UNIVERSITI TEKNOLOGI MARA

SUSTAINABLE DESIGN PERFORMANCE OF CEMENTITIOUS BINDER FOR CONSTRUCTION OF STABILISED SOIL/SUBGRADES

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Thesis submitted in fulfilment
of the requirements for the degree of
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CONFIRMATION BY PANEL OF EXAMINERS

I certify that a Panel of Examiners has met on 10th October 2025 to conduct the final examination of Ghazali Bin Ismail on his Doctor of Philosophy thesis entitled "Sustainable Design and Performance of Cementitious Binder for Construction Stabilised Soil/Subgrades" in accordance with Universiti Teknologi MARA Act 1976 (Akta 173). The Panel of Examiners recommends that the student be awarded the relevant degree. The Panel of Examiners was as follows:

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I declare that the work in this thesis was carried out in accordance with the regulations of University Teknologi MARA. It is original and is the results of my own work, unless otherwise indicated or acknowledged as referenced work. This thesis has not been submitted to other academic institution or non-academic institution for any degree or qualification.

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ABSTRACT

The enhancement of both existing and virgin pavement materials has been successfully executed in Malaysia since the 1980s using the procedure of In-situ Stabilisation. Most designated projects have utilised cement or lime as the binding agent for subgrades, granular bases and sub-base materials. The utilisation of cementitious binders is increasingly favoured in Malaysia as asset owners acknowledge the need of establishing robust foundations for pavement projects. In pavement design, the two most pivotal inputs that affect the overall thickness of a pavement structure are the traffic loadings and the design subgrade California Bearing Ratio (CBR). The stiffness of soil/subgrade is the critical variable that can be optimised for enhancement and should be improved wherever feasible. Cement and lime are very effective in significantly enhancing bearing capacity, mitigating volumetric instability and specifically incorporating lime to diminish the moisture susceptibility of clayey, expansive and plastic materials frequently found in pavement subgrades. Some of the soil characteristics may contain organic i.e. peat soil that may require the additional imported suitable earth or additional material such as crusher run, fly ash or slag shall be taken into consideration. The examination of the soil characteristics, this research is focusing the design process and validating the performance of the admixture from laboratory and field evaluation based on the design charts and specifications addressed for cement and lime including of pavement layers to be treated and process of the soil treatment. The methods of the stabilisation's research analysis which referred to laboratory testing from few projects undertaken to compare the results findings are met or achieved accordingly. Therefore, for design criteria choosing binder agents for type soils can be relate to plasticity index (PI) range between 10% to 20% by using cement and 20% and above which is high plasticity and moisture which may produce to acidic reaction to be used quicklime as a binder agent. Identifying the proportion of the binders is crucial to ensure the flexibility of the pavement layers. The average results of the proportion of the binders shown at 4% which same as industrial design. Overall result from the design and process of this research can be concluded as significantly help the stakeholder i.e. designer, site supervisory and end user for cost and time saving. This study presents a novelty contribution through the completion and integration of laboratory based on Field Density Test (FDT) California Bearing Ratio and Unconfined Compressive Strength (UCS) test to validate the mechanical performance of stabilised soil or subgrades). A key innovation of this research is the development of a simplified calculation method whereby compaction density and plasticity index data which derived from field measurements, can be used to manually estimate soil performance parameters without the need for extensive laboratory testing. Furthermore, the proposed formulation provides a practical and reliable approach for determining the optimum percentage of cement or lime required for soil stabilisation, thereby offering a cost effectiveness, time efficient and field applicable solution for pavement and ground improvement design.

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Finally, this thesis is dedicated to wife and children for the vision and determination to educate me. The unwavering support is meaningful and this winning piece of victory is a gift and should be dedicated to both of you. Alhamdulilah.

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LIST OF SYMBOLS

Symbols

Al_2O_3	Aluminium Oxide
AS	Australia Standard
C_p	Cement Content
CO2	Carbon Dioxide
CaO	Calcium Oxide
Fe_2O_3	Ferric Oxide
Mg	Milligram
kN	Kilo Newton
LL	Liquid Limit
MgO	Magnesium Oxide
m^3	Meter Cubic
m^2	Meter Square
Ph	Potential of Hydrogen
PL	Plastic Limit
PSD	Particle Size Distribution
P_d	Maximum Dry Density
SL	Shrinkage Limit
SiO_2	Silicon Dioxide
T	Tonnage

T	Traffic Categories

LIST OF ABBREVIATIONS

Abbreviations

ADT	Average Daily Traffic
ALF	Accelerated Loading Facility
ATJ	Arahan Teknik Jalan
BS	British Standard
BSCS	British Soil Classification System
CBR	California Bearing Ratio
CIPR	Cold In-Place Recycle
CMV	Compaction Meter Value
DCP	Dynamic Cone Penetrometer
ESAL	Equivalent Single Axle Load
FDT	Field Density Test
GBSF	Granulated Blast Furnace Iron Slag
JKR	Jabatan Kerja Raya
kg	Kilogram
LEF	Load Equivalent Factor
m	Metre
mm	Millimetre
MDD	Maximum Dry Density

MDP	Machine Drive Power
MPa	Mega Pascals
Msa	Million Standard Axles
MSDS	Material Safety Data Sheet
OMC	Optimum Moisture Content
OPC	Ordinant Portland Cement
PI	Plasticity Index
RAP	Reclaimed Asphalt Pavement
RTVM	Road Traffic Volume Malaysia
SLC	Structural Layer Coefficient
TA	Thickness Equivalent
UCS	Unconfined Compressive Strength

CHAPTER 1
INTRODUCTION

1.1 Research Background

Whether building a new road or rehabilitating an existing road, subgrade stiffness is by far one of the highest risk elements to ensure the finished pavement meets design life expectations. Pavement designers recognise that there is great sensitivity in the outcome of a pavement design when the subgrade stiffness (California Bearing Ratio - CBR) changes. It is therefore critically important to ensure that selection of an appropriate design subgrade CBR is well understood during the modelling phase.

Unfortunately, some of the design engineer demonstrated their pavement designs by drawings that show the design subgrade CBR with a chosen value, but this is rarely understood. For example, if a design subgrade CBR is shown as 5%, what thickness of subgrade is this modelled over? How does this information translate to the construction phase if subgrade treatment is required?

Another example is where many of our design manuals stipulate that based on a given subgrade CBR a minimum pavement thickness is to be considered. The most common understanding of this concept in Malaysia is that the design subgrade is modelled over a thickness of 300 mm. There are numerous international papers, design guidelines and technical notes that have demonstrated through various forms of research that this is not adequate. In fact, the most common approach internationally is to model at least 1m of subgrade in the determination of a Design CBR value for input into the pavement design process.

The first section of this research will explore in more detail with examples showing the differences in pavement design life when 300 mm (sub grade layer) or 1m (pavement layer) of subgrade material is modelled. The second phase of this research will then explore the two common methods of improving subgrade properties, being Remove and Replace or In-situ Stabilisation. Finally, a case study will be presented which explores the use of cementitious binder (cement/lime) to treat soil/sub grade on a project site in Malaysia.

The stabilisation of subgrade material at the pavement layer has been effectively implemented in Malaysia since the 1980s using the procedure of in-situ stabilisation. The stabilising operations have been finalised utilising cementitious material as the binding agent. The utilisation of cementitious binders is increasingly favoured in Malaysia by design engineers who recognise the need of establishing robust foundations for pavement projects. The two most crucial data inputs for pavement layer design that affect the whole thickness of a pavement structure are loadings and the design subgrade through the California Bearing Ratio (CBR) test. The strength and stiffness of soil/subgrade are the critical variables that can be engineered for enhancement and sustainability. Cement/Lime is a cementitious material that significantly enhances bearing capacity and mitigates volumetric instability by using a cementitious lime product to lower the moisture susceptibility of clayey and plasticity materials typically found in pavement subgrades. This research paper will examine the factors influencing on the design of cementitious (lime and/or cement) stabilised soil/subgrades in Malaysia, given the current absence of implementation design methodologies and construction specifications for cementitious stabilised materials, with a focus on mix design parameters.

1.2 Problem Statement

In Malaysia, there are various geological formations that contribute the needs of executing soil stabilisation. Construction of pavement layer on soft deposit has been a major problem since the pavement experience weakened structural properties due to insufficient stabilisation of the subgrade formation. Hence, it is necessary to ascertain the soil characteristics at different geological conditions, especially for the water table fluctuates due to low or flat regions and where access to remote areas is restricted. In essence, most ground conditions on lowland areas are made of either marine or alluvium clay, depending on the deposition of the weathered material. These issues need to be addressed with cementitious binders which could absorb water and increase the bearing resistance of the subgrade. In addition, at localities where the deposition is rather young, the residual soil has not experienced over-consolidation, therefore, the need to stabilise the material with cementitious agents is vital.

Based on the existing ‘Manual on Pavement Design’ (Arahan Teknik Jalan 5/85) was published in 1985 and was largely reliant on the use of empirical design methods that did not consider specific material properties. In 2013 JKR released a revised design guideline titled, ‘Manual for the Structural Design of Flexible Pavement’ (ATJ 5/85 Pindaan 2013) which promotes the use of mechanistic design modelling techniques. (JKR, 2017)

One of the most significant changes for designers was the minimum properties required for subgrade materials (JKR, ATJ 5/85 (Pindaan 2013), JKR 21300-0041) with a minimum design CBR of 5% required for pavements with traffic categories of T1 to T3 and a design CBR of 12% required for pavements with traffic categories of T4 or T5 (>10.0E+06).

For most pavement rehabilitation projects carried out in Malaysia, preliminary testing of subgrade stiffness using soaked CBR testing is generally done on the top 300 mm of the subgrade. If test results show values less than CBR5 or CBR12, then it is often the same 300 mm that is specified as requiring treatment to improve the subgrade strength to at least the design value. Here lies the inherent problem in that the remaining 700 mm of untreated subgrade retains a 'non-complying' strength which has a direct impact on the ability of the overall subgrade to support the pavement structure for the designated design period.

Once the top 300 mm of unsuitable soil has been treated sufficiently, according to the various layered elastic analysis programs used for design modelling, the entire 1m of subgrade must still be considered. In practice often the top 300 mm is analysed for conformance against the design subgrade CBR – which is incorrect. Whilst there are many methods that have been published to describe how to model the subgrade stratum using different CBR values within the profile, JKR already have an accepted approach shown in Equation 1 from the 1985 Design Manual.

Further, it has been widely published (Jawad et. al., 2014) that the use of cementitious (quicklime) to treat weak soils provides significant improvements to the material properties, including increased strength, reduced plasticity, increased workability, increased durability and reduced consolidation settlement characteristics. With the correct amount of lime addition, swell is typically reduced close to zero in most cases (Jawad et. al., 2014) which provides significant advantages for the use of the treated material.

The above information for problem statement is address to inadequate design even though have a standard specification from JKR and experienced especially for soil properties, design parameter, design approach and best practice for design, laboratory and operationally.

1.3 Objectives of the Research

The objectives of the research are given below:

i) To assess the soil characteristic for the best suitable binder agents according to design and specification taking into consideration the design perspective for permeability and capillary rise that influence design considerations.

ii) To evaluate the significance of pavement layer from treated soil and without treated soil for two significant soil types i.e. residual soil and clay.

iii) To analyse the performance of the admixtures from laboratory test and field evaluations based on the developed design charts and specifications addressed for cement and lime.

iv) To develop the process of stabilise incorporating specialised mixing equipment with rotating mixing drums that efficiently mix the existing material with a chemical powder binder i.e. cement and lime which are spread at design application rates in front of the mixing machine.

v) To produce design charts and design specifications of soil stabiliser i.e. cement and lime against two significant types of soil characteristics based on laboratory and field performance of the binding agents.

1.4 Research Questions

From the objectives above, there are several research questions derived to further explain the objectives.

i) How important is the drainage system in the design of subgrade stabilisation?

ii) What is the influence of soil characteristics in selecting the suitable design mix and material for sub grades stabilisation?

iii) Why is the performance-based tests and laboratory verification need to be executed for various design admixtures?

iv) What are the elements included to the design chart for stabilisation design process?

v) Are there any process and method of specifications used to guide designers and installer on the use of stabilisation admixtures?

vi) How to evaluate the entire construction process to ensure the product follow according to the new design and improved specification?

1.5 Scope and Limitation of Study of Research

This research will focus on the sub grade on pavement layers. The case studies have been located and analyses based on problem statements and objectives including of design, type of soil and characteristics and construction. The analysis techniques are proven to be sufficient to gain a more appropriate design, best practice of soil stabilisation and drainage maintenance. The existing soil shall be identified from a few samples and will be tested and analysed at laboratory.

The tests to be conducted for the samples taken including of Natural Moisture Content Test, Sieve Analysis, Proctor test – Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) Test, Atterberg Limit Test for Plasticity Index (PI) and Soaked California Bearing Test (CBR).

For design mix process, the same process shall be executed with additional mixture agents and additional test shall be carried out, example Unconfined Compression test (UCS) to derive the strength as per specified (in MPa). The laboratory test process shall propose mixing at 3 stages of mixtures (in percentage, %) for example 3%, 5% and 10%. Those samples of 3 stages shall require 6 nos. of mould and each mould need 8 kg sample of soil and the process will take about 5 days for initial test and 7 days for design mix process.

