.915 km in length and stretches from CH 90+500 to CH 74+415 (Serian roundabout to Pantu Junction) and Pantu Section stretch is from CH 33+050 to CH 74+415. Most of Pantu section is a hilly area and there are many cut-slopes in this stretch, mainly from CH 50+000 to CH 73+000. The fault lines map is shown in Figure 4.13 and the recent earthquakes in Sarawak is given in Table 4.5 also with the GMPE's shown in Figure 4.14. Ground motion based on a probabilistic seismic hazard map where 10% of events are likely to occur at bedrock in 50 years period with PGA 0.11g and 0.14g shown in Figure 4.15 and Figure 4.16.

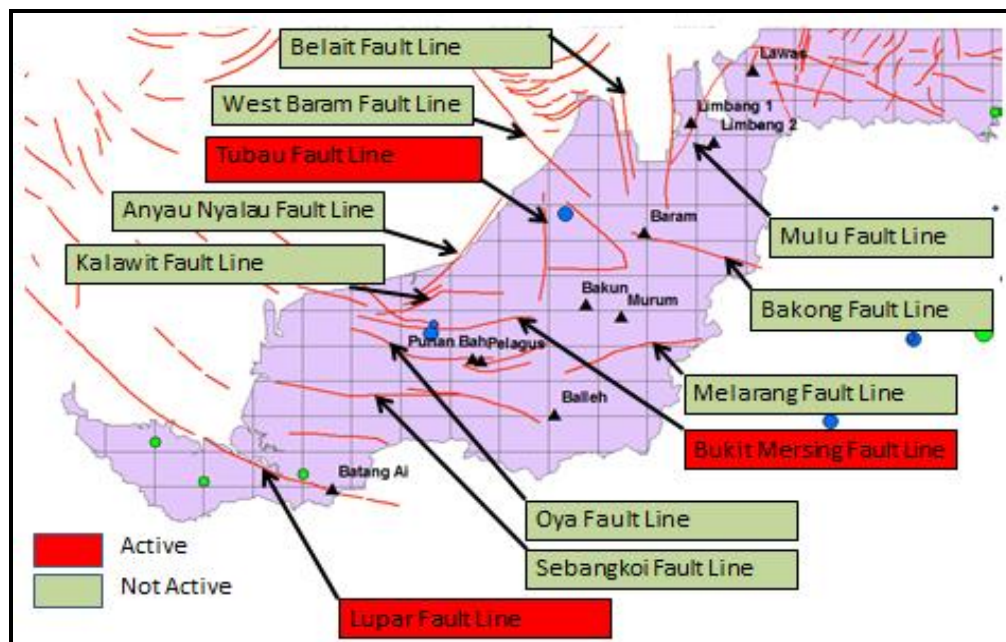


Figure 4.13: Fault Lines within and Surrounding Sarawak and 3 Active Fault Lines

Table 4.5: List of Observation Data at 6 Stations in East Malaysia after Mw= 4.1, Sebuyau, Sarawak Earthquake, 2011, (Malaysian Meterological Department)

Station of Observation	Latitude	Longitude	Distance from Epicentre (km)	PGA (g)
BTM Bintulu	3.20N	113.08E	312.46	0.000088
KKM Kota Kinabalu	6.04N	116.21E	778.99	0.000010
SBM Sibu	2.45N	112.21E	202.20	0.000019
KSM Kuching	1.47N	110.31E	98.83	0.000185
TSM Tawau	4.29N	117.87E	843.50	0.000005
SPM Sapulut	4.70N	116.47E	725.73	0.000103

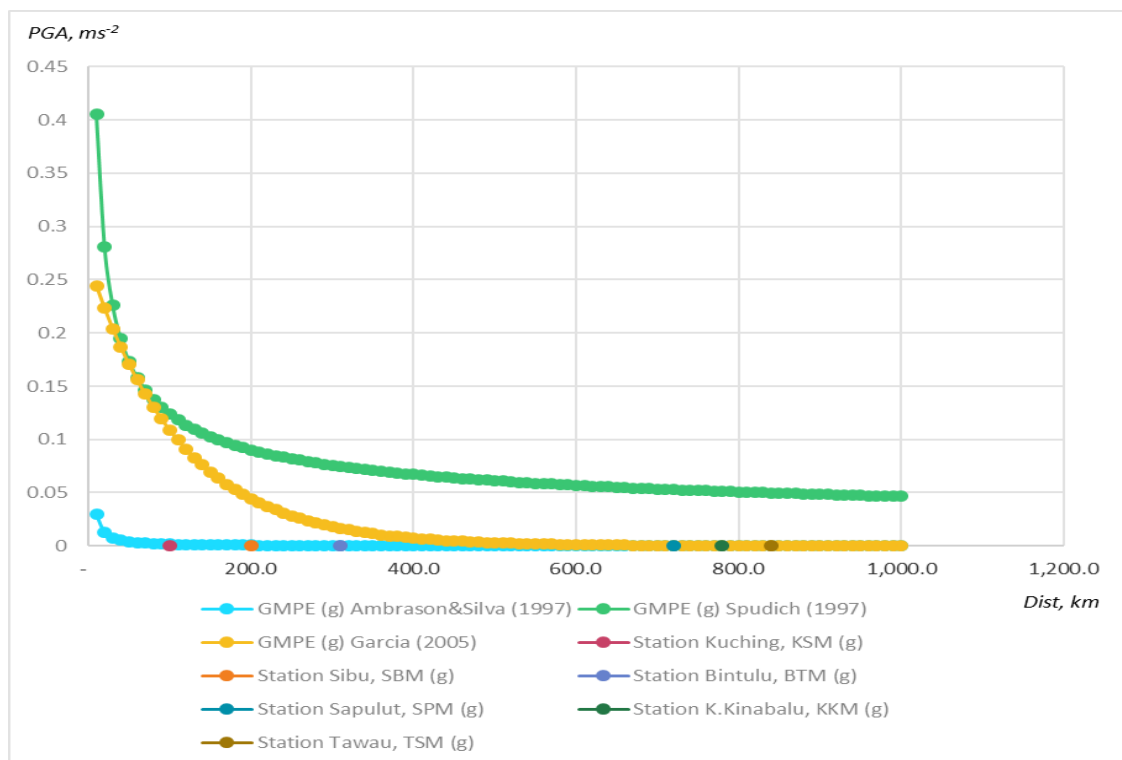


Figure 4.14: Analysis of GMPE vs Actual Ground Motion Captured

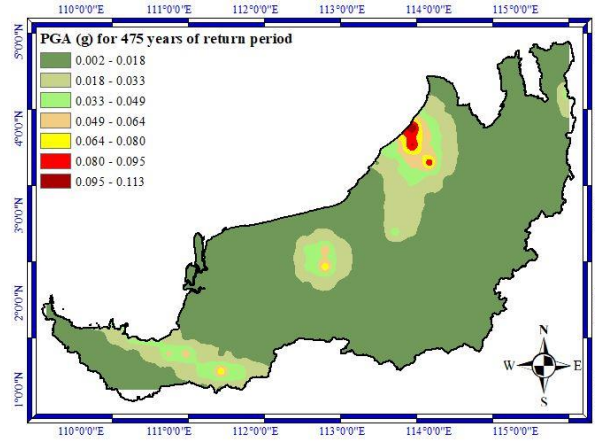


Figure 4.15: Value of PGA Attained, 11%g (Bedrock Level) from CRISIS2007 for the Area of Study corresponding to 475 Years of Return Period

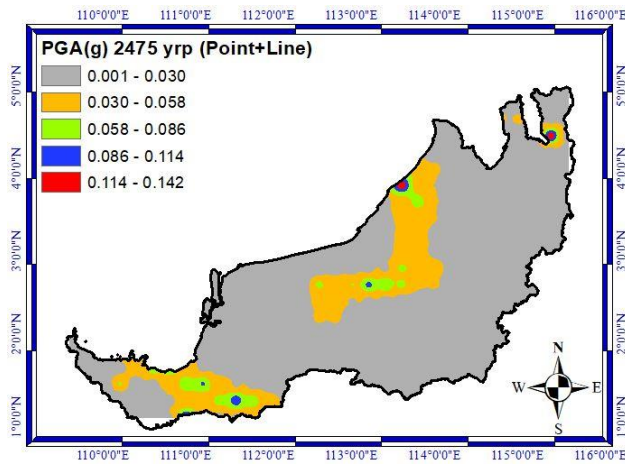


Figure 4.16: Value of PGA Attained, 14%g (Bedrock Level) from CRISIS2007 for the Area of Study corresponding to 2475 years of Return Period