This research shall cover the affected area for marine clay at Pulau Burung, Seberang Parai for coastal area and laterite soil at Sungai Buloh for road project. In Malaysia that marine clay at coastal area is a geotechnically significance of soil type, particularly along the west coast of Peninsular Malaysia and parts of Sabah and Sarawak. The research may limit to one (1) metre depth which is considered as pavement layer.

1.6 Significance of Research

Cementitious (Cement/Lime) stabilisation of subgrade materials in Malaysia is growing in acceptance with asset owners and designers recognising the benefits of improvement of the engineering properties for the untreated soil. Benefits include enhanced working platforms, reduced pavement thickness, reduced overall construction cost and construction time, reduced risk of premature failure from expansive and reactive subgrade materials and the ability to minimise the volume of imported materials.

The process of using in situ stabilisation as an alternative to traditionally remove and replace options provides greater benefits in several areas, such as Recycling pavement materials, Minimising use of quarried products and disposal. Saved or reduced on construction cost and time as well as energy use, minimise trucks on road network which means less fuel and pollution. Reduced CO2 emissions by up to 40% over conventional methods (Vorobieff and Murphy, 2003).

From the work proposed in this research, it is hoped that the local authorities especially JKR and/or KPLB will fully implement and execute the design and construction standards suitable for successful implementation of subgrade stabilisation in Malaysia especially for rural road. This will become the novelty in this research, where the development and innovation of the design and implementation are modified and enhanced.

1.7 Gaps of Research

In this research gab, there are some areas that have significant scope for more research, but the Author may or not be described further nor not been investigated, unexplored or underexplored areas by other researchers.

There are 3 types of research gap included in this thesis.

a) Literature Gap

In this literature gap, all information related in the design pavement will be reviewed thoroughly and the final report will show the recommendation for latest updates.

b) Conceptual Gap

As a technical point of view might be different with actual on site. In this thesis will describe the different gap from designer and actual on site. Therefore, conceptual gap analysis may explain further to find the best method to ensure the quality followed as per specification.

c) Methodological Gap.

The whole points in this thesis may consist of methodological data. The data collected also explained the proper process for the entire scope of stabilisation i.e. design, construction and quality control (performance).

Table 1.1
Significant Scope of Research from other Researchers

Author	Title	Objective	Findings	Recommendation
Parnam et. al. (2015)	Soil Stabilisation using Quicklime	To determine the optimum percentage of the mix and obtained the modified characteristics of the stabilised material	Based on the results, the authors found that the optimum percentage for quicklime is 12 %, however, the processes	This research will delft into the modified properties of the stabilised material based on its microstructure and macrostructure.

Author	Title	Objective	Findings	Recommendation
			destabilised the soil characteristics.	
Ausroad, 1998	Guide to Stabilisation in Roadwork	To ensure selection on the suitable binder agents for subgrade stabilisation	That information given/shown in this thesis, the best stabilisation to achieve result as per specification is cementitious products i.e. OPC cement and quicklime.	Few tests and trial have been conducted and results shown according to specification. Meanwhile, those cementitious products i.e. OPC and quicklime are available in Malaysia markets.
JKR, 2019	Standard Specification for Road Works; Section 18: Soil Stabilisation	To ensure the quality and performance followed as per Specification.	There are no identified soil categories in this specification	This thesis will identify based on the 2 identified soil categories i.e. Laterite and Marine Clay.
JKR, 2012	Design Guide for Alternative Pavements Structures. Low-volume Roads	Basic guideline for references only. Basic guideline design calculations	The simple and basic guidelines shown in tables based on the Equivalent Standard Axle Load (ESALs)	Need to improve an accordance to specifications requirement for CBR shall be calculated accordingly.
REAM, 2005	Specification for Cold In-Place Recycling	To get and achieved a minimum result and to ensure quality and performance followed according to specification.	Only highlighted the general binder agents for stabilisation and not for soil/sub grades. Process stabilisation described a simple explanation.	In this thesis, the process of stabilisation will describe deeply to ensure the performance through testing will meet the expectations.
REAM 2020	Best Practice of Sub Grades Stabilisation	To refer the workflow/process doing the soil stabilisation or sub grades stabilisation.	To find out the Standard Operation Procedure and analysed the work performance according to the specifications.	That explanation shown sentence by sentence. This thesis will demonstrate the workflow for easy references and simplified information in next Chapter 3: research Methodology.

1.8 Thesis Outline

The research is outline as below:

i) Chapter One

This chapter includes the sections related to thesis requirements. It comprises the backdrop, problem statement, research objectives, research questions, scope and limitation of the study and significant of the research. All above items are explained extensively to encompass overall study and research plan to progress until the results of this research are achieved.

ii) Chapter Two

The overall explanation in this chapter put forward studies by other researchers and references about the stabilised soil/subgrades. The deliberation covers pertinent areas such as stabilisation technology, categories/types of soil, binder agents, design criteria including mix design until treated soil, process of stabilisation works and quality control. This research also explained about the problems of current/existing soil and how to mitigate the problems. Laboratory test may require ensuring the problem soil area could be reused using stabilisation technology.

iii) Chapter Three

The explanation will be based on the flow chart given for the entire process of stabilisation. The crucial part~~s~~ of this chapter are the data collection and analysis based on 3 methodologies. This includes laboratory test which 2 samples have been selected i.e. laterite and marine clay. The samples will be analysed and described about soil contents in detail. Second method would be designing stage on how to calculate structural design for pavement layer and doing the mix design to identify the best binder agent for weak soil to increase their strength. And finally, the third method is 2 pilot project**s** have been selected as a case study. These 2 pilot projects have been selected to represent each type of soil i.e. laterite and lime. As stated in specification by JKR (JKR, 2017) the minimum results for treated soil required are CBR and UCS.

iv) Chapter Four

Expected results/preliminary result shall be described in this chapter. It will conclude the findings for the entire chapter above. Benefits of stabilisation technology to stakeholders involved will be thoroughly presented to highlight significant contribution to the discipline.

This chapter presents a comprehensive description of the research work and its substantial contribution to the world of engineering study. It delineates the significance of stabilisation technology as follows.

a. Academic Contribution: the research conducted based on the industrial practice and transform into academic documented. The analysed data taken will guide the engineer to identify a testing process and construction process.
b. Stakeholders Needs; the engineering practice nowadays are looking toward practicality with the cost savings. So, consideration must be started from the planning design, construction stage and maintenance stage. This thesis elaborates and analyses data and formulated with the proportion of the binder and a proper machine to ensure results meets according to specifications. There are the stakeholders involved including of Consultants, Local authorities, Governments, Developers, Site supervisory s and Practising Engineers.
c. Environment Sustainability
d. Economic viability
e. Community

v) Chapter Five

This chapter is the summarised from all above chapters and inclusively to puts some conclusion and recommendations. Overall finding for all chapters covers every aspect of design and construction process, analyse, formulated and the performances via standard and examination outcomes.

CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

Subgrade or soil stabilisation is typically utilised in pavement and geotechnical projects. This research will concentrate on pavement construction utilising cementitious binders and mechanical stability. Subgrade or soil stabilisation is predominantly utilised in the pavement layer of road building. Stabilisation rectifies and enhances the weaknesses in existing minerals, enabling the utilisation of otherwise inappropriate materials to create a temporary base for machinery operating in that area. Subgrade or soil stabilisation is a technique that involves mixing existing soil or materials with additives, either in situ, to enhance strength, create a more solid surface and improve bearing capacity.

According to BS1377-1:1990 (BSI, 1990), soil is defined as "an assemblage of discrete particles forming a deposit, typically of mineral composition but occasionally of organic origin, which can be separated by gentle mechanical methods and contains varying amounts of water and air (and sometimes other gases)." Soil typically comprises a naturally occurring deposit that is part of the earth's crust; however, the phrase also includes engineered ground constructed from replacement natural soil or artificial materials that demonstrate comparable properties, such as crushed rock, crushed blast-furnace slag and fly ash. In this research, stabilisation refers to the enhancement of subgrade/soil or pavement material by incorporating a minimal quantity of binder or additive together with water.

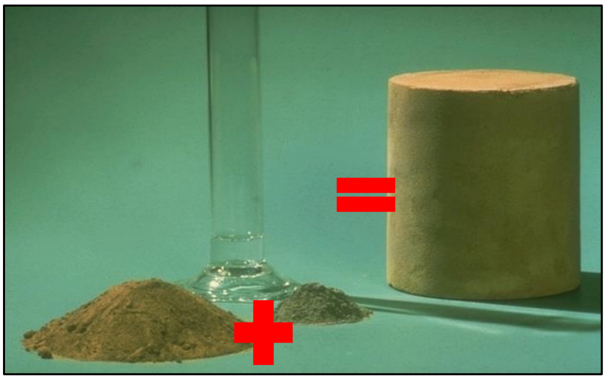

Figure 2-1 Typical Mix of Laterite Soil with Cement to Form a Cylindrical Sample

(Source: Stabilised Pavement (Malaysia) Sdn. Bhd.)

The primary aims of soil stabilisation are outlined as follows.

i) To enhance the current sub grade California Bearing Capacity (CBR) value.
ii) To enhance the strength of the current pavement layer for soil/material.

The essential considerations when selecting the best appropriate stabilisation method or technique are as follows (JKR, 2019). According to JKR (2019), factors contribution the selection of method of stabilisation as follows.

i) Condition of the current soil/material designated for stabilisation/treatment.
ii) Climatic conditions.
iii) Category of stabiliser. Proposed quantities for the utilisation of the stabilising soil/material.
iv) The competencies and expertise of the construction workforce.
v) The appropriateness and accessibility of machines and stabiliser.
vi) The availability of testing facilities for pre and post construction including investigations and subsequent quality control.
vii) Relative Costing Analysis.

The procedure for stabilising actions shall encompass as follows.

i) Assessment of soil/material properties and formulation of mix design.
ii) When necessary, disintegrating and/or grinding existing material via a suitable equipment.
iii) When necessary, altering the material properties of the current soil/material by the incorporation of imported materials and binding agents. Natural or imported materials (such as sand, gravel, laterite, etc.), crushed stone products (including graded products and crushed dust), by-products (like steel slag and fly ash) and/or suitable salvaged or recycled materials from any site must be combined with the stabilisation material to modify the grading of the stabilised material and enhance its properties.
iv) The provision and application of stabilising with binder agents and/or water; and
v) Mixing with suitable machinery, placing with binder agent, compacting with a vibro-roller compactor and shaping the stabilised material to form a new layer using a motor grader.

The newly stabilised layer must adhere to the specifications regarding lines, grades, thickness and standard cross-section as depicted in the drawing and/or outlined in the detailed design report (JKR, 2019).

2.2 The procedure for Cementitious Binder for Sub Grade Stabilisation

Subgrade stabilisation involves the incorporation of either cement or extra cementitious materials for enhancement. The combination and interaction of cementitious materials consist of blends of pozzolanic substances with lime, fly ash, or cement. The substantial response of cementitious stabilising agents occurs when they interact with moisture in the subgrade or soil. The reactions were contingent upon the characteristics of the subgrade/soil, which necessitated the use of cementitious materials encompassing a diverse array, from cohesionless sands and gravels to silt and low plasticity cohesive substances.

Lime can be obtained in the form of quicklime or hydrated lime. Quicklime is manufactured by calcinations of limestone at high temperatures, hence chemically transforms calcium carbonate into calcium oxide. Hydrated lime is crated when quicklime chemically reacts with water. For the lime treatment to be successful, the clay content of soil should not be less than 25 % and the sum of the silt and clay fraction should preferably exceed 35%, which normally the case when plasticity index of the soil is larger than 10. (JKR/SPJ/2019-S18, 2019).

Cement used in the soil stabilisation process shall be complying with the relevant Malaysia Standard (MS) and suitable for soil stabilisation purpose. The use of any other class or type of cement shall be only considered if the modified cement can be demonstrated that it will provide a cost benefit solution without any technical detriment in the work. (JKR/SPJ/2019-S18, 2019).

2.2.1 Types of Cement

'Cement' is a fundamental term utilised to describe a broad range of organic and inorganic binding agents. The predominant binding agents are hydraulic cements—finely ground inorganic substances that exhibit a robust hydraulic binding capability, particularly when combined with water, resulting in a stable and lasting product.

The objective of cements derived from a combination of calcium carbonate, alumina, silica and iron oxide is to create a new set of chemical compounds by the reaction of calcined and sintered materials at elevated temperatures, which can then react with water.

2.2.1.1 General Purpose Ordinary Portland Cement (OPC)

OPC is characterised in AS3972 (Ausroads, 1998) (Portland and Blended Cements) as: "a hydraulic cement produced as a uniform product by grinding Portland cement clinker and calcium sulphate, at the manufacturer's discretion, which may include up to 5 % of mineral additives."