4.4 Proposed Soil Instability Index

The relationship between conductivity and resistance is inverse. Resistivity is high when conductivity is low and vice versa. Since conductivity is recorded in Siemens ($1/\Omega\text{m}$) and resistivity in Ohm-meters (Ωm), the porosity (ϕ) in both should be equal. However, when water is present and filling the volume of the void (V_v) during data collection, ϕ conductivity / ϕ resistivity equals water content (w_c) / liquid limit (LL). Where $w_c/LL > 1$ indicate the baseline of soil instability and good methodology of identification of critical hydraulic load and site-specific material susceptibility to failure. The hypothesis has been

proven by this case study of soil failure at the high-level water tank Pantu Section of Pan Borneo Highway Sarawak shown in Figure 4.17 and Figure 4.18, where soil instability index value more than 1, the soil is unstable. This research has acquired ERT data for this particular segment at the high-level water tank at CH69+450 Pantu Section of the Pan Borneo Highway Sarawak in January 2021 due to landslide failure as per Figure 4.20, Figure 4.21 and Figure 4.22 – incident report by JKR Sarawak. ERT data take out was carried out 2 meters from the collapsed line top slope for best identification. The majority of the subterranean data collected at that location indicated that the materials are prone to failure where electrical conductivity value against resistivity reading $(1/\sigma)/\Omega$ exceeded the threshold value of 1 from the proposed soil instability index. $(1/\sigma)/\Omega = w_c/LL =$ liquidity index (LI), where for stability of soil the LI must not exceed the value of 1.

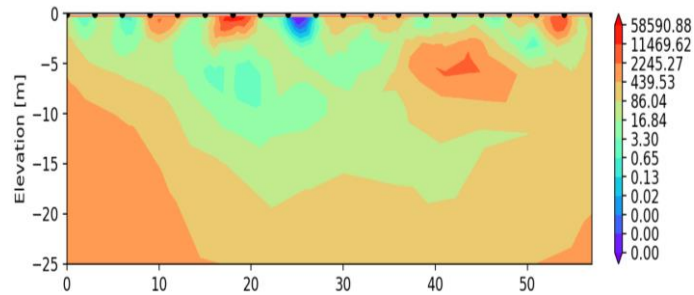


Figure 4.17: Electrical Resistivity Test Imaging for High-Level Water Tank Pantu Section of Pan Borneo Highway Sarawak

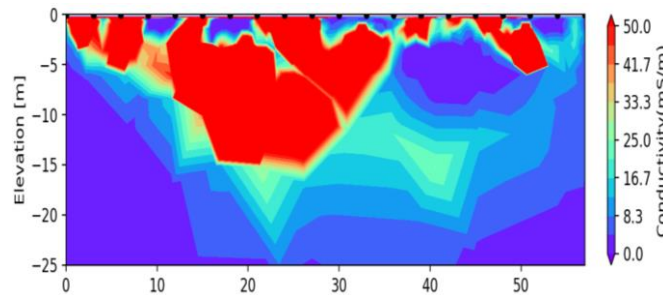


Figure 4.18: Electrical Conductivity Imaging for High-Level Water Tank Pantu Section of Pan Borneo Highway Sarawak

According to Bray (2006) the author of assessment of the liquefaction susceptibility of fine-grained soils, the soil indexes of the fine-grained soils with a collapse failure characteristic exhibit a certain regularity in the clay content, the Atterberg limits, and the water content. In fact, the clay content, the plasticity index, and the liquid limit of the studied fine-grained soils range from 3 to 20%, from 0 to 9.6, and from 22.5 to 36. In addition, the ratio of the water content of the fine-grained soils to their liquid limit is generally larger than 0.85. Correlation between SPT-N and electrical resistivity is above average is shown in Figure 4.19.

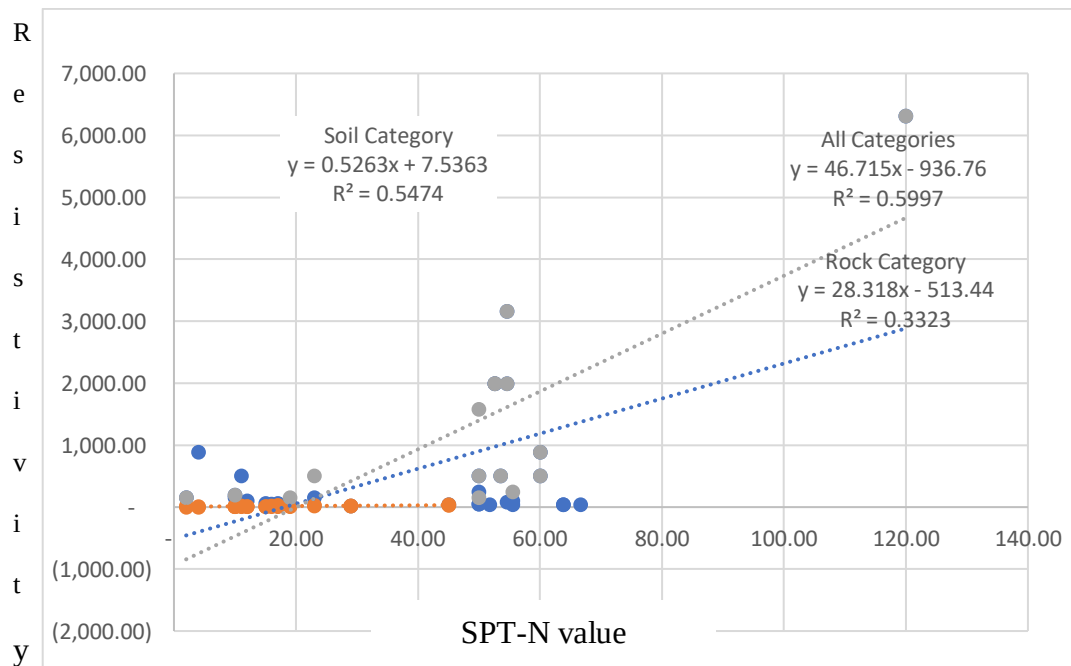


Figure 4.19: Correlation between SPT-N and electrical resistivity is above average



INCIDENT REPORT (LANDSLIDE / SLOPE FAILURE DATA BANK)

(FILL IN BOXES OR MARK APPROPRIATE ANSWER)