2.2.1.2 General Purpose Blended Cement.

Blended cement is characterised as a hydraulic cement that includes Portland cement plus a proportion of one or both subsequent components shall be greater than 5 % of fly ash or ground granulated iron blast furnace slag, or both and up to 10 % silica fume.

Both types of cement are utilised in stabilisation; however, there is a growing inclination towards the use of mixed cements because of their increased setting times. Increased setting times allow longer working times for achieving a smoother riding surface and closely spaced cracks from early trafficking which minimise the risk of shrinkage and cracks (Ausroads, 1998).

2.2.2 Lime

In Oxford dictionary lime can be defined as a white caustic alkaline substance consisting of calcium oxide, which is obtained by heating limestone and which combines with water with the production of much heat of quicklime.

In British Lime Association (1990) can be classified in 2 definitions as Lime Modification – "where an immediate improvement in workability, peaceability and compatibility of the soils is achieved" and Lime Stabilisation – "where an additional long-term improvement in strength and resistance to adverse weather conditions is obtained.

2.2.3 Pozzolans

A pozzolan is a siliceous or alumino siliceous material that, in finely divided form and in the presence of moisture, chemically reacts at ordinary room temperature with calcium hydroxide released by the hydration of Portland cement or lime to form compounds possessing cementitious products (AustStab, 2015 2nd Edition).

2.2.4 Bituminous Based Stabiliser

Bituminous based stabiliser can be used in soil stabilisation provided it can be proven to achieve the required strength and safety requirements.

2.2.5 Fly Ash

Fly ash is a product of the power generation industry. The type of coal used and the mode of the operation of the plant determine the chemical composition and particle size distribution.

Consequently, not all fly ashes are suitable for stabilisation. Generally, fly ash derived from burning black coal is high in silica and alumina and low in calcium and carbon and is well suited for use in stabilisation. Fly ash produced derived from burning brown coal contains large percentages of calcium and magnesium sulphates and chlorides and other soluble salts and is unsuitable for use in stabilisation. (AustStab, 2015 2nd Edition).

2.3 Stabilised Materials Characteristics

2.3.1 Reaction of Sub Grade/Soil with Cementitious (cement/lime) Agents

The hydration reaction of the cementitious agent (cement/lime) with water in the soil results in the creation of cementitious materials, specifically calcium silicate and aluminium hydrates as in concrete. These responses transpire nearly independently of the soil's characteristics.

The hydration process produces hydrated lime, constituting around 30 % by mass of the applied cement in general-purpose cement stabilisation, which may induce subsequent reactions with any pozzolans present in the soil. The secondary reaction generates cementitious products analogous to those produced by the initial reaction. (AustStab, 2015 2nd Edition).

The hydration reaction commences instantaneously upon the interaction of the cementitious stabilising agent with water. The process accelerates with the addition of cement, resulting in substantial strength increases within the first day. The secondary reactions generated by cementitious materials resemble those in lime stabilisation and progress gradually over time.

Pozzolanic reactions often progress slowly but persist for an extended duration, contingent upon the availability of sufficient moisture. Reactions are also temperature sensitive, with the rate of reaction increasing with the increasing temperature. Organic material and sulphates may cause retardation of the reaction. The optimal outcomes of stabilisation utilising extra cementitious materials are contingent upon the quantities of lime, pozzolan and pavement material. A volumetric ratio of around one part lime to two parts fly ash will yield optimal strength of the paste. The combined quantity of lime and fly ash incorporated into a pavement composition should not exceed about 5 % by mass of the pavement material. These ratios must be validated through testing. (AustStab, 2015 2nd Edition).

2.3.2 Types of Cementitious-Stabilised Materials

According to Ausroad, 1998. Soil characteristics gradually alter with elevated cementitious binder content. The various properties obtained from different cementitious binder contents require a variety of assessment methods to ensure proper utilisation. Although the continuity of features is highlighted, two primary kinds of cementitious stabilised materials can be delineated as follows:

i) Modified Materials - minimal quantities of cement are utilised, resulting in materials that can be classified as unbound granular materials for pavement design applications.

ii) Bound Materials - materials whose stiffness and tensile strength are significantly improved through the incorporation of cement, rendering them suitable for practical applications in pavement stiffening. This is regarded as an element of the pavement design process. These bonded items are frequently classified as either lightly bound or heavily bound.

Table 2.1
Characteristic Properties of Modified, Lightly Bound and Heavily Bound Materials

Type of Material	**Thickness (mm)**	**Design Strength (MPa)[1,2]**	**Design Modulus (MPa)**
Altered/Modified	Suitable for any thickness	UCS ≤ 1.0	≤ 1,500
Lightly Bound	Generally ≤ 250mm	UCS: 1-4 (7 days strength: 1-7)	15,000 – 20,000
Heavily Bound	Generally > 250mm	UCS ≥ 4	2,000 – 20,000

[1] 28 days test results
[2] The test findings for slow-setting binders will be lower than the indicated values but will persist in increasing in the field for a duration of 6 to 12 months.
(Source: AustStab, 2015 2nd Edition).

The incorporation of cementitious binder materials is typically employed to diminish moisture in weak soils and enhance shear and bearing strengths, without substantially augmenting tensile strength and modulus. The altered material often displays a densely arranged network of fine fractures and does not form a network of widely spaced open fissures as observed in certain bound materials. Modified materials are classified as unbound materials in pavement design (AustStab, 2015 2nd Edition).

Lime is usually a better additive for modifying materials, especially those with high plasticity, as cementitious binders have tendency to bind particles together even at very low additive contents. Bound materials are engineered to endure tensile stresses exerted by traffic and environmental factors over the pavement's intended lifespan. Bound materials are defined for pavement design by their elastic modulus and Poisson's ratio (often assumed to be 0.20), with performance assessed based on fatigue resistance (AustStab, 2015 2nd Edition).

2.3.3 Properties of Cementitious - Bound Materials

The subsequent sections elucidate the impact of cementitious binders on soil characteristics. As the effect of cementitious binders on soils relies on the type of binder, the type of soil and its environment, these considerations should be treated in broad terms only and testing for individual situations will be required.

Minor increments of cementitious binders substantially influence the volume stability of expansive materials without necessarily resulting in considerable strength enhancements. Cementitious binders significantly diminish moisture-induced shrinkage by binding the particles. Consequently, whereas plastic qualities are commonly employed to categorise the volume stability of pavement materials, they are unsuitable for cementitious-bound materials (AustStab, 2015 2nd Edition).

There are two primary types of cracking in cementitious-bound materials:

i) Shrinkage cracking due to hydration and drying, as well as cracking originating from inside the layer, is mostly an issue in base courses, although reflection cracking from underlying layers may also occur. The interplay of elevated shrinkage and tensile strengths in cementitious-bound bases may result in the formation of widely spaced (typically 0.5 to 5.0 m) transverse and/or block cracking. While this may diminish the ride quality of the pavement, it often does not result in significant structural issues, providing the cracks are narrower than 2 mm and are sealed or treated with a polymer-modified or scrap rubber binder wearing surface. Geotextile-reinforced seals have been effectively employed to alleviate the impacts of cracking (AustStab, 2015 2nd Edition). The use of minimum asphalt at 50 mm will mitigate the impact loading of vehicles on the cracks, hence minimising surface cracks. Failure to seal any cracks may result in moisture infiltration into the pavement, potentially causing the displacement of particles due to erosion and accelerated degradation of the pavement's loads. Even with substantial cracking, if moisture infiltration into the pavement is mitigated, the pavement may retain considerable longevity in the post-cracking phase, where the cementitious-bound layer functions as either an unbound or modified layer. This is especially true if the layer serves as a subbase layer (AustStab, 2015 2nd Edition). Early trafficking of cementitious-stabilised layers is advantageous as it promotes the formation of closely spaced tiny cracks, which are more manageable than the widely spaced large cracks that often arise in the absence of early trafficking (Yamanouchi, 1975). The improved utilisation of slow-setting cementitious binders, such as OPC cements and supplementary cementitious materials, combined with precise curing methods, early trafficking and sophisticated crack sealing and wearing surface technologies, can alleviate the adverse effects of shrinkage cracking in cementitious-bound bases.

ii) Fatigue Cracking. Fatigue cracking transpires when the frequency of tensile strain repetitions imposed on the cementitious-bound layer by vehicular traffic surpasses the layer's capacity. The primary criterion for designing cementitious bound pavement courses is the prevention of fatigue cracking (AustStab, 2015 2nd Edition). Thin cementitious-bound pavement layers with a high modulus are especially susceptible to fatigue cracking. The fatigue resistance of cementitious-bound layers is highly dependent on the thickness of the course and the stiffness of the layer. As an approximation, a 10% reduction in either the thickness or stiffness of a cementitious-bound layer could lead to about 90% reduction in fatigue life. The thorough incorporation of the correct binder content, adequate compaction, suitable construction tolerance, thickness and curing are therefore all critical to the fatigue performance of cementitious-bound materials. The use of lightly bound cementitious stabilised layers also greatly reduces the possibility of any adverse effects of shrinkage cracking. (AustStab, 2015 2nd Edition).

2.4 Physical Strength Parameters/Characteristics

2.4.1 Shear and Bearing Strength

Shear and bearing strength assessments, such the Modified Texas Triaxial Test and the California Bearing Ratio (CBR) test, have some application to modified materials but little application to bound material. Small addition of cementitious binders in well-graded granular materials often result in substantial and insignificant increases in the outcomes of these tests. Both tests are ultimate strength assessments and therefore evaluate parameters at elevated strains, much above the operational strains in cementitious-bound materials (AustStab, 2015 2nd Edition).

The predominant specification test for assessing the strength of cementitious-bound materials is the Unconfined Compressive Strength (UCS) Test. This test is rather straightforward, although the outcomes fluctuate significantly based on the curing and conditioning of the samples. The curing conditions employed for the UCS test should strive to replicate probable field situations. The UCS test findings can provide a preliminary estimate of the modulus of cementitious-bound materials (AustStab, 2015 2nd Edition).

2.4.2 Tensile Strength and Strain

Tensile strength and strain tolerance are crucial in the design of bound pavement layers. Cementitious-bound materials (heavily bonded) exhibit considerable brittleness, characterised by a high modulus and minimal tensile strain at failure. Their behaviour under triaxial compression aligns with the Mohr-Coulomb theories (shear failure criteria) for modified materials and more closely adheres to the Griffith crack theory which materials break because tiny crack inside then grows bigger when the stress is high enough or its variations for bound materials. The failure of cementitious-bound materials under traffic loading aligns with fatigue tensile failure (AustStab, 2015 2nd Edition).

2.4.3 Elastic Properties

The elastic properties necessary for pavement design are the elastic modulus and Poisson's ratio. Cementitious-bound materials are isotropic in the context of pavement design. The typical moduli for well-graded cementitious-bound materials intended for use as base materials range from 2,000 to 20,000 MPa, in contrast to approximately 200 to 500 MPa for unbound materials. Nevertheless, fine-grained materials typically exhibit lower moduli compared to well-graded materials. The moduli of cementitious-bound materials exhibit minimal sensitivity to temperature variations. Nonetheless, moisture sensitivity may be significant.

2.4.4 Variation with Age

After rapid strength gains in the first one or two days, cement-bound materials continue to gain strength slowly, provided curing is sustained. Elastic modulus also increases with age. The rate of gain of strength with materials stabilised with supplementary cementitious materials is less than that for cement.

The variability of properties with time is quite dependent on both type of material and binder type and should be evaluated by testing for individual situations. (AustStab, 2015 2nd Edition).

2.5 Conditions Suitable for Cementitious Stabilisation

Two primary aspects must be considered for cementitious stabilisation:

2.5.1 Material Considerations

Materials with a low void volume when compacted typically necessitate the incorporation of minimal cement compared to poorly graded materials with large void content. Specific material components can significantly hinder the development of stabilised materials. Sugars and reactive organic chemicals can impede the hydration process and inhibit the hardening of the binder. Under some conditions, soluble sulphates might lead to the deterioration of the hardened paste (AustStab, 2015 2nd Edition).

2.5.2 Production Considerations

The quality of the compacted stabilised material is highly contingent upon the production method, which include:

i) Breakdown and Mixing

A uniform distribution of binder over the whole design depth of the bound material is essential to ensure performance aligns with design specifications. An even distribution of binder guarantees its optimal use and yields the most consistent material characteristics. It is customary to augment the laboratory-determined binder content (often by 0.5 to 1.0 %) to account for mixing inefficiencies. The impact of these changes must be examined during the laboratory testing protocol (Ausroads, 1998).

Both plant mixing and in-situ mixing can yield high-quality stabilised mixtures with adequate control and may be specified when superior materials are necessary. Two passes of the stabiliser are necessary for optimal mixing outcomes in field mixing. Further passes of the stabiliser may generate excessive fines and adversely affect the stabilisation process.

The necessary degree of breakdown is typically a concern solely with cohesive materials like clays and sandy clays. When these materials are utilised for pavement layers, often limited to the subbase level, a significant degree of disintegration is preferred. This also facilitates the diffusion of lime products from the cementitious binder into the aggregates, hence improving stabilisation.

High early strength is frequently unnecessary for subgrade materials if moisture susceptibility is mitigated. Subgrade materials must be fragmented to ensure that a significant amount of the clay aggregates passes through the 19 mm screen.

Pre-conditioning using minimal quantities of hydrated lime or quicklime frequently enhances the disintegration of polymeric clay soils. This proposal should be incorporated into the laboratory testing program (AustStab, 2015 2nd Edition).

ii) Water Quality

Water for cement stabilisation must be drinkable, devoid of organic matter and contain less than 0.05% sulphates. Whenever feasible, the actual water source intended for field usage should be utilised in the laboratory testing program.

The water source for the curing of cement-stabilised products must also be evaluated. Exercise caution while utilising saline fluids for curing, as they may lead to an accumulation of surface salts that can disrupt the adherence of subsequent seal coats (AustStab, 2015 2nd Edition).

iii) Compaction

Enhanced resistance to compaction arises from the fast development of cementitious linkages that counteract the applied compaction force. Compaction should be executed promptly following the incorporation of the binder. Delays in compaction following the incorporation of binder and water, especially when the binder is cement, will lead to diminished densities and hence lower strengths (AustStab, 2015 2nd Edition).

iv) Curing Conditions

Curing is essential to guarantee that:

The requisite strength is attained in the stabilised material which is consistently sufficient water for the hydration reactions to occur; and drying shrinkage is constrained while the hydration reactions are ongoing and the material is gaining strength (AustStab, 2015 2nd Edition).

Due to the continual loss of water to the atmosphere, it is advisable to apply a curing membrane, such as a prime or primer seal, to prevent the stabilised materials from drying out prior to the completion of the hydration reactions. If impractical, the pavement must be moist for up to 7 days with frequent surface irrigation or by overlaying with an additional pavement layer. It is crucial to avoid excessive watering of cementitious stabilised layers to prevent leaching. Premature drying inhibits the formation of robust cementitious linkages and significantly exacerbates the severity of shrinkage cracking. Under these conditions, the material will exhibit low tensile strength and an excessive number of closely spaced, broad cracks, inhibiting the development of substantial slab action to counteract traffic loads (AustStab, 2015 2nd Edition).

Curing is essential for laboratory specimens and should replicate probable outdoor settings; samples must be stored in sealed, airtight bags at a consistent temperature. The efficacy of accelerated curing at elevated temperatures must be proved through comparison with standard curing conditions.

2.6 Consideration and Selection Type of Binder Agents

A diverse array of cementitious binders is appropriate for application in stabilisation technology. The contemporary trend of creating thicker stabilised layers favours the utilisation of cementitious materials with extended setting times. This allows us additional time to attain compaction and a high-quality riding surface on stabilised layers.

Using modern construction equipment, deep lifts of cemented base layers (range between 250 to 300 mm thick) can be compacted in a single layer. Under these conditions, theoretically the bottom one-third of the layer may have a relative density of up to 5% less than the top two-thirds. As a result, the UCS and modulus of the bottom one-third can be approximately half that of the upper two-thirds of the layer. This is a normal and acceptable part of the construction process, but it can influence pavement performance. Testing of these types of pavements under accelerated loading conditions has indicated that fatigue performance was not reduced because of this effect (Jameson et al. 1995).

2.6.1 Application of Cementitious - Bound Materials

The application cementitious at pavements layer including of sub grades, sub-base, base course and wearing surface for bound material may contributes the changes of the characteristics and functionality to ensure the results as per designed or specifications.

i) Sub-grade

Sub-grade stabilisation is typically performed out to enhance subgrade strength and to furnish a functional base for construction machinery. Therefore, it can transform subgrade quality material to lower subbase quality. Sub-grade stabilisation can mitigate construction issues related fluctuating sub-grade strengths and to establish a water-resistant subbase for permeable or jointed pavements, acknowledging that this may induce permeability reversal inside the pavement which must be considered into design.

ii) Subbase

Sub-base stabilisation can be employed to enhance the quality of existing materials for utilisation as a modified subbase. Augment the working platform characteristics of a sub-base and to decrease the overall pavement thickness and enhance the pavement design; and supply a material with less moisture susceptibility. Cementitious-bound materials are applicable in heavy-duty pavements to create a rigid, load-bearing layer. Generally, these are positioned at a depth within the pavement that minimises reflection cracking. An advantage of employing a cementitious-bound subbase rather than a cementitious-bound base is the reduction of reflection cracking from the subbase to the wearing surface. The application of 150 mm of unbound granular basecourse has proven, in numerous instances, to be an effective method for postponing reflection cracking (AustStab, 2015 2nd Edition).

iii) Base-course

Cementitious-treated materials may serve as either modified or bounded materials in base-course. Modification might be employed to enhance a marginally inadequate material, to improve low cohesiveness base materials that distort and shove when subjected to traffic and to mitigate moisture sensitivity.

Additional caution must be used when mixing low binder contents to achieve uniformity and reduce variability in material qualities. Failure to do so may result in reflection cracking of the pavement surface, permitting moisture infiltration into the pavement. The materials may frequently become vulnerable to pumping and erosion when exposed to moisture and traffic; and may fail to sufficiently rectify structural deficiencies in the pavement.

Cementitious-bound bases may be beneficial in the sequence applications which are moderately to intensively trafficked highway on low-strength subgrades. By enhancing the load-bearing capacity of a pavement, especially in situations when options may be constrained by elevation and/or the existence of utilities. The areas prone to recurrent flooding and where life-cycle expenditure be reduced.

When constructing cementitious-bound base materials in many layers, it is crucial to ensure complete bonding to attain sufficient shear strength between the layers; otherwise, there would be a substantial decrease in fatigue resistance. Techniques that can be utilised to attempt to connect cementitious-bound pavement layers include the use of cement slurry or a bituminous seal interlayer (Kadar et al. 1986). Nonetheless, if feasible, the fabrication of cementitious-bound layers in a single application is advantageous to mitigate the danger of debonding and consequent performance degradation.

iv) Surface Layer

Utilisation and operability. Adhesion issues may arise between sprayed sealants, asphalt and the surfaces of some cementitious-bound materials. This is mostly attributable to the sealing binder's incapacity to infiltrate the surface of the bound components.

CHAPTER 3
RESEARCH METHODOLOGY

3.1 Introduction of Research Methodology Process

This research will emphasise the 3 approaches related to the research objectives consist of soil analysis with mix design, construction process and evaluate the performances. This will conclude the total process until completion of stabilisation work including quality assurance and quality control. The process will be divided into 3 phases including of site investigation, design and construction. (Refer to flow chart in Appendix 3)

There are Two (2) types of methodology approach for this thesis.

3.1.1 Case study of Soil Stabilisation through Pilot Project Preferences

Two (2) projects involved for this report i.e. Landfill Project at Seberang Prai, Penang and Sungai Buloh Interchange Project, to produce results followed as per mix design using two (2) types of soil i.e. laterite and marine clay with mixture of cement and/or lime. Finally, able to access the performance of the stabilisation works.

3.1.2 Laboratory Testing

Those samples collected will be send to laboratory at College of Engineering, UiTM Shah Alam. The process will be involved including materials (soil) classification, mix design and initial test result. Therefore, for comprehensive laboratory tests shall be executed at project's site laboratory.

3.2 Physical Stabilisation Process Characteristic

Physical stabilisation process characteristic is crucial and will affect the final outcomes from the design calculation.

3.2.1 Mix Design

Specifications for materials, application rates of the stabilising agent, types and application rates of necessary additives, particular pre-treatment requirements and the compacted thickness of the stabilised layer must be established prior to the commencement of the job (JKR, 2019). Initial Laboratory analyses of materials collected from the intended site and/or borrow areas shall be conducted to ascertain the fundamental qualities of the soil/material before stabilisation. The collected samples must accurately represent the material designated for stabilisation in the specific area. The current soil/material is considered appropriate for stabilisation under the subsequent condition for CBR value is below 5% for T1 – T3 and below 12% for T4 & T5. The traffic categories T1-T5 can be referenced in ATJ 5/85 (Amendment 2012). The Plasticity Index exceed 10. The proportion of fine particles exceed 25%.

Samples from the trial pit will also be used to carry out the mix design to determine the parameters including of depth of pavement layer to be stabilised. Types and quantities of stabilising agents. The strength achieved by the new road base layer after stabilised and an optimum moisture content and dry density.

The design mix is to determine by identifying the type and quantity of stabilising agent, grade and quantity of stabilised pavement layer material. The combination of all stabilised materials and stabilising agents should achieve the specified minimum strength.

The initial laboratory testing will establish the appropriateness of the current soil/material for stabilisation. Further consideration shall be given to appropriate soil/material (JKR, 2019).

For portions deemed feasible for stabilisation, subsequent laboratory tests shall be conducted as follows.

1. The characteristics of the soil/material designated for stabilisation.
2. The type and ideal application rate of the stabilising agent.
3. The quantity and classification of any necessary additions.
4. The volume of supplementary material to be imported from designated borrow locations (JKR, 2019).

3.2.2 Pavement Evaluation and Assessment

Paving assessment is necessary before stabilisation work is carried out at a location to determine the suitability of the selected site. Among the things that need to be identified in the assessment of the pavement are as follows:

i) The structure of the existing pavement layer
ii) The mortar grade of the existing pavement layer on the site
iii) The plasticity index of the current pavement layer on site
iv) Moisture content of the existing pavement layer on site

Assessment of pavement at the proposed stabilisation site including of trial pit - for the determination of the above parameters is made by conducting laboratory tests on the samples taken. Dynamic Cone Penetrometer Test - determination of the structure of the existing pavement layer.

3.2.3 Laboratory Tests Conducted on Samples Taken Through Trial Pits:

For design mix process, there would be a few basic steps for laboratory test for materials sample.

i) Sieve analysis to determine the grade of mortar

A particle size distribution study is an essential classification test for soils and granular materials, particularly coarse-grained soils, as it delineates the relative proportions of various particle sizes. This allows for the identification of whether the material mostly comprises gravel, sand, silt, or clay sizes and to a limited degree, which of these size ranges is likely to influence the engineering qualities of the material. BS1377: Part 4: 1990.

Determining particle size in large aggregate samples is essential to guarantee that aggregates function as intended for their designated use. A sieve analysis or gradation test assesses the size distribution of aggregate particles in a specific sample.

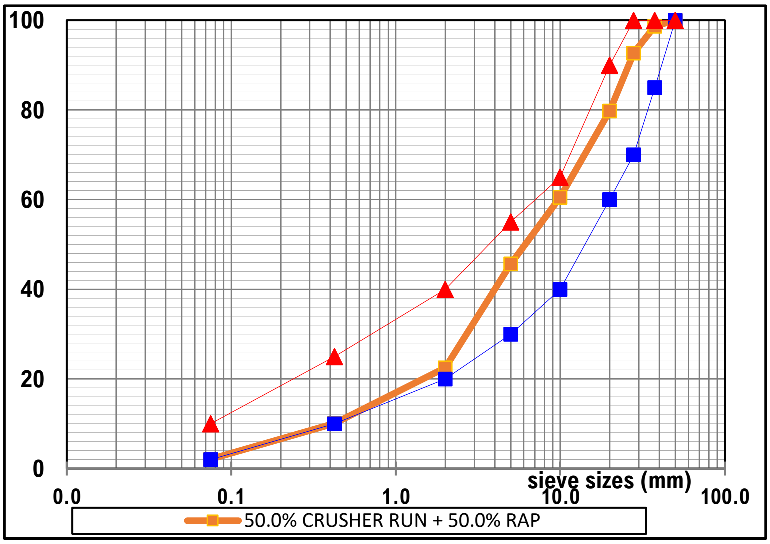

Figure 3-1 Sieve Analysis for Aggregate Segregation

ii) Sieve analysis to determine the grade of mortar

The Proctor compaction test is a laboratory procedure for empirically establishing the optimal moisture content at which a specific soil type attains maximum density and achieves its highest dry density. Method employing 4.5 kg rammed (BS Heavy) existing materials; in this test, the competing effort exceeds that outlined in the preceding test. The mass of the rammer is elevated to 4.5 kg, the drop height is raised to 450 mm and the number of compacted layers is augmented from three to five. The test is conducted on material that passes through the 20 mm test sieve utilising the 1 litre mould, or on material that passes through the 37.5 mm test sieve employing the CBR mould as previously mentioned. The sample preparation and testing procedure are identical to the test, with the exception noted.

The rammer weighs 4.5 kg and has a drop height of 450 mm. The damp earth will be compressed in five uniform layers within the mould. Each layer shall receive 27 strikes. Utilise the vibration-rammed machine; each layer must be vibrated for one minute. BS1377: Part 4: 1990.

iii) Aggregate Crushing Value (ACV)

Aggregates utilised in construction must possess sufficient strength to withstand crushing forces. Weak aggregates are likely to negatively impact the stability of the structure. The strength of coarse aggregates is evaluated by the aggregate crushing test. The aggregate crushing value offers a comparative assessment of resistance to crushing when subjected to progressively applied compressive force. A lower aggregate crushing value indicates a stronger sample of aggregates. If the aggregate crushing value is 30 or greater, the result may be anomalous; thus, the ten percent fines value should be assessed instead. BS1377: Part 4: 1990

iv) Flakiness Test

The Flakiness Index represents the weight percentage of particles whose minimum dimension is less than three-fifths of their average dimension. The Elongation Index is the weight proportion of particles whose maximum dimension exceeds one and four-fifths times their average dimension. BS1377: Part 4: 1990

v) Soaked California Bearing Test (CBR) Test

The California Bearing Test (CBR) is a penetration test used to assess the mechanical strength of natural ground, subgrades and foundation courses under new roadway construction.