State & District	Route No: (Road only)	Slope ID: (Office use only)	Occurrence No: (Office use only)																										
STEP 1 INSPECTION DETAILS: 1.1 INSPECTED BY: JKR PBHU, PMSB, ZECON KIMLUN CONSORTIUM SDN BHD, ICE																													
1.2 DATE & TIME:		DATE: 31 / 12 / 2020	TIME: 10.00 AM / PM																										
1.3 WEATHER:		☒ SUNSHINE ☐ CLOUDY ☐ LIGHT RAIN ☐ HEAVY RAIN																											
STEP 2 GENERAL INFORMATION:																													
2.1 LOCATION NAME:		CH69+400 LHS																											
2.2 SECTION NO: (Road only)	PANTU	2.4 FEATURE TYPE:	☐ EMBANKMENT ☒ CUT																										
2.3 DISTANCE (m): (Road only)			☐ NATURAL SLOPE ☐ OTHERS																										
2.5 COORDINATE: (WGS 84)	NORTHING : ° EASTING : °	2.6 ELEVATION : 34.817 m	2.7 FEATURE ASPECT: 0																										
2.8 ROAD SIDE:	☒ LEFT ☐ RIGHT	2.9 AGE: (if known) -																											
2.10 DATE FAILURE OCCURRED:	Dec-20	2.11 ESTIMATED TIME FAILURE OCCURRED:	TIME: 8:00 AM / PM																										
STEP 3 STATUS OF FEATURE																													
3.1 SEVERITY CODE																													
<table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th colspan="2">CUTS and NATURAL SLOPES</th> </tr> </thead> <tbody> <tr><td>1</td><td>Minor shallow failure (<5m in length)</td></tr> <tr><td>2</td><td>Localised surficial loosening, localised evidence of fallen material (rock) at base of slope</td></tr> <tr><td>3</td><td>General evidence of fallen blocks and/or boulders at base of slope</td></tr> <tr><td>4</td><td>Failure affecting more than one berm but not the whole slope</td></tr> <tr><td>5</td><td>Failure extending height of slope</td></tr> </tbody> </table>		CUTS and NATURAL SLOPES		1	Minor shallow failure (<5m in length)	2	Localised surficial loosening, localised evidence of fallen material (rock) at base of slope	3	General evidence of fallen blocks and/or boulders at base of slope	4	Failure affecting more than one berm but not the whole slope	5	Failure extending height of slope	<table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th colspan="2">EMBANKMENTS</th> </tr> </thead> <tbody> <tr><td>1</td><td>Cracks on embankment crest, road unaffected, no subsidence, no other evidence of failure.</td></tr> <tr><td>2</td><td>Shallow failure on embankment surface, no crack or depression on road.</td></tr> <tr><td>3</td><td>Arcuate cracks on road and/or depression less than 10cm - otherwise no mass movement evident.</td></tr> <tr><td>4</td><td>Slope Failure with arcuate cracks on road and/or depression less than 10cm.</td></tr> <tr><td>5</td><td>Slope Failure with extensive arcuate cracks on road and/or depression greater than 10cm.</td></tr> <tr><td>6</td><td>Failure of slope and road - diversion required.</td></tr> </tbody> </table>		EMBANKMENTS		1	Cracks on embankment crest, road unaffected, no subsidence, no other evidence of failure.	2	Shallow failure on embankment surface, no crack or depression on road.	3	Arcuate cracks on road and/or depression less than 10cm - otherwise no mass movement evident.	4	Slope Failure with arcuate cracks on road and/or depression less than 10cm.	5	Slope Failure with extensive arcuate cracks on road and/or depression greater than 10cm.	6	Failure of slope and road - diversion required.
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*arcuate means 'in the shape of an arc'																													
3.2 NO. OF FAILURES PRESENT WITHIN A SLOPE		2	This failure No. 2																										
3.3 FAILURE TYPES	3.4 FAILURE DETAILS																												
☐ Toppling/Wedge/Planar/Rockfall ☐ Rotational/Translational/Slump ☒ Circular/Mud Flow/Debris ☐ Creep/Subsidence	<table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th>W Width(m)</th> <th>D Depth(m)</th> <th>I Length(m)</th> <th>AREA AFFECTED (%)</th> </tr> </thead> <tbody> <tr> <td>22.5</td> <td>15.0</td> <td>100.0</td> <td>60%</td> </tr> </tbody> </table>			W Width(m)	D Depth(m)	I Length(m)	AREA AFFECTED (%)	22.5	15.0	100.0	60%																		
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22.5	15.0	100.0	60%																										
3.5 LOCATION OF FAILURE																													
☐ Top ☒ Middle ☐ Base ☐ Whole Slope																													

Figure 4.20: JKR Sarawak Incident Report

3.6 (a) SOIL TYPE	Brownish Silty CLAY with Gravel		3.7 MAJOR STRUCTURAL GEOLOGY		
(b) ROCK TYPE (filled by geologist)			(filled by geologist)		
3.8 SLOPE GEOMETRY					
(a) SLOPE LENGTH (metres)	27	(b) BASE LENGTH (metres)	100	(c) SLOPE HEIGHT (metres)	15
				(d) SLOPE ANGLE (α)	37°
3.9 RUNOUT DISTANCE			3.10 ANGLE OF FAILURE MATERIAL	0	
3.11 TENSION CRACK					
	Presence of Tension Crack?		W, Maximum Width	L, Maximum Length (m)	Water Filled ?
PRIMARY FAILURE	NO	YES	200mm	5	NO YES
3.12 PRIMARY CAUSE?					
APPARENT CAUSE OF FAILURE (APPLY YOUR ENGINEERING JUDGEMENT)					
Overly steep	Poor/Weak Material	Rainwater	Erosion	High Groundwater	Geological Factor Other
3.13 FAILURE MATERIAL (Mark one only)					
	Soil	Soil with cobbles	Soil with boulders	Rock	Minor (≤50m3) Moderate (>50m3, ≤ 500m3) Major (> 500m3)
3.14 Volume		40500			

STEP 4 FAILURE IMPACT

4.1 ROAD CONDITION		1 Passable (only minor clearing works required)
		2 Passable but required immediate remedial work
		3 Passable following minor clearing / remedial works
		4 Passable only after clearing / remedial works
		5 Closed until completion of major clearing / remedial works

4.2 ESTIMATED ROAD CLOSURE TIME		0 Day	1-2 Days	≥ 3 Days
---------------------------------	--	-------	----------	----------

4.3 SERVICES TYPE				CABLES				PIPELINES			Other (describe)
(a)	VICINITY OF SLOPE CREST	NO	YES ---->	High Voltage	Low Voltage	Telephone	Fibre Optic	Water	Sewerage	Gas	
(b)	VICINITY OF SLOPE TOE	NO	YES ---->	High Voltage	Low Voltage	Telephone	Fibre Optic	Water	Sewerage	Gas	

4.4 FACILITY TYPE AFFECTED BY FAILURES	
GROUP	DESCRIPTION
1	Presence of any residential building, commercial office, store, shop, hotel, factory school, power station, ambulance station, market, hospital, clinic, welfare centre, bus shelter, dangerous goods storage area (e.g. petrol station)
2	Built up area (e.g. indoor carpark, building, mosque, church, temple, manned substation, sewerage treatment plant, railway, flyover, subway, reservoir, construction site)
3	Densely used open space and public waiting area (e.g. playground, open carpark, picnic area)
4	Lightly used open-air recreation area, non-dangerous goods storage area, intensive agricultural area, golf course
5	No facilities present or it is a remote area (e.g. Jungle, Plantation, forest, low intensity agriculture).

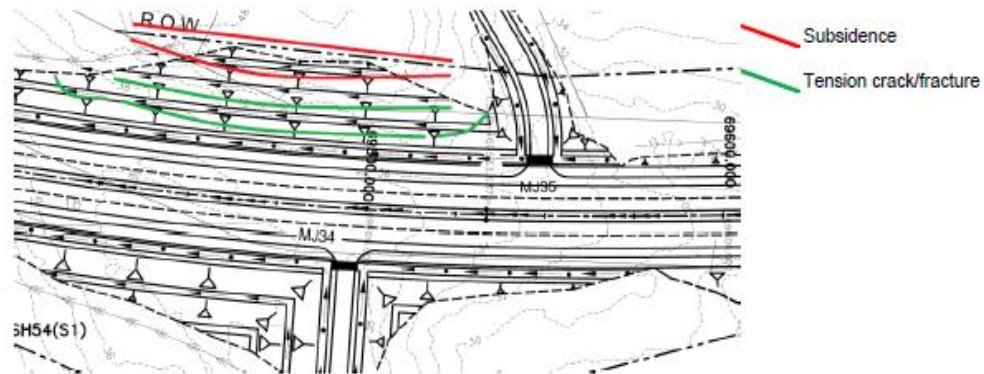
4.5 INJURY / FATALITY / DAMAGES			
(a)	No. of people injured	none	
(b)	No. of fatality	none	
(c)	Other damages & nos.	1. Building () 2. Bridge () 3. Vehicles () 4. Livestock () 5. Others (High level water tank) ✓	
(d)	Estimated Economic Loss (RM)	none	

4.6 ENGINEERING JUDGEMENT (IN YOUR OPINION)			
(a) The Consequence of a failure at this location was.....	Low	Moderate	High
(b) Probability of additional movement	Low	Moderate	High
(c) Probability of significant impacts to the roadway, structures, adjacent properties or features	Low	Moderate	High

Figure 4.21: JKR Sarawak Incident Report

STEP 5 SITE SKETCH

5.1. Site Sketch (Use additional paper if required)



5.2. Photograph (Please include picture reference)

(e.g. Areas of heavy seepage, surface flow, saturated ground, etc)



- (1) Underground water flowing continuously at toe slope
- (2) Failure point of origin (epicenter)
- (3) Tension crack/fracture visible on hard clay layer indication of slide slip not circular
- (4) Fault line (independent) at the back of high level water tank indication of suffusion
- (5) Fault line extended to the adjacent property

Figure 4.22: JKR Sarawak Incident Report

Slope surface consist of sandy material and behave like sandstone surface, difficult for turf to grow. Scarce vegetation, after hydro seeding carried out, on slope surface were observed. Generally, high drainage on slopes causing the water runs down the slope faster. The soil dries out very quickly as it is unable to retain most of the water it receives. It is also found that the soil at slopes is lacking in nutrients, as these get washed away in the runoff. This leads to poor soil properties underneath the slope surface for turf to grow. Thus, it is difficult for the turf or any vegetation to grow on the surface for erosion control and slope hydrology. Undrain condition before cutting of slope is at equilibrium state, however drain condition after excavation and during slope formation had promoted volume change that induced migration of fine particles filling voids at a lower level shown in Figure 4.23, where suffusion is affected by hydraulic conductivity or stress condition transformation or suffusive type of soil (high void ratio). High rainfall collection also induced erosion to slope surface because of barren most outer layer i.e., no vegetation to reduce the velocity of runoff on slope. This is what seemed to be the problem of soil failure on-site before this research being carried out, however, ERT being performed and soil properties of the affected slopes were ascertained, internal instability and suffusion is found to be the cause of soil failure. Results from all soil samples taken at test site 1, 2 and 3 along Pan Borneo Highway Sarawak section Pantu showing suffusion is the main reason for slope failure is shown in Figure 4.24.