The CBR rating was established to assess the load-bearing capacity of soil utilised in road construction. The CBR can also be utilised to assess the load-bearing capacity of unimproved airstrips or the soil beneath paved airstrips.

The subgrade's strength is the primary determinant of the necessary thickness of flexible pavements for highways or airfields. BS1377: Part 4: 1990

The CBR test will be conducted on material that passes through the 20 mm test sieve. If the soil includes particles exceeding this size, the portion retained on the 20mm test sieve must be extracted and weighed prior to the preparation of the test sample. If this fraction exceeds 25%, the test is deemed unacceptable. BS1377: Part 4: 1990.

The soaked CBR value requires the soil sample to be immersed in water for a minimum of four days before the test. Consequently, doing a saturated CBR test necessitates a minimum of five days, rendering it a time-consuming and laborious procedure. In the road construction project, it is necessary to ascertain the soaked California Bearing Ratio (CBR) values of the subgrade soil at regular intervals throughout the project's length to evaluate its strength.

Laboratory tests on the stabilised material shall be conducted to ascertain the application rate of the binder agent. Specimens must be manufactured at the ideal moisture or fluid content for the binder agent, as indicated by the optimal moisture content of the untreated material established in AASHTO T 180 (modified moisture density relationship) or BS 812 / BS 1924.

The adjustment of the specimen's moisture content to achieve the optimal moisture/fluid content for the binder agent shall adhere to established best practices.

The scope of work for the content of the stabilising agent shall be established based on testing, together with all other mix specifics for quality control, including target moisture content and density.

vi) Unconfined Compression Strength (UCS) Test

The Unconfined Compressive Strength (UCS) is a straightforward laboratory testing technique used to evaluate the mechanical properties of rocks and fine-grained soils. It offers an assessment of the undrained strength properties of the rock or soil. The unconfined compression test is frequently incorporated into the laboratory testing regimen of geotechnical investigations, particularly in the context of rock analysis. BS1377: Part 4: 1990.

UCS Test for design mix specification requirement is must more than 2 until 5 N/mm^2 for strength with age a cube curing is 7 days for base course layer.

Samples for the assessment of Unconfined Compressive Strength (UCS) shall prepare with 3% and 4% cement contents with the OMC already determined. A 150 by 150 mm split mould is used. The specimens were compacted in accordance with methods describe in BS 1881: Part 108. The specimens should be cured and tested compression in accordance with test 11, BS 1924 or BS 1881: Part 116.

3.2.4 Trail Section

Preferable a trial section shall be identified and constructed in which the existing soil/material exhibit different properties and different design requirements. Every approved trial section must exhibit adherence to the specifications and guidelines or references during the building phase. At a minimum, the subsequent details must be given.

i) Specification of compaction requirements, including the type of vibro-roller compactor and the rolling pattern necessary to get the desired density.
ii) To establish the strength of the stabilised soil/material in accordance with stipulated quality control standards for soil stabilisation, measured by UCS and CBR.
iii) Compacted thickness of stabilised soil/material as delineated in comprehensive contract agreement.
iv) In-situ moisture content to ascertain the water application rate necessary to attain the Optimum Moisture Content (OMC).
v) Proportion by weight of the stabilising chemical utilised as delineated in the quality control specifications for soil stabilisation.
vi) Rate of progression of the stabilising machinery as delineated in the quality control specifications for soil stabilisation.

The trial section must have a minimum length of 50 meters at the specified laying width for quality control sampling. If the trial lay results fall within the acceptable tolerance, they shall be classified as Job Standard Mix (JSM) and will be applicable to the referred project. The trail segment that meets the specification requirements shall be accepted and approved as part of the permanent works, allowing the site supervisory to proceed with that specific piece.

3.2.5 Quality Control Requirements for Soil Stabilisation

Quality assurance and control for soil stabilisation is required to ensure the performance able to sustain as per design requirements. Below table describes the preference for parameter, test method, frequency and minimum acceptance.

3.3 Soil Stabilisation Activities

To ensure appropriate controls are in place for the construction phase using the stabilisation method, JKR’s Standard Specification for Roadworks provides sound guidance for all aspects related to the process. As below Figure was illustrated a cross section of the mixing procedure.

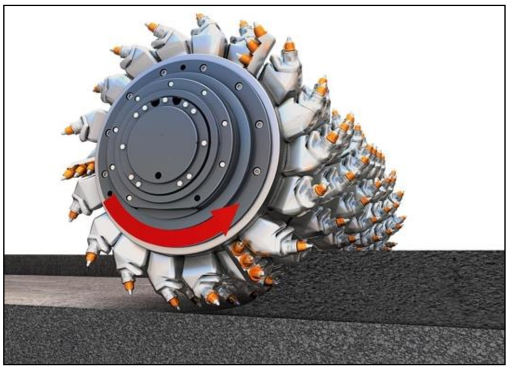

Figure 3-2 Mixing Process

The project photographs below demonstrate examples of soil/subgrade stabilisation that is carried out all over the world, with most developed, developing and underdeveloped nations choosing to stabilise subgrade materials.

Figure 3-3 Mixing Process for Subgrade Stabilisation Projects

The process of stabilisation can be applied to the rehabilitation of existing pavements where unsuitable subgrades are required to be improved, as well as on greenfield sites where the same condition exists (JKR, 2017)

3.3.1 Production Levels

It is well recognised that in-situ stabilisation offers a fast rehabilitation and new construction solutions to practitioners. In some instance, unrealistic production rates are assumed. The factors affecting production levels are:

i) Number of items of equipment and their size/power. One stabiliser may require two water tankers and three sets of rollers to achieve optimum production rate.
ii) Width of stabilisation (e.g. full road width or lane-by-lane and whether the whole road can be closed to traffic.
iii) Depth of stabilised pavement – deep-lift pavements with up to three spreading operations will reduce output.
iv) Thickness of asphalt and heavily bound layers and patches. When the asphalt is greater than 70 mm in thickness, a profiler is used to blend the material or occasionally to remove the asphalt. Many patches can slow production.

Pre-milling may be completed on a day before stabilisation. Where construction tolerance is "tight", mixing and other processes would slow down considerably to minimise the potential to repeat operations (REAM, 2020).

i) The stiffness of original material – i.e. previously stabilised layers will slow the mixing operations.
ii) Weak subgrades will slow the compaction process.
iii) Coarse materials used in the mixing process will slow trimming operations.
iv) Inclement weather after the commencement of work.
v) Longitudinal profile of the road, where there are steep sections, steep cross-falls (> 6%), intersections and where intermittent guard rails narrow the road profile.
vi) The extent of traffic control – greater productivity can be achieved when work is not restricted by traffic control measures.
vii) Services – where unidentified services are encountered/damaged during stabilising and time is lost while they are relocated/repaired. Many manholes cover also reduce production levels and require the operator to stop and start.
viii) Breakdowns of equipment will reduce production levels unless replacement equipment is on site.
ix) Location of site and source of binder and on-site storage facilities.
x) Cooperation of owner/occupier of adjacent properties.
xi) Availability and location of water source and water tanker capacity

Experience indicates that standard output levels for stabilisation depths up to 200 mm range from 3,000 to 5,000 m^2/day, whereas for depths exceeding 200 mm, rates of 2,000 to 4,000 m^2/day are anticipated. These rates represent projections for a single set of plants (REAM, 2020).

3.3.2 Site Preparation, Pumping and Drainage

Site preparation is very important activities on site and may cause a delay to site activities and affected on the overall work program. Below are activities shall take into consideration during site preparation.

a. All bushes, vegetation and other unwanted material shall be disposed offsite.
b. Natural subgrade must be compacted to at least 95% of the maximum dry density.
c. Appropriate drainage if applicable shall be constructed in the form of drainage channels, ditches, curbing, culverts, subterranean drains, in slope, out slope, road crowning design, rolling dips or other approved methods to ensure water is channelled away from the road system and inherent erosion of water runoff is contained.
d. During the construction phase, the Site supervisory should provide and maintain sufficient equipment and facilities to prevent water intrusion into the construction area, thereby ensuring appropriate working conditions.
e. The Site supervisory shall be responsible for controlling ground water, surface- water runoff and run-on around the construction area.
f. Surface water shall be pumped or drained from the site area to ensure the area is devoid of standing water. Surface water shall be pumped or drained in a manner which prevents flow or seepage back into the construction area. Pumping and drainage practices shall be reviewed by the authorised personnel prior to initiation of activity.
g. Diversion ditches, either permanent or temporary, may seal the created diversion for surface water around or away from the construction area with a topical application of the polymer solution if necessary.

3.3.3 Granular Overlays for Mixing

Most existing pavements were initially designed for much lower traffic loads than those they are subsequently required. The continuous demand for pavements to cater for increasing traffic loads and tyre pressures can result in early pavement failures.

The simple solution of adding an overlay and sealing the existing pavement to prolong the life is not always satisfactory because of the poor subgrade strength and/or pavement material quality. There may also be restrictions preventing the raising of drainage levels. The alternative of the addition of an overlay and its incorporation by granular stabilisation into the existing pavement material has the following benefits:

i) Provides increased pavement thickness and additional cover over the subgrade.
ii) Improves the strength of the existing pavement.
iii) Improvement of the overall particle size distribution (i.e. improved grading).
iv) Improvement to ride quality.
v) Reduction in plasticity index (PI).
vi) Improved formation for drainage.
vii) Reduction of imported overlay thickness.
viii) Elimination of the need to rip the existing seal

The range of particle size distributions for the pavement material shown in below is a suggested limits for cementitious binders (REAM, 2020).

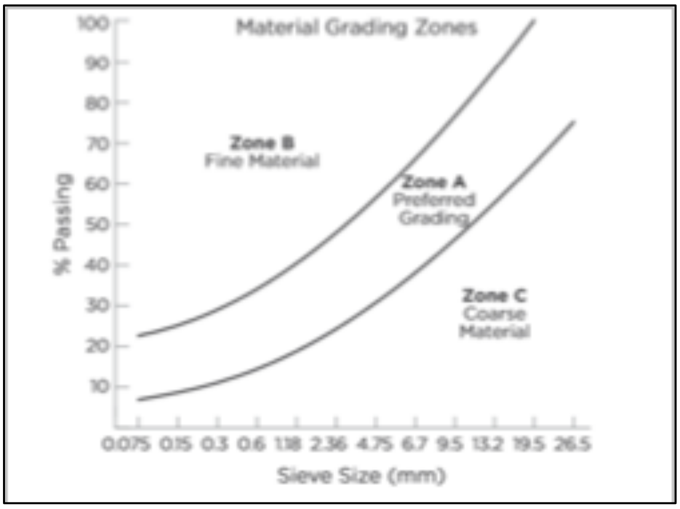

Figure 3-4 Suggested Particle Size Distribution Range for Base Materials to be Stabilised

The range of particle size distributions for the pavement material shown are suggested limits for granular materials with bituminous binders. For lime and cementitious binders, the grading limits are not as critical but will have an impact on the quantity of binder required to meet the desired strength or stiffness compared to a material that is well graded. The coarser end of the grading curve is preferred for pavement materials that are to be heavily trafficked. For granular stabilisation the local grading curve for unbound pavement materials are normally specified.

% Passing sieve (mm)	Zone A (ideal particle size distribution)	Zone B (fine material)	Zone C (coarse material)
26.5	73-100	100	<73
19.5	64-100	100	<64
9.5	44-75	>75	<44
4.75	29-55	>55	<29
2.36	23-45	>45	<23
1.18	18-38	>38	<18
0.6	14-31	>31	<14
0.3	10-27	>27	<10
0.15	8-24	>24	<8
0.075	5*-20	>20	<5*

Figure 3-5 Particle Size Distribution Envelopes for Bituminous Binders (REAM, 2020)

3.3.4 Selection Materials for Stabilisation

The work shall consist of all labour, supply of materials, tools, transportation etc. necessary to install the material for stabilisers in conformity with the Drawings in the specifications, inclusive of material and performance tests where specified. All requirements are accordance to the JKR/SPJ/2019-S18. All work shall be carried out generally in conformity with the principles of relevant codes of practice or design manuals current at the time of tender, including those referred to in this Specification. All materials and workmanship shall be in conformity with the related codes of practice current at the time of tender, including those listed in this Specification, except where the requirements of the codes conflict with the specification.

i) Design Mix

For design mix process, the engineer shall submit detail specification of the use of suitable binder agents and related documentation before any work commences. Any changes due to the site constrains shall be approved by the consultants. The proposed design mix and any changes due to site constrains shall submit request for approval prior to supply of any binder agents to site.

ii) Storage

Each consignment and category of binder agents must be stored individually to facilitate access, identification, inspection and sampling. Bagged binder agents kept in silos must be introduced through a 6 mm mesh screen that is either welded or fastened to the whole feed area of the silo charging hopper. Binder agents placed in silos must be sufficiently safeguarded against precipitation, humidity and dew and all charging and discharging ports of the silos must be appropriately sealed. Silo aeration equipment shall integrate dehumidifiers if required. The site representative shall be responsible at storage area for safeguarding materials and works from damage caused by weather, traffic, construction staging, fire, vandalism and other factors.

iii) Handling

The site supervisor shall ensure the delivery of all materials in the manufacturer's sealed original containers, clearly labelled with the manufacturer's name and product identification, in a manner that prevents damage, breakage or moisture infiltration.

The Site supervisory must store and manage all products in this part in complete compliance with the manufacturer's directives and safety regulations.

iv) Protection, Cleaning and Safety

The Site supervisory shall clean all adjacent areas of excess material, powder, cement and/or droppings. Chemicals used for cleaning shall be non-hazardous and non-flammable.

The Site supervisory shall process the binder agents using appropriate protective gear including gloves, masks, or goggles and appropriate clothing as described and in conformity with the manufacturer’s MSDS sheets.

The Site supervisory shall abide by the manufacturer’s recommendations for product safety and disposal of the material. The Site supervisory shall comply with all OSHA regulations for drilling procedures using protective gear including highlighted vests, face shields or goggles.

v) Quality Assurance

The Site supervisory shall provide the testing of the ground conditions before and after the commencement of work. The site requires shallow investigation of the proposed work location for inspection.

The testing procedures required for the shallow investigation especially for Dynamic penetration test. Then, physical properties test before and after the work. An engineering properties test before and after the work and finally performance monitoring.

The Site supervisory shall submit the tests requirements, schedule and details of each test in accordance with BS1377 or ASTM standards to ensure the work carried out is as conformity in the Drawings.

The material tests shall be executed to ensure the quality of the materials as specified by the supplier and the manufacturer. The Site supervisory shall cast either cube or cylindrical sample with a minimum dimension of 150 mm by 150 mm or 50 mm diameter by 100 mm long for strength purposes. The requirements are as below:

The compressive strength conducted shall comply to BS1377 or ASTM standards and JKRSPJ Section 18: Soil Stabilisation.

vi) Environmental Aspects

The Environmental Management System (EMS) aims to minimise the impact of production on the environment. The binder agents have a lifetime of several hundred years and can be reused in future applications when the pavements layer reaches the end of its design life.

3.3.5 Application and Mixing of Binder

There are four classes of binder used in stabilisation:

i) Quicklime is a granular product that is applied to the pavement by a spreader and requires a water cart to apply water which hydrates the quicklime (slaking) transforming it to hydrated lime. Because approximately 30% of hydrated lime is water the required application rate is 30% less for quicklime. Normally the required lime is calculated in the laboratory using hydrated lime, so a reduced percentage is required if quicklime is being used in the field.
ii) Powder Binders are also applied to the pavement by a spreader. There are a few powder binders available which can also be blended, these include cements, hydrated lime, slag, fly ash and powdered chemicals.
iii) Foamed Bitumen is produced by specialist stabilisers and introduced directly into the mixing chamber.

iv) Liquid Chemicals are normally mixed in the water cart and then sprayed onto the pavement. It is imperative that a fit for purpose spreader be used to apply the binders to ensure an accurate, consistent and uniform application.

Basically, a typical binder, such as cement, hydrated lime and quicklime are processed by the method of spreading the binder with a load-calibrated mechanised spreader from a backdrop chute. Spreading the binder from a reclaimer equipped with an integrated hopper, wherein the binder is transported within the reclaimer and delivered into the mixing chamber. Spotting of bags and hand raking are only suitable for minor patching (REAM, 2020).

Before using any binder type, staff should be familiar with the content of Safety Data Sheets. Two methods of verifying the binder application rate for mechanised spreaders to be used of 1 metre trays or mats (see below Figures attached). The utilisation of measurements obtained from on-board load cells on the spreader.

Figure 3-6 Mats/Trays Test on Road (Source: Stabilised Pavements (M) Sdn. Bhd)

Figure 3-7 Weigh Trays or Mats to Verify the Binder Application Rate

The preferred method of specifying the tolerance on spreading is to adopt a figure ranging from 5 to 10% of the specified rate. Another approach is to adopt a maximum specified application rate with a minus zero tolerance which will result in tenderers bidding on the specified rate only rather than adopting a suitable rate around the design value.

Consideration needs to be given to retaining spread binder on steep crossfalls and grades. This may be in the form of reducing the application rate to limit the potential flow, or the provision of physical barriers (such as wind rows) to reduce the flow of the binder on the surface prior to mixing.

Once spreading work completed, construction equipment may move the binder sideways resulting in a variable transverse application rate (see below Figure). It is therefore important during operations to reduce the movement of the construction plant until the binder has been mixed. The use of a direct injection system or direct feed system on the stabiliser will eliminate this effect.

Figure 3-8 After Spreading, The Front Wheels of The Stabiliser or The Rear Wheels of a Centre Drop Spreader Can Move the Binder Sideways (Source: Stabilised Pavements (M) Sdn. Bhd.)

Figure 3-9 The Hose from Tankers to the Stabiliser Will Need to Be Kept from Dragging Using Supports and Chains (Source: Stabilised Pavements (M) Sdn. Bhd.)

Figure 3-10 Desirable Features of a Mechanical Spreader Include Load Cells, Apron, Variable Width Spreading, Sealed Tanker and Filters for Binder Intake Management. (Source: Stabilised Pavements (M) Sdn. Bhd.)

The rotor system on the spreader must be always kept clean and tight as moist air will cause the hydration of cementitious material. This will reduce the effectiveness of the cementing action after mixing and spreading of the binder due to the formation of clumps.

In some wet materials it would not be prudent to spread some water for the slaking of quicklime and in this instance, a proportion of the specified quicklime may be slaked inside the mixing chamber of the stabiliser (Once the wet material dries the site supervisory should check if the quicklime has slaked). The final specified rate may be added on the same day or within 72 hours of the first application depending on the wetness of the material or the mellowing period.

Water should be introduced to the pavement by metering the water connected into the stabiliser's mixing chamber (see above Figure). Using a water cart to spray water onto the pavement should only be used when it is not possible to connect the water truck to the stabiliser for example in restricted areas.

All stabilisation processes utilising cementitious binders necessitate two applications of the stabiliser for completion. The exceptions to the two passes are as follows.

i) If the aggregate is pliable, a subsequent pass may negatively impact the aggregate grading.
ii) For ambiguous tasks, including unsealed road maintenance and pavement drying. During the initial pass, the mixing depth is typically 90 % of the ultimate depth to mitigate the effects of bulking on achieving the target depth.

The mixing operation must occur at a depth shallower than the final desired depth; otherwise, there will be insufficient binder in the crucial bottom part of the pavement layer. This will affect the pavement's performance. Single pass mixing may be appropriate for unsealed pavements.

The pulverisation of the pavement and the addition of the binder lead to the expansion of the pavement material. The extent of bulking must be evaluated to achieve the required pavement thickness and survey elevations as indicated. When asphalt patches constitute 50 % or more of the road surface area, the inclusion of Recycle Asphalt Profiling (RAP) material will result in a non-homogeneous foundation material for stabilisation. The asphalt and previously stabilised patches may be removed; however, it is more customary for the patches to be uniformly cross blended across the pavement.

The method for measuring the stabilised depth can be conducted following the second or final mixing pass, either through survey measurement or by excavating a small hole and utilising a string line to determine the depth of the stabilised layer as the difference between the target surface level and the base of the stabilised layer. This depth may also be verified post-final trimming or following the application of the initial seal coat, utilising the same reference string line.

The final pavement profile must be established before stabilising. Trimming to get the specified profile post-stabilisation will yield an inconsistent layer thickness, thereby affecting pavement longevity.

When the mixing chamber is positioned centrally between the front and rear axles of the stabiliser, the operator can enhance the mixing process to satisfy the depth specifications. Rear-mounted mixing chambers often elevate while traversing areas of more rigid pavement material (refer below Figure), resulting in inconsistent mixing depths. Rear-mounted stabilisers are generally less powerful, resulting in inefficient mixing and should be utilised solely for modest patching (less than 15 m^2) (REAM, 2020).

Figure 3-11 Rear-Mounted Mixing Chambers Often Elevate When Encountering Segments of More Rigid Paving Material. (Source: Stabilised Pavements (M) Sdn. Bhd.)

3.3.6 Joints

Vertical transverse joints in stabilised pavements manifest at the commencement and conclusion of a workday. The overlap at the start of work on the next day for transverse construction joints is established at 1.5 m to guarantee complete mixing and compaction at the joint, owing to the cylindrical configuration of the drum.

A longitudinal joint is created when material is compacted against a neighbouring section that has already been mixed and compacted and the designated working time for that part has elapsed. The permissible overlap for longitudinal joints is a minimum of 100 mm and they must be adequately far from the wheel tracks.

In instances when a more substantial overlap is indicated along a longitudinal joint, it is imperative to avoid excessive moisture during the mixing process. This will diminish the probability of excessive moisture in the blended material, potentially leading to a reduction in strength and the occurrence of shrinkage cracking at the junction. Equipment must be equipped with appropriate switches to deactivate water supply to the jets.

When the width for compaction of many contiguous runs surpasses 6 m, it is advisable to construct a designated longitudinal joint to prevent the occurrence of an unintended longitudinal fracture.

Below figure, illustrates standard daily activities for a lane-by-lane or full-road-width mixing procedure appropriate for country roads. The choice between the two is determined by the project's scale, traffic volume and the dimensions and arrangement of the equipment (REAM, 2020).

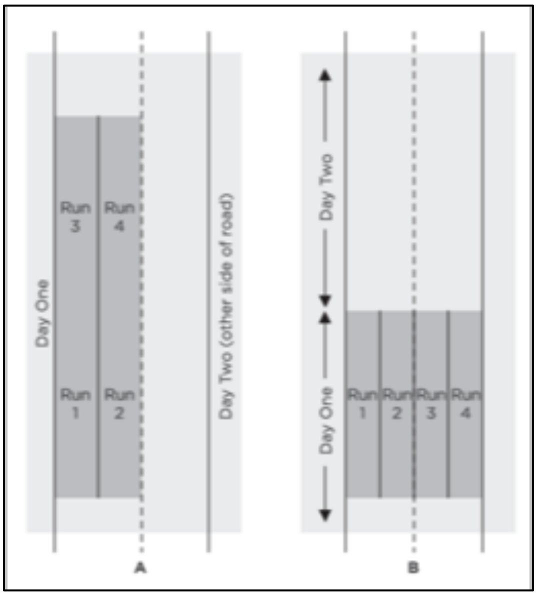

Figure 3-12 Typical Spreading and Mixing Arrangements for Half-Width (A) and Full-Width (B) Operations

Compaction typically commences as soon as practicable after mixing. As most stabilisers and rollers are nominally 2.4 m and 1.8 m wide respectively, the roller operator can stay well clear of the longitudinal joint adjacent to another run of the stabiliser. The technique reduces the potential for a longitudinal shrinkage crack to appear as shown in below figure.

Figure 3-13 Longitudinal Shrinkage Cracks May Appear If 'Full' Compaction Is Applied Near a Longitudinal Run of the Stabiliser Where Multiple Runs Are Used on Site.

There has been some success with cement slurries to improve the long-term bonding but this needs to be well controlled in the field. However, with the modern stabilisation and compaction plant it is current practise to stabilise layers up to 300 mm (REAM, 2020).

The complete extent of stability achieved by the technique is anticipated to remain unaffected by fluctuations in density profiles and corresponding modulus with depth. When same binder content is utilised for the stability of both layers, the design model typically employs the same modulus. This design methodology necessitates uniform material quality over the whole pavement depth and adequate subgrade support to ensure effective compaction (REAM, 2020).

3.3.7 Compaction

Compaction work should commence promptly following the mixing process. Typically, the area where one or more contiguous lines of the stabiliser exist, a width of roughly 300 mm close to the shared longitudinal joint, should remain uncompacted until the succeeding run is finalised. This technique will eradicate the propensity for a longitudinal crack to emerge at the joint. Occasionally, a longitudinal fissure may form; however, the application of polymer-based sealants or asphalt will inhibit the crack from extending to the surface (REAM, 2020).

Recommended the use of slow to medium setting time binders. These binders usually have a mixture of cement, slag, lime or fly ash components. These achieve two important advantages:

i) Increased working time so ensuring adequate time for optimum compaction.
ii) These binders have lower shrinkage and slower strength development so minimising the chance of cracking.

For deep-lift stabilised pavements, the prevalent types of rollers employed for compaction are a pad foot vibratory roller and/or a road compactor (34 t), along with a smooth drum roller. The pad foot roller (refer to Figure 2.16) aids in the compaction of the subgrade at the lower pavement layer, while the smooth drum is efficient in compacting the base course or top pavement layer. It is crucial to remove the pad foot roller from the compacted region as compaction approaches the surface to mitigate pad foot markings (refer to Figure 3.14) and the risk of inducing delamination at the pavement layer's surface. Padfoot rollers are unsuitable for the stabilisation of unsealed roads, as the pad foot imprints are visible through the pavement. (REAM, 2020).