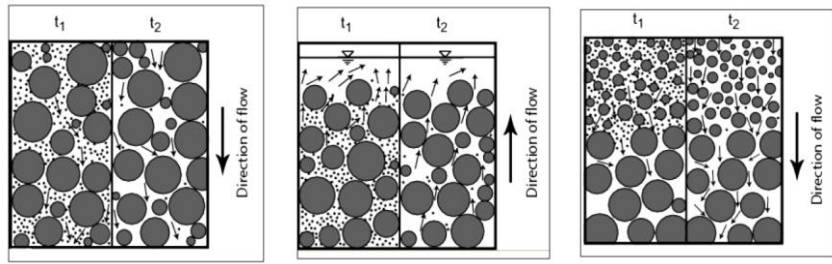


Figure 4.23: Internal Instability or Suffusion (US Bureau of Reclamation, 2015)

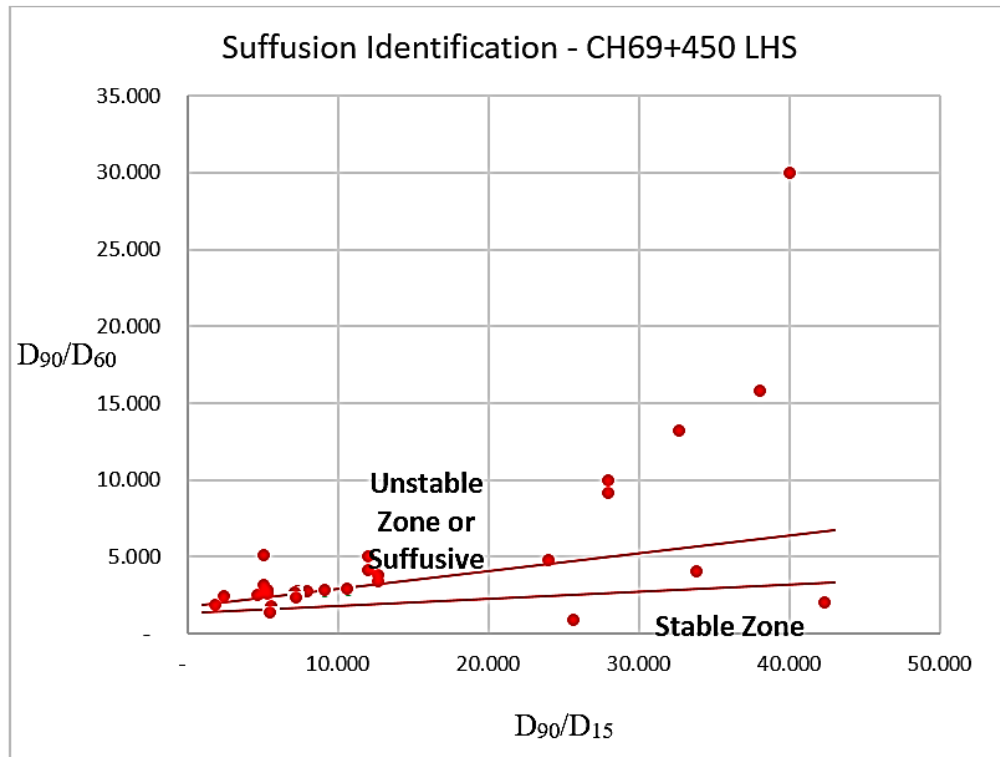


Figure 4.24: Results from all soil samples taken at test site 1, 2 and 3 along Pan Borneo Highway Sarawak section Pantu showing suffusion is the main reason for slope failure

4.5 Soil Amplification

Borehole data as presented in Table 4.1, 4.2 and 4.3 is processed to establish shear wave velocity, V_{S30} . Instead of using the arithmetic average of V_S to a depth of 30 m, V_{S30} is computed as the amount of time needed for a shear wave to travel from a depth of 30 m to the ground surface. 30 m divided by the sum of the shear wave transit periods through

each layer yields the time-averaged V_{S30} . Layer thickness (d) divided by V_s is used to compute the travel time for each layer and modelled as below;

$$V_{S30} = 30 / \sum (d/V_s) \quad \text{Equation 4.1}$$

Where in Imai and Tonouchi (1982) analysis of the largest dataset, which included 1654 data pairs from 386 borings at 250 sites around Japan. V_s correlation formulae based on N-value, soil type, and geologic age were created by Imai and Tonouchi.

$$V_s = 97.0 N^{0.314} \quad \text{Equation 4.2}$$

The conversion of SPT result to V_s is represented as calculated using correlation equation between SPT and $\Omega.m$ i.e., apparent resistivity, $\Omega.m = 0.6171N + 5.5342$, where N is number SPT blows subsequently used equation 4.2 to get shear wave velocity. Comparison of S_a/PGA to $1.1 \cdot 10^{-5}g$ and $0.03g$ of V_{S30} soil layer shown in Figure 4.25. In addition, Caltrans/NEHRP soil profile types is shown in Table 4.6. Calculation of the elastic response spectrum, based from site-specific data attained from SPT in terms of spectral acceleration and spectral displacement that reflect the site-specific seismic action in the horizontal or vertical direction. Useful for seismic isolation systems, seismic displacement calculations, and the construction of structures that remain within the elastic range. Elastic response spectra at soil surface and design response spectra, chart of site-specific seismic soil amplification in average (fourier & new model amplification limit) + design response spectra are shown in Figure 4.26, and Figure 4.27 respectively.

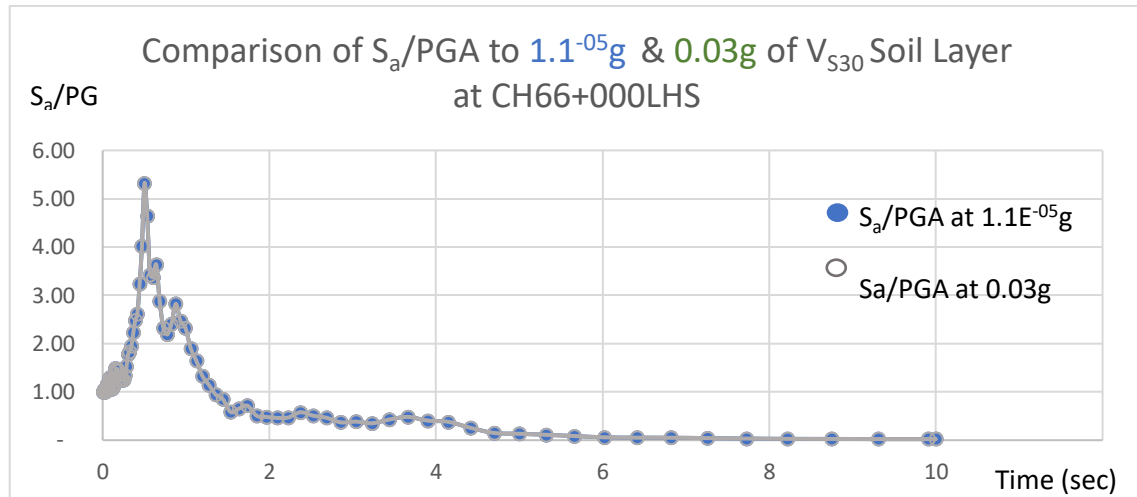


Figure 4.25: Two Intensities of PGA used to Test the Variants and its Effect on Soil Amplification. The Results Shown Similar Soil Amplification

Table 4.6: Caltrans/NEHRP Soil Profile Types (Wair et al., 2012)

Site Class	Soil Profile Name	V_{s30}	SPT-N Value	Undrained shear strength
A	Hard Rock	$>1,500 \text{ ms}^{-1}$	----	----
B	Rock	$1,500 > V_{s30} > 760$	----	----
C	Very Dense Soil and Soft Rock	$760 > V_{s30} > 360$	$>50 \text{ bpf}$	$>100 \text{ kPa}$
D	Stiff Soil	$360 > V_{s30} > 180$	$15 < \text{Stiff s.} < 50$	$100 > \text{Stiff s.} > 50$
E	Soft Soil	$<180 \text{ ms}^{-1}$	$<15 \text{ bpf}$	$<50 \text{ kPa}$
F	Soil requiring site-specific evaluation (see notes below)	----	----	----

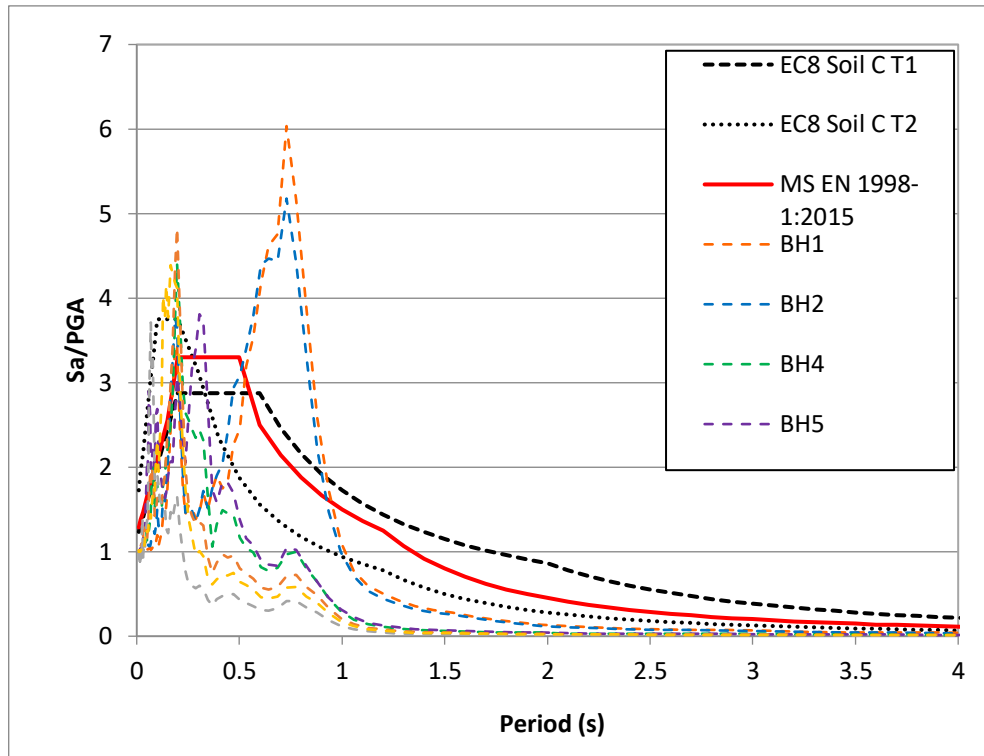


Figure 4.26: Response Spectra at Soil Surface and Design Response Spectra

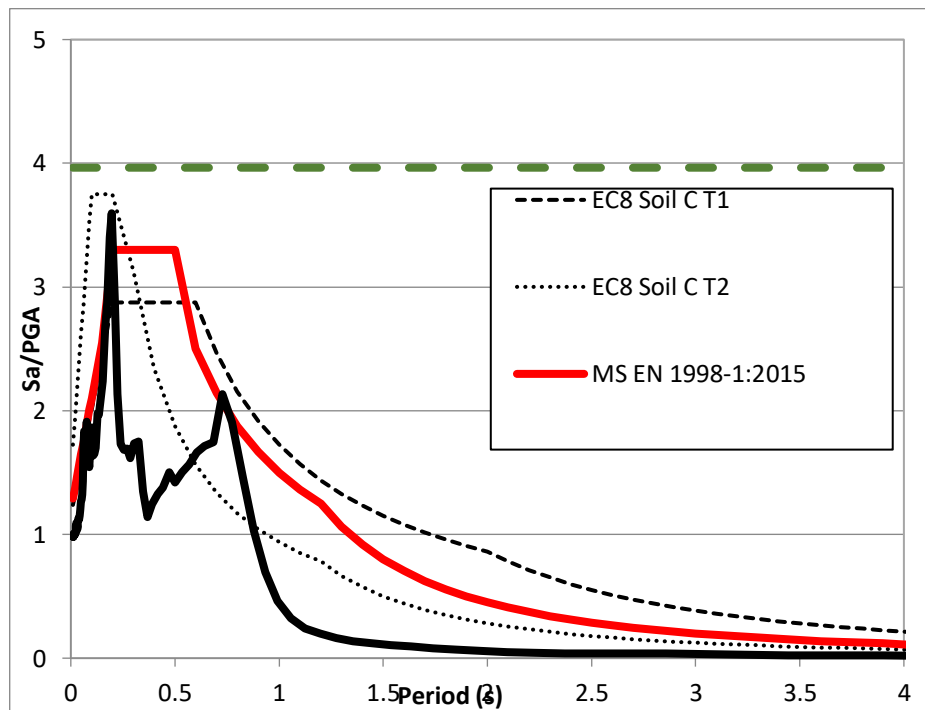


Figure 4.27: Chart of Site-specific Seismic Soil Amplification in Average (Fourier & New Model Amplification Limit) + Design Response Spectra

4.6 Site-Specific Suffusion Assessment Framework

Using a highly developed and cutting-edge electrical resistivity geophysical exploration system is a revolutionary change. Putting an emphasis on the ideal response to current technological breakthroughs, cost effectiveness, adaptability in field techniques, high accuracy in geo-electrical exploration, and long-term viability for both novice users and seasoned researchers. The 2D ERT Resistivity/IP measurement, a flexible geophysical technique used for subterranean imaging, is used in this study. It can measure the spontaneous potential (SP), time domain induced polarization method (IP), and resistivity. The surveying method called electrical resistivity test for imaging is used in complex geological Pantu areas where boreholes for soil investigation and other methods are ineffective in providing accurate subsurface information over a small region. Multi-electrode systems are typically used for electrical resistivity investigations. For this research surveys, a multicore wire is attached to 20 electrodes that are placed in a straight line with constant spacing. The imaging printout with resistivity data, in log value for detailing, against depth of underground scanned. Electrical conductivity option is also available for printout and further translated to soil porosity, void ratio, soil liquid limit, type of soil/rock, water content which further can be correlated with proven methodology such as Fourier Amplification to attained instability index (suffusion) in soil against soil serviceability limit state.

Detail electrical resistivity data attained in the 3 test sites within Pantu District of Pan Borneo Highway Sarawak project, geo-hazard landslide area, proved to be suffusive or internally unstable via Burenkova (1993) and Fourier amplification versus MS EN 1998-1:2015. Correlation between soil amplification and suffusion void ratio as displayed in Figure 4.30 $\sum(e/e_{min})_{s30}^{1/D}$, is successful at 94% correlated. Correlation of Fourier Amplification at $S(T_0)$ and Suffusion Void Ratio of BH samples from all test sides, see

Figure 4.31, also successful at 94% interrelated. Correlation of Soil Max. Amplification (Fourier Amplification) and Suffusion Limit, see Figure 4.31, as well at 94% correlated, chart of site-specific seismic soil amplification (Fourier & new model) + design response spectra is shown in Figure 4.32.