Figure 3-14 The Pad Foot Vibratory Roller

A multi-tyred roller is occasionally employed to compact the surface and to seal the surface pores. The surface must not contain slurries, as this may lead to delamination and result in a soft crust (see to Figure 3.14), which will adversely affect the adhesion of the seals. (REAM, 2020).

Figure 3-15 The Surface Must Not Be Slurries, Since This May Result in Delamination and the Formation of a Soft Crust.

Compacting marginal and weak granular material at maximum dry density may lead to particle degradation. When employing heavy vibratory rollers over 18 tonnes, it is imperative to ensure that adjacent structures are not subject to structural or architectural damage. Non-vibratory rolling can effectively prevent harm to adjacent structures.

Selection between smooth drum and padfoot drum.

i) Smooth drum – for material such as rock, sand, gravel and silt. Also suitable for mixed soil with proper amount of moisture. Usually used to get smooth and flat surface after each time material filling.
ii) Padfoot drum – for use after soil stabilising, where higher compaction force needed to compress loose soil with additives. Also suitable to use for clay and recycle areas where flatness of the surface is not necessary.
iii) Selection between high and low vibration,
iv) High vibration – for the first 30-40% of total passes where the material is still loose and bouncing effect is low. The material will be compacted significantly with less time, but quality of compaction is not fully achieved.
v) Low vibration – for the remaining 60-70% from the total passes where inner material starts to fill up empty spaces. Oscillation vibration is the best to use here as more areas will be covered and material will be sinking to avoid any empty spaces and produces high quality compaction.
vi) Note - Using high vibration for every compaction job will create Bouncing Effect to the drum which can damage the drum's inner mechanism. Empty spaces between certain material will not be filled up too using high vibration.

To have minimum hilly surfaces and maximise compaction at flat surface as optimum compaction will be easier to achieve when machine's vibration is not influenced by gravity. Also, machine moving forward achieves better compaction than machine moving in reverse (REAM, 2020).

To use good operating machine with minimum wear and tear parts. This is to avoid less compaction achieved even after required passes are completed and failing the density test.

Table 3.1
Suitability of Compaction Plants (REAM, 2020)

Type of Plant	Suitability	Unsuitable
Smooth –Wheeled roller	Well graded sand and gravels; Silts and clays of low plasticity.	Uniform sands; silty sands; soft clays
Grid roller	Well graded sand and gravels; soft rocks; stony cohesive soils.	Uniform sands; silty sands; soft clays
Sheepsfoot roller	Sands and gravels with more than 20% fines; most fine. grained soils	Very coarse-grained soils; gravels without fines.
Pneumatic –tyred roller	Most coarse-grained and fine-grained soils	Very soft clay; soils of highly variable consistency
Vibrating roller	Sand and gravels with no fitness; wet cohesive soils	Silts and clays; soils with 5% or more fines; dry soils
Vibrating plates	Soils with up to 12-15% fines confined areas	Large-volume work
Power rammer	Trench backfills; work in small areas or where access is restricted	Large-volume work

As an alternative, using technology such as Accelerometer (Compaction Meter Value, CMV) and Resistance Energy meter (Machine Drive Power, MDP) to determine the compaction of material immediately while rolling. Also known as ‘Intelligent Compaction”, technology used to achieve optimum compaction passes while correlating the value to density measured using conventional way (FDT). This is to eliminate waste in time, fuel and energy which directly relates to cost saving. (REAM, 2020).

3.3.8 Trimming

Angle of Motor Grader blade ideally works between 10 degrees to 40 degrees, to get a smooth and free flowing motion of material rolling off the end of the blade. This reduces the horsepower required and avoids the Motor Grader from functioning like a Dozer. A blade angle that is too large will reduce the trimming area per pass and reduces overall productivity.

Lower gear of 1st or 2nd is preferred for final trim applications, providing lower speeds and better machine controls. Higher gears of 3rd or 4th is preferred for earthmoving applications.

The down pressure applied to the ground should be just enough to trim the material. Excessive down pressure of the blade can cause rapid cutting-edge wear, increase fuel consumption and reduce productivity. This can also cause sliding and tire spinning which increases tire wear.

As an alternative using technology of Trimble Automatic Lifting of Motor Grader to achieve better accuracy of surface grade. With 3D mapping of the area to be trimmed, the design and elevation of the area can be input into the system for the Motor Grader to automatically adjust the blade lift to achieve the specified level, reducing dependency on operator's judgement (REAM, 2020).

3.3.9 Curing Process

The curing procedure of a stabilised material may involve the use of softly and frequently sprayed water until the bituminous seal or subsequent layer is applied. While most of the pavements stabilised with a binder that has a cementitious component may be trafficked immediately after completion, it is very important that the pavements be cured. Best results are achieved if the pavement is kept damp (not wet). It is preferable to apply this seal within four days of stabilisation.

The cementitious process, which takes place progressively after stabilisation continues to provide measurable strength gains for a variable period, which may extend up to 12 months and 2 months for chemical product, depending on the stabilisation agent used. This process requires moisture that is taken into crystallisation during the hydration process (REAM, 2020).

If the moist environment is not provided in the pavement, a significant drying profile will develop in the upper layer, particularly in the first week after compaction. This drying will result in shrinkage and shrinkage cracking. It will also lead to a strength profile in the pavement that is variable and low and will result in poor surface performance.

However, treatment of the stabilised pavement by watering intermittently during the first week will normally provide adequate curing. The pavement should be sprayed to make the surface should not be allowed to dry out. Curing will be more difficult under highly evaporative weather conditions, that is when temperatures are high and/or windy condition prevail. If an additional pavement layer is to be constructed to complete the pavement structure, this layer should be constructed as practicable, for curing to be optimised.

3.3.10 Sealing (Optional)

Sealing works at the upper finished layer is carried out after the assessment of relative compaction and other conformance criteria have been met. Modern seals especially polymer modified, or geotextiles will usually prevent any cracking coming to the pavement surface (REAM, 2020). The additional requirement for sealing is for adhesion of the seal to the surface of the stabilised layer. Some factors effecting adhesion are dust on the surface, high surface porosity and voids, the ability of the bitumen to penetrate, the compacted surface, a low pavement surface temperature and a slurries surface. The removal of existing slurry prior to spray sealing may be part and considered as final trimming process.

3.3.11 Testing

To ensure conforming material it is imperative that the following procedures are well controlled:

i) Accurate spread rate and efficient mixing. (using fit for purpose plant).
ii) Moisture control with regular hand checks behind the stabiliser.
iii) Thorough compaction to be completed as soon as possible after mixing. (REAM, 2020).
iv) The test regime could include the Grading, Density, Unconfined Compressive Strength (UCS), CBR, Straight edge and Ride quality.

Acceptance criteria should be reasonable as it must be remembered the parent material is variable for Protection of Work, shall use all means necessary to protect all prior work, including all materials and completed work of other Sections. In the event of damage, person in charge shall immediately make all repairs and replacements, shall be responsible for traffic control and restriction of traffic in the areas of construction unless otherwise specified. (Refer to JKRSPJ Section 19: Traffic Management at Work Zone). In the event of rain, shall protect all disturbed areas of work such that the subgrade shall be kept dry to achieve proper curing.

3.3.12 Material Safety Data Sheet

Material Safety Data Sheet (MSDS) or Safety Data Sheet is document pertaining to the use of chemicals and its content for the purpose of industrial usage. The document provides the manufacturer/distributor/supplier chemical content, characteristics and environmental details of the proposed chemical material to be used as soil stabilisers. The typical information included in the MSDS are below:

i) Appearance/Identification

The appearance and identification of the stabilisers shall be described either powder or liquid form, depending on the nature of its usage. The colour and viscosity of the material shall be described where appropriate.

ii) Application

This section describes the general usage of the stabilisers used and whether it is additional to be applied as enhancer, activator etc.

iii) Chemical characteristics/composition

The chemical composition is described based on the compound and amount of chemicals present in the stabilisers. However, the given ratios and amount distribution of the chemical compounds shall be the confidentiality of the manufacturer/distributor/supplier.

iv) Storage and shelf life

This section provides the time it can be stored in the factory and/or on site based on the environment surrounding the storage area. It is necessity to evaluate its storage life since there is an expiry of the chemicals which might affect the performance of the stabilisers during usage.

v) Preparation prior to processing

In the event where the stabilisers are to be mixed on site, general preparation shall be described where appropriate for variation of chemicals. Any personnel protective equipment (PPE) for minimum exposure to hazardous chemicals shall be included where appropriate.

vi) Possible hazards/hazards identification

Any possible hazards and harmful effects to the personnel shall be informed in this section and extreme precautions shall follow the guidelines as described in the MSDS.

vii) Waste disposal requirements

In the event where material wastage and excess concur, the personnel shall abide by the requirements as stated in the environmental act described in the MSDS. Failure to do so will cause environmental hazards to the applicator and construction use of the stabilisers if improper disposal is not met.

viii) Specifications

This section describes the physical characteristics of the stabilisers such as density, viscosity (if in liquid form) and vapour density (if in solid form). In the event where the dust is airborne, concentration of chemical that can perceive by smell, such as odour threshold shall be evident.

ix) Typical processing data

The time of mixing, cream time, gelling time and curing time shall be included where necessary for stabilisers reaction based on trial mix and admixture to substance where appropriate. Typical suitable material for subgrade stabilisation shall react with the proposed chemical during mixing by the personnel accordingly based on the specified mixing time provided by the manufacturer/distributor/supplier.

x) Typical physical properties

Typical properties of the stabilisers shall be described where appropriate such as core density, compressive strength, thermal conductivity, water absorption and stiffness modulus where appropriate and their given standard requirements used. Additional details of the properties shall be included herein.

xi) Safety advice

Additional safety requirements as specified by the manufacturer/distributor/supplier shall be abide by when dealing with toxic chemicals and airborne chemicals. National and international regulations in accordance with Safety, Health and Hazard Act.

xii) Fire resistance

Additional information on fire resistance shall be enclosed to avoid direct contact with the changing temperature and weather conditions. Storage area shall be away from any act of open burning and miscellaneous to avoid direct flame of the stabilisers.

xiii) Environmental considerations

The description must incorporate environmental issues pertaining to the stabilisers used. If it is deemed hazardous, proper PPE and direct contact precautions need to be included in this section as stipulated by the national and international Environmental Act.

3.4 Case Study for Soil Stabilisation

Case study selected consist of two (2) samples of soil for laterite and marine clay. It was conducted at UiTM laboratory and project laboratory.

3.4.1 Mix Design; Laboratory Testing at UiTM Shah Alam

3.4.1.1 Soil Sample for Laterite using Ordinary Portland Cement (OPC)

This research is based on laboratory testing. Moisture Content, Particle Size Distribution, Atterberg limits, Compaction and pH Test are among the laboratory studies performed on natural soil. While the modified soil test is the same as the natural soil test, UCS is added for strength. The soil's physical properties were determined according to BS 1377: Part 1-9, 1990 and the stabilisation tests were carried out according to BS 1924: Part 1-2, 1990. The test will be carried out with varying concentrations of cement. Then, the tests were carried out in this study:

i) Moisture content
ii) Particle size distribution
iii) Atterberg Limit (Plastic Limit and Liquid Limit)
iv) Compaction (Maximum Dry Density and Optimum Moisture Content)
v) pH
vi) Unconfined Compressive Strength

3.4.1.2 Sample Preparation and Design Mix Configuration

Most of the method above used the preparation of the natural soils are sieving and oven dry. However, there are different size of sieving and weigh to use. In addition, for moisture content use the original soil without sieving with crumble the soil only. The method of sample preparation in an accordance to BS 1377: Part 1-2, 1990. Meanwhile, the method preparation for the mixing soil samples according to BS 1924: Part 1- 2, 1990. Soil samples are mixed with Ordinary Portland Cement and a specified percentage of water is added to act as a medium for the reaction process. The design mix configuration Ordinary Portland Cement are 4%, 5% and 6%. It was done using the "dry mix procedure" according to Ndububa in 1995. Any of the stabilisers had to be well mixed with laterite when it was dry and then the water had to be added gradually while mixing was still going on to reach the desired consistency. The stabilisers' mix amounts were 4 percent, 6 percent and 8 percent used in this study. To prevent segregation, the mixing was done thoroughly and mechanically.

3.4.1.3 Preliminary Result and Discussion

Summary of the properties for laterite soil is given in table below. According to that, the moisture content for laterite soil ranges from 18.25 percent to 18.86 percent, with an average value of 18.50 percent. These outcomes are consistent with research ranges, said Bell and Gidigasu. It is essential to note that the natural moisture content values are within those ranges indicated by the past studies on laterite soils, even if it is not necessary to comply with these requirements. The average moisture content around Ayodele in 2020 in Osun State, southwest Nigeria, which was 18%, can be compared to the laterite of this study. A specific area's natural moisture content can be determined. This knowledge can be used to determine the optimum way to apply it to engineers. Refer Appendix 2.