4.6.1 Methodology to Convert Site-Specific Porosity to Void Ratio

Void ratio (e) is the ratio of volume of voids to the volume of solids. Porosity (n) is the ratio of volume of voids to the total volume of the soil. Void ratio of soil shall later be input to the above new modelling that has been well correlated with Fourier amplification is shown in Figure 4.32. The relation between the two can be derived as:

$$\text{Void ratio } (e) = V_v/V_s$$

$$\text{Porosity } (n) = V_v/V_T$$

$$V_T = V_v + V_s$$

$$n = V_v / (V_v + V_s), \text{ where } 1/n = 1 + V_s/V_v \text{ or } 1/n = 1 + 1/e$$

$$1/n = (e + 1)/e, \text{ therefore } n = e/(1 + e), \phi = e / (1+e)$$

Where, ϕ = porosity and directly obtained from Archie's Law and Apparent Resistivity reading from ERT takeout/printout, and void ratio i.e., $e_{\max} = 0.072 + 1.53e_{\min}$ from Cubrinovski et al., 2002. Figure 4.28 depicts the equation for calculation of soil suffusion limit. In addition, correlation of soil amplification and suffusion void ratio, correlation of amplification at $s(t_0)$ – fourier amplification and suffusion void ratio of BH samples, correlation of soil maximum amplification (fourier amplification) and suffusion limit, and chart of site-specific seismic soil amplification (fourier & new model) + design response

spectra are shown in Figure 4.28, Figure, 4.29, Figure 4.30, Figure 4.31 and Figure 4.32 respectively.

Calculate Soil Suffusion Limit, $S_L \approx S_e/PGA$

$$S_L = a_s SF = a(18523((\sum e/e_{min})_{s30}^{1/D})^3 - 59784((\sum e/e_{min})_{s30}^{1/D})^2 + 64297(\sum e/e_{min})_{s30}^{1/D} - 23040)$$

Where, $S30 = 30m$ depth of soil properties in layers
 $D = 30m$
 $a = 1.6$

$$SF = (18523((\sum e/e_{min})_{s30}^{1/D})^3 - 59784((\sum e/e_{min})_{s30}^{1/D})^2 + 64297(\sum e/e_{min})_{s30}^{1/D} - 23040)$$

Figure 4.28: Calculation of Soil Suffusion Limit

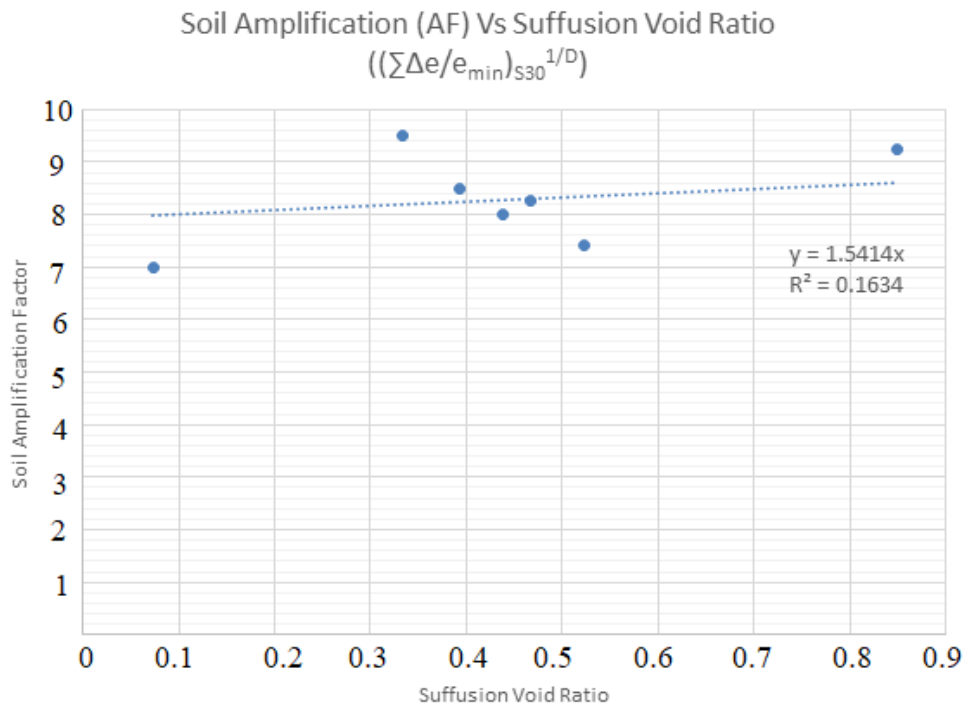


Figure 4.29: Correlation of Soil Amplification and Suffusion Void Ratio of BH Samples

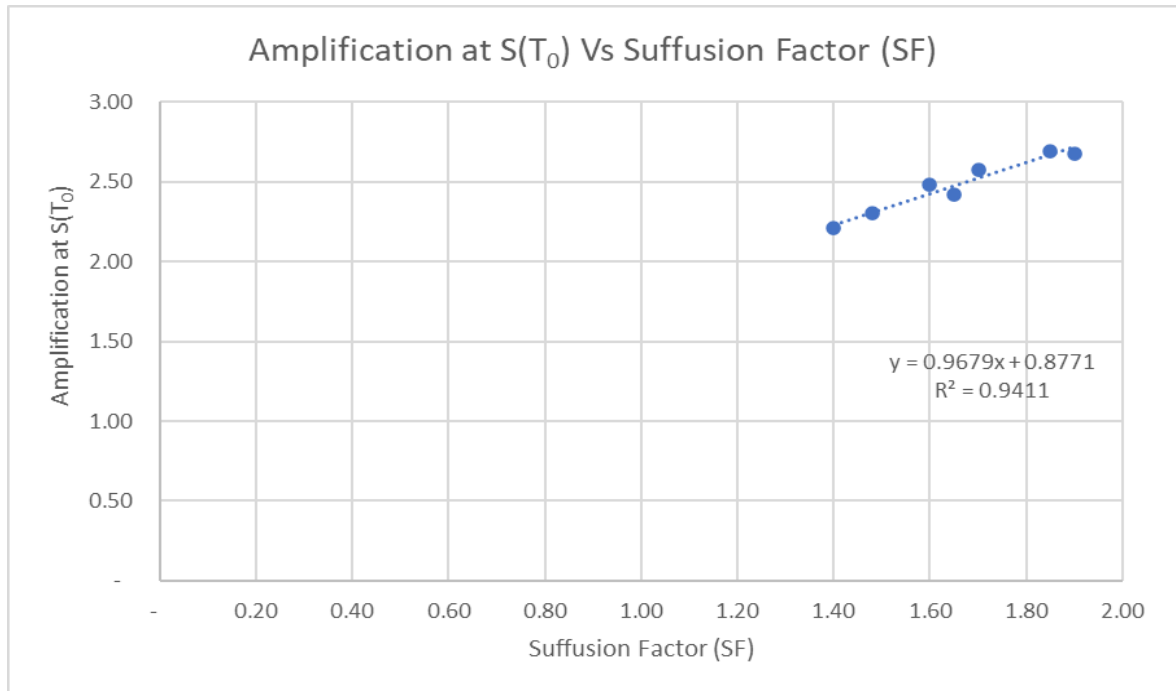


Figure 4.30: Correlation of Amplification at $s(t_0)$ – Fourier Amplification and Suffusion Void Ratio of BH Samples

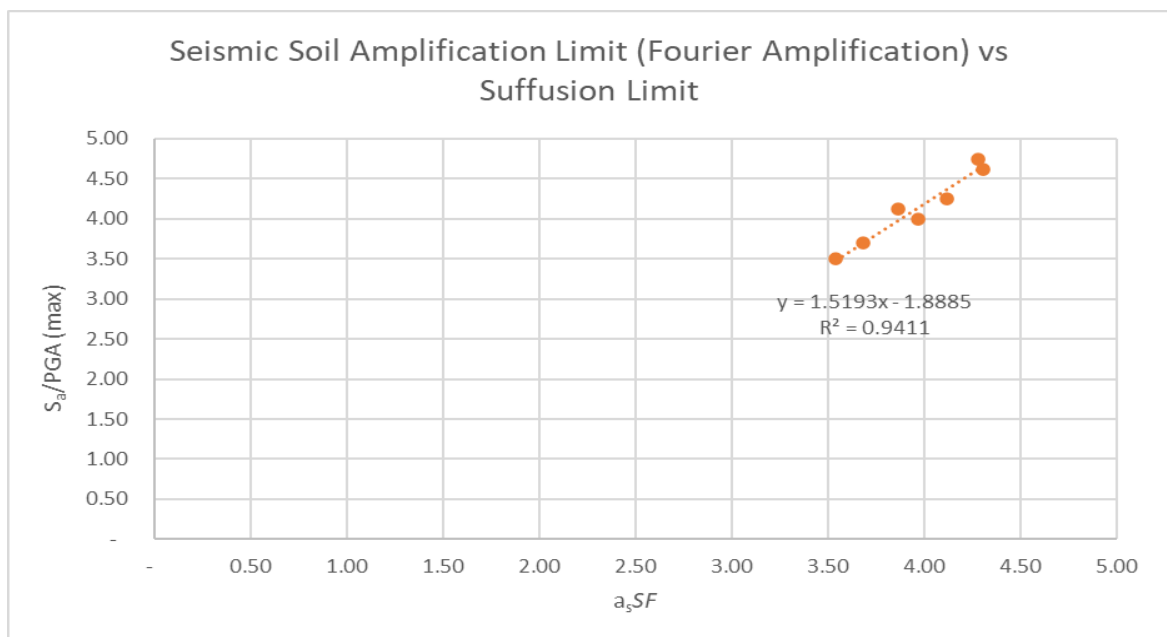


Figure 4.31: Correlation of Soil Maximum Amplification (Fourier Amplification) and Suffusion Limit

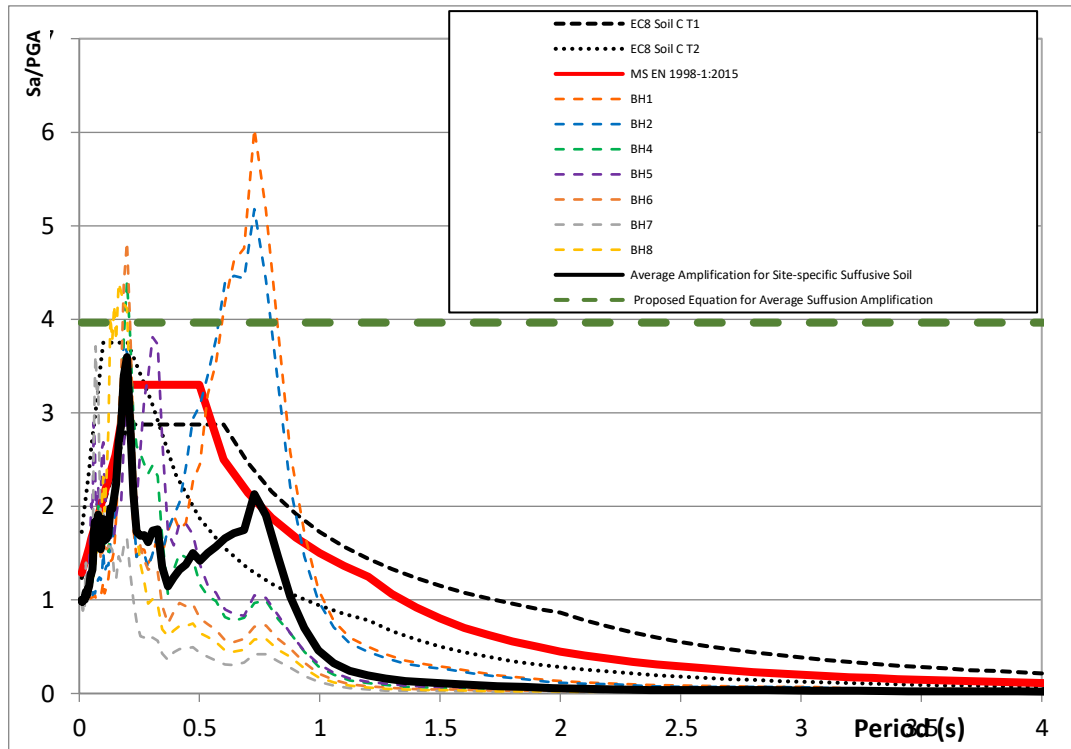


Figure 4.32: Chart of Site-Specific Seismic Soil Amplification (Fourier & New Model) + Design Response Spectra

4.7 Liquid Limit via Electrical Resistivity Tomography (ERT)

CH66+000 of work package contract 03 of Pan Borneo Highway Sarawak in Pantu shown in Figure 4.33 and Figure 4.34, were selected to be the area of study to extract the value of cementation exponent (m). In addition, result of soil samples taken from test site 1, 2 & 3 including ERT is shown in Table 4.7.

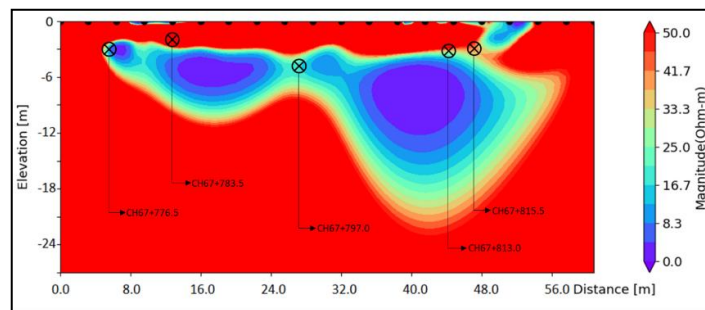


Figure 4.33: Electrical Resistivity Tomography of the Area of Study – Test Site 2

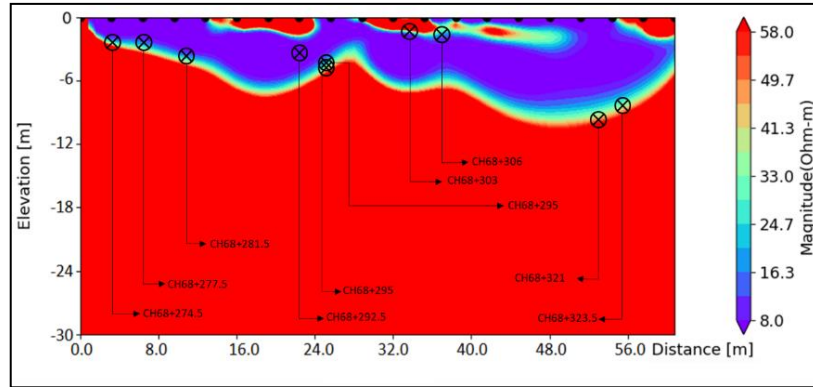


Figure 4.34: Electrical Resistivity Tomography of the Area of Study – test site 3

Table 4.7: Result of Soil Samples Taken from Test Site 1, 2 & 3 Including ERT Takeout

Location/CH	65974.5	65976.5	65993.0	66004.5	66006.0	66022.0	67776.5	67783.5	67797.0	67813.0	67815.5	68274.5	68277.5	68281.5	68292.5	68295.0	68303.0	68306.0	68321.0	68323.5
D90	0.685	2.100	2.100	0.800	2.450	0.400	0.140	2.855	0.380	0.900	0.950	0.350	0.400	0.400	0.180	1.800	0.950	0.540	3.000	0.550
D60	0.250	0.210	0.230	0.280	0.185	0.140	0.075	0.181	0.075	0.220	0.280	0.140	0.155	0.150	0.075	0.380	0.120	0.212	0.100	0.230
D15	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075	0.075
PL	24.270	17.090	23.380	22.810	14.740	25.400	18.780	20.950	26.250	30.950	25.110	21.270	26.530	22.630	27.570	26.970	22.570	24.960	22.290	24.810
LL	54.900	27.700	40.900	38.400	31.200	53.300	73.600	39.800	64.700	47.900	44.400	54.900	63.000	61.700	83.200	46.700	41.300	59.900	37.800	44.700
$\Omega \cdot m$ (ERT)	23.228	124.238	48.587	56.281	94.187	23.306	11.212	50.863	14.189	31.487	38.760	23.682	15.699	16.665	8.073	32.906	46.506	43.694	57.528	36.745
ϕ	54.098	27.662	40.270	37.970	30.902	54.026	72.396	39.539	65.888	47.900	44.079	53.681	63.276	61.783	82.561	47.063	40.981	59.545	37.639	45.031

The liquid limit of a soil is the moisture level at which it changes from a plastic to a liquid state. Utilizing the liquid limit, cohesive fine-grained soils can be identified and categorized accordingly. Changes in a soil's moisture content can have a considerable impact on its shear strength, especially in fine-grained soils. The moisture content of a soil is defined as the amount of water that can be removed from the pore space between soil grains by oven drying at a temperature no greater than 110 °C. The moisture content has a big impact on how the soil behaves. It is the percentage of a soil's dry mass that is composed of the water content of the soil. Results for cone penetrometer of samples taken from test site 1 of the area of study are shown in Table 4.8.