Table 3.2
Summary of Physical and Mechanical Properties of Laterite Soil

Physical and Mechanical Properties	**Value**
Moisture Content (%)	18.5
Gravel (%)	0.6
Sand (%)	1.4
Silt (%)	94.91
Clay (%)	2.91
Liquid Limit (%)	40.85
Plastic Limit (%)	28.84
Plasticity Index (%)	12.01
Classification	CI
Maximum Dry Density, MDD (Mg/m3)	1.62
Optimum Moisture Content, OMC (%)	25.92
pH	4.35
Unconfined Compressive Strength After Curing 7 Days (kPa)	195

According to the research conducted by the earlier authors referenced, the Atterberg limit test for laterite and lateritic soils yields a noticeably wide range of values. (Gidigasu, 1976) states that this is affected by the various climatic and soil conditions in the vicinity of the location where the soil samples are gathered. The LL and PL values were 40.85% and 25.84%. It can be said that the kaolinite clay material is contained in the soil samples. This is because kaolinite soils have liquid limit values of between 35 and 100 percent and plastic limit values of between 20 and 40 percent (Ayodele et al., 2020). The liquid limit shown 20 mm of the penetration to the linear line. The PI was the difference between the LL and PL values which was 12.01%. The range of water concentration where the soil changes from a plastic condition to a liquid state is known as the plasticity index. The Atterberg limits or consistency limits acquired from the tests assist in describing the consistency of the fine-grained laterite soils. The name relationship is due to this. From the plasticity chart, based on the LL value and the PI, classify the laterite soil as clay of intermediate plasticity. Furthermore, the laterite soil is acceptable for stabilisation because the PI is greater than 10% and the percentage of fine material is greater than 25% (Malaysia Public Work Department, 2017). From studied, cement was suitable for the stabilisation of laterite soil.

According to Ayodele in 2020, the pH value was 4.35. The soil was acidic since the pH value was lower than 7, which indicated. The low pH of the soil samples was also a sign that the soils included kaolinite because kaolinite often has a low pH.

The soils become denser by compaction, increasing their dry unit weight when more water is added. This improves shear strength, reduces permeability and lessens the effect of the settlement. The MDD beyond which soil starts to lose density is the biggest restriction when water assists in maximizing the dry density. The moisture level that corresponds to that is the OMC for that specific soil. From the compaction test, the MDD and OMC were 1.62 Mg/m3 and 25.92% based on below table, illustrates how more water was added to the soil, effectively replacing the soil particles with water and lowering the soil's density.

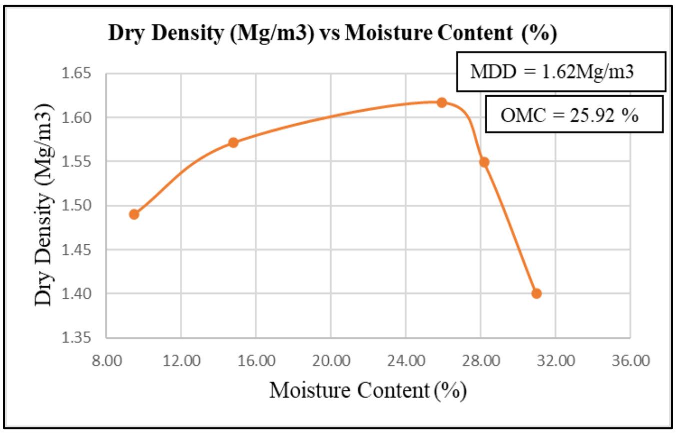

Figure 3-16 The Graph of the Dry Density (Mg/m3) VS Moisture Content (%)

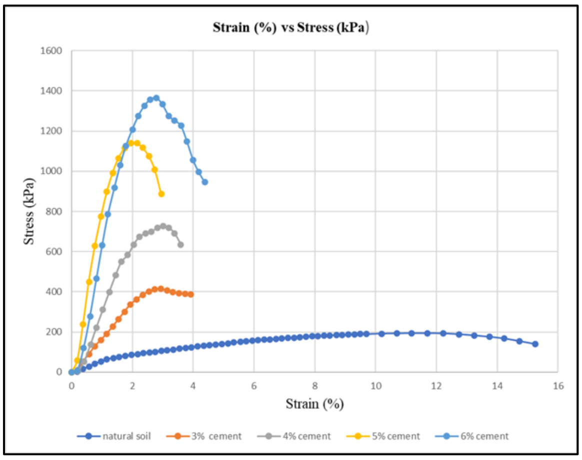

Figure 3-17 Strain (%) vs Stress (kPa)

Figure above, showed the Stress-Strain Curve in kPa of a soil sample for 0%, 3%, 4%, 5% and 6% of cement content. From the results shown in the above table, the maximum stress was 195 kPa, 480 kPa, 728 kPa, 1140 kPa and 1365 kPa for 0%, 3%, 4%, 5% and 6% of cement content with axial strain of 11.7%, 3.35%, 3.01%, 1.95% and 2.79%. The dry density for 0%, 3%, 4%, 5% and 6% of cement content were 1.94 Mg/m3, 1.79 Mg/m3, 1.96 Mg/m3, 1.92 Mg/m3 and 1.96 Mg/m3. The shear strengths in kPa for 0%, 3%, 4%, 5% and 6% of cement content were 97, 240, 364, 570 and 683.

Table 3.3
Results of UCS After Curing 7 Days

Cement Content (%)	UCS after Curing 7 days (kPa)		Dry Density (Mg/m3)	Axial Strain (%)
	Sample 1	Sample 2		
0	161	195	1.94	11.7
3	386	416	1.93	2.93
4	659	728	1.96	3.01
5	1010	1140	1.92	1.95
6	1145	1365	1.96	2.79

A cohesive soil sample, which includes clay, must undergo the unconfined compression test (UCT) to ascertain its unconfined compression strength (qu) and failure shear strength parameter. Based on above table, it can be shown that as the stress reaches its maximum level, it also decreases with axial strain. According to that, natural laterite soil had an UCS of 195 kPa after 7 days of curing, making it unsuitable for use as a subgrade. Then, the laterite soil needs to stabilise. From previous studies, all laterite soil was not suitable as subgrade and needed to stabilise at a range of 128.88 kPa to 580 kPa. As the cement amount and curing period increased, the strength parameters (CBR and UCS) also causing the UCS to increase from 195 kPa to 1365 kPa when stabilised (Dabou et al., 2021).

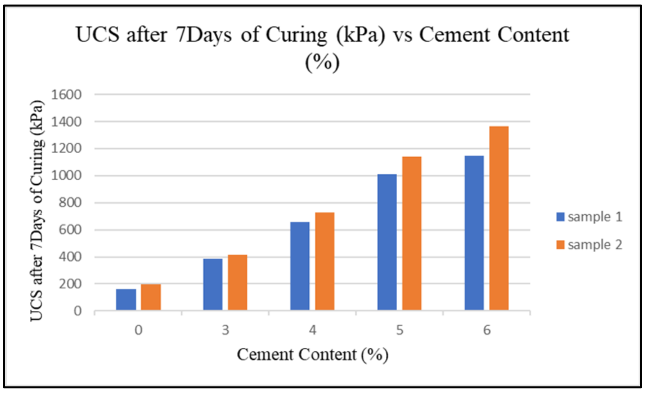

Figure 3-18 Graph of UCS After Curing 7 Days, kPa vs Cement Content (%)

3.4.2 Mix Design; Pulau Burung, Seberang Perai Landfill, Pulau Pinang

Penang Council district is largely built on marine clay which presents long term issues for the construction and maintenance of any load induced structure due to the highly reactive and expansive properties inherent with this material. In 2017, Penang Council issued instructions for the construction of multiple new land fill cells to facilitate ongoing waste collection. Below figure, shows the landfill site to be stabilised.

Figure 3-19 Landfill Site (Cell 1) to be Stabilised

For the first time, Penang Council decided to stabilise the existing marine clay shown above, to reduce the effects of the expansive influences and to reduce the upfront capital cost compared to previous design solutions which incorporated significant removal of marine clay and replacement with better quality material, covered by geotextile fabric.

Figure 3-20 Marine Clay Soil (Source: Stabilised Pavements (M) Sdn. Bhd.)

3.4.2.1 Mix Design

Samples were taken from the site to analyse the material properties and then determine an appropriate mix design to improve the CBR. Host material properties are shown below (Refer Appendix 1):

a.	Composition:	Silty Clay 86% and Sand 14%
b.	Atterberg Limits:	Plastic Limit 43%
		Liquid Limit 71%
		Plasticity Index 28%
c.	4 Day Soaked CBR:	1%

The selection of lime as a binding agent is based on research conducted by Mr Siaw Yah Chong and Khairul Anuar Kassim from the Department of Geotechnics and Transportation at University Technology Malaysia, focussing on the effects of lime on the compaction, strength and consolidation characteristics of Pontian Marine Clay. In conclusion, lime effectively enhanced the compaction, strength and consolidation properties of Pontian marine clay. Nonetheless, curing length is a critical factor in assessing the efficacy of lime stabilisation (Siaw Yah Chong, 2015).

The research indicated that marine clay is a troublesome construction material frequently seen in Malaysia's coastal regions. Prior researchers demonstrated that lime stabilisation significantly improved the engineering qualities of clay. In conclusion, lime effectively enhanced the compaction, strength and consolidation properties of Pontian marine clay.

Sample	Age *t* (day)	Coefficient of Permeability *k* (mm/s)
Pontian Marine Clay + 10% Lime	7	3.153×10^{-06}
	14	3.195×10^{-06}
	28	2.446×10^{-06}
	56	1.485×10^{-06}

Figure 3-21 The Effect of Soil Stabilisation on Permeability and Strength

(Source: Siaw Yah Chong, 2015)

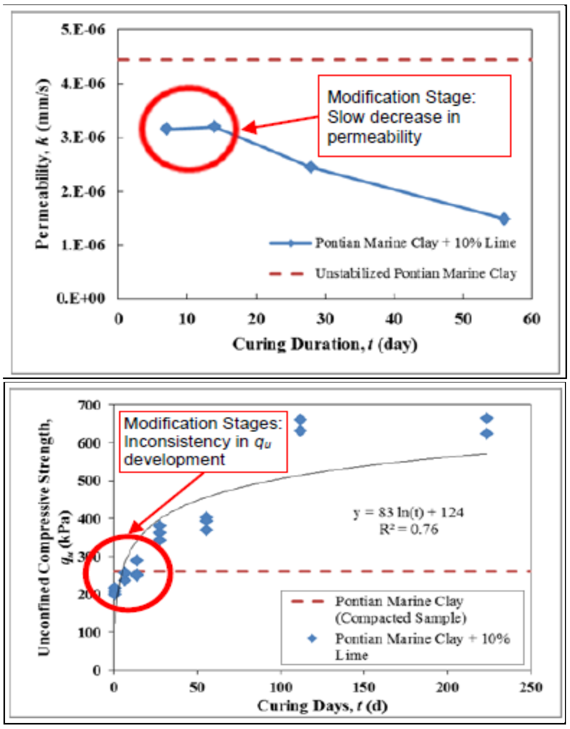

Figure 3-22 The Effect of Soil Stabilisation on Permeability and Strength

(Source: Siaw Yah Chong, 2015)

Lime was then ad ded to calculate the minimum quantity of lime required to ensure the chemical reaction remains permanent upon the pH reaching at least 12.4 and then plateauing (Young, 2014). This was achieved with the lime demand (or lime saturation) test which produced a minimum lime content of 4%. Soaked CBR testing was then undertaken at the 4% lime application rate to determine what the increase in CBR would be. An average value of 37% was obtained. This level of increase is not uncommon with the incorporation of lime between 3% and 6% achieving increases in strength from 2.5 to 11 times of the untreated soil (Khairul et.al., 2004).

The subsequent construction process utilised specialised equipment with a purpose-built stabilising machine, capable of mixing the lime into the marine clay at 300mm thickness in single lift. Below figures, shows the process of clay stabilisation using lime.

Figure 3-23 In-situ Stabilisation of Marine Clay with 4% Lime

(Source: Stabilised Pavements (M) Sdn Bhd.)

With the forward thinking of Penang Council and their decision to utilise in-situ stabilisation as a means of treating the marine clay satisfied the consultants brief as well as significantly minimising further waste materials being sent to landfill and eliminating the need to import other materials from offsite.

3.4.2.2 Soil Sample for Marine Clay using Lime

In this research, the soil that were used in this study is marine clay. The soil was taken at landfill area in Nibong Tebal, Pulau Pinang. Clay soil was chosen as the testing soil for this study since it is regarded as problematic soil because it is particularly vulnerable to water logging because of its highly mineral content such as montmorillite, which causes structure settlement. For laboratory testing, the soil sample should be more than 30 kg to conduct all the proposed testing.

Figure 3-24 The Site Location Marine Clay Soil at Pulau Burung Landfill Project

(Source: Stabilised Pavements (M) Sdn. Bhd.)

The quicklime used in this study is obtained from RCI Lime Sdn. Bhd. The certificate and analysis for this quicklime powder