Table 4.8: Result of Soil Samples Taken from Test Site 1 Including ERT Takeout

Location/CH	65974.5	65976.5	65993.0	66004.5	66006.0	66022.0
LL	54.900	27.700	40.900	38.400	31.200	53.300
$\Omega \cdot m$ (ERT)	23.228	124.238	48.587	56.281	94.187	23.306
ϕ^{-m}	4.646	24.848	9.717	11.256	18.837	4.661
m	-2.56	-2.50	-2.54	-2.53	-2.52	-2.45

Where, ϕ is equivalent to LL, therefore m or cementation exponent is constant at 2.5. The value of cementation exponent is said to increase with decreasing permeability (Archie, 1942) which $m = 2.5$ is strongly related to low permeability soil i.e. clay. This constant will be used to determine the liquid limit for test site 2 and 3 then later compare to the laboratory test result via cone penetrometer to confirm the correlation.

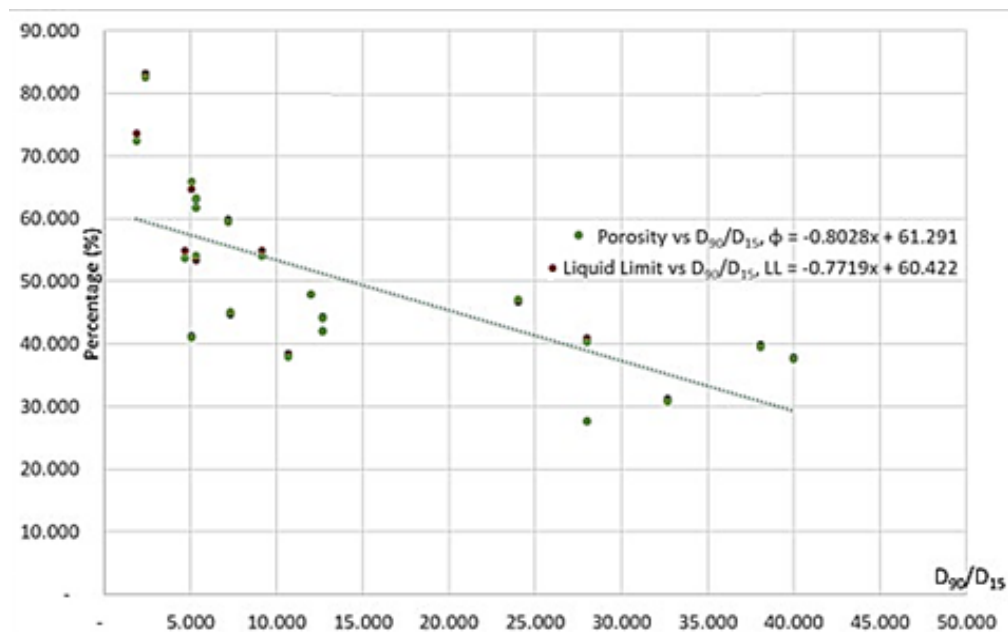


Figure 4.35: Linear Regression of Soil Porosity (ERT) vs Liquid Limit (Lab Test) for Test Site 1, 2 & 3

Linear Regression of Soil Porosity (ERT) vs Liquid Limit (Lab Test) for Test Site 1, 2 & 3 shown in Figure 4.35. As a result of the water filling the pore space between the soil grains, which is a gap that can only be inhabited by air or water, the study's hypothesis states that the porosity of soil is equal to its liquid limit. The factor contributing to the variance is pointing towards the constant, a , i.e., tortuosity because of different soil particle size distribution resulting to different matrix of path length of the hydraulic conductivity.

4.8 Chapter Summary

This chapter depicted all the results and their respective discussions. The highest peak ground acceleration in Sarawak is 0.09g corresponding to 50 years of return period. Simplified amplitude limit of failure has been developed to alternate Fourier amplitude to propose ERT data collection and new approach to develop serviceability limit state (SLS) or design response limit. Data acquisition via ERT that sensibly converted to shear wave velocity and soil shear strength shall be verified its compliance to factor of safety (FOS) in finite element method and has been justified with the existing guideline for slope design from the public works department of Malaysia. Output of this research improved the reliability of design safety factor for any constructions by integrating advanced technology application. As a result, ERT raw soil properties data that correlated well with soil liquid limit where volume of void over total void (V_v/V_T) is in equivalent and soil conductivity (σ) over soil resistivity (Ω) is consistent with liquidity index. This research also discovered that electrical conductivity against electrical resistivity, $1/\text{Sigma } (\Sigma^{-1}) \div \text{Ohm } (\Omega) = 1$ and is a threshold index of soil instability. It is also discovered that when the soil instability index higher than 1 the site-specific soil is unstable.

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Suffusion failure is a phenomenon not just for Malaysia but it happened in many places in the world. Pantu area in Pan Borneo Highway Sarawak project has experienced an increase in slope failure incidents due to site-specific material that is susceptible to suffusion and high moisture or high hydraulic conductivity in soil. This research has proven that major cause of soil failure contributed by suffusion or material susceptible to failure or unstable soil. This research has also verified that electrical resistivity imaging can provide detail soil properties needed to analyse stability of soil. This research can benefit geotechnical engineering field by providing porosity or void ratio analysis, contrary to many researches carried out before this that based on hydraulic conductivity and its relation to suffusion, and using ERT to ascertain the soil properties that is simpler, reliable and faster to attain. The correlation with soil amplification proposed will benefit both geotechnical and sub-structural engineering where design response limit or amplification factor is attainable using ERT and new proposed empirical model for suffusion factor (SF) can be a supplementary constant of FOS to slope design. In future research ERT may be useful to detect suffusive material and suffusion zone. It is also beneficial to ascertain void ratio in soil for geotechnical treatment such as consolidation via surcharge treatment and correlate between Asaoka method and Darcy's law.

Many of soil failures around the world were initiated by suffusion, this research brings tremendous benefit to include suffusion via void ratio, e_0/e_{min} , and able to propose a

simplified equation of soil suffusion factor that equivalent to seismic soil amplification factor, where Suffusion Factor; and Suffusion Limit, $S_L = a SF$, where $a = 1.6$.

$$SF = (18523((\sum e/e_{min})_{S30}^{1/D})^3 - 59784((\sum e/e_{min})_{S30}^{1/D})^2 + 64297(\sum e/e_{min})_{S30}^{1/D} - 23040)$$

Observation data via ERT & SPT were analysed and showed that there is correlation between void ratio in soil, e_0/e_{min} that caused suffusion and shear wave velocity of soil which influence seismic soil amplification. ERT has an enormous potential to alternate SPT and this substitution is reliable to investigate soil properties and provide theoretical consolidation settlement, S_c . Suffusion factor (SF) increase in linear to seismic soil amplification factor (AF). SF can be useful as supplementary factor to factor of safety (FOS) to slope design.

This research also discovered that electrical conductivity against electrical resistivity, $1/\Sigma (\Sigma^{-1}) \div \text{Ohm } (\Omega) = 1$ and is a threshold index of soil instability. It is also discovered that when the soil instability index higher than 1 the site-specific soil is unstable. Further research on the instability index is potential and beneficial to the geotechnical engineering industry for geo-hazard identification. In summary, the following evidence was attained from a series of analysis into the research between influences from synthetic seismic loading via Fourier amplification against serviceability limit state of soil and correlation of site-specific void ratio of maximum soil amplification at 30m thick surface layer:

- Archie's law is valid for application in soil as well as in rock.
- Suffusion or soil internal instability can be determined via void ratio in soil and correlated Fourier amplification for maximum amplitude against limit state boundary.
- Soil void ratio data acquisition through ERT is attainable with Archie's law.

- Liquid limit of soil can be determined via ERT.

Electrical Conductivity against Electrical Resistivity, $1/\Sigma (\Sigma-1) \div \text{Ohm } (\Omega) = 1$ and resemble to Liquidity Index where it is also a threshold index of soil instability.

5.2 Recommendations

This research discovered numerous potentials in ERT for acquiring field data and its further possible usage. From Archie's law alone one can further research to correlate as follows:

- Factor m from Archie's law, $\rho = a\Phi^m f^n \rho_w$ i.e., cementation exponent that potentially c' i.e., cohesion coefficient for saturated soil
- Tortuosity ($F = \sigma_w/\sigma_f$), where σ_w = conductivity of water, σ_f = conductivity of formation, that can be correlated with friction angle
- This shall be very meaningful to appreciate Mohr-Coulomb equation and failure criterion can be quickly established
- The recommendation also promotes slope design at site-specific and slope failure can be minimized avoiding potential disaster to properties and human lives

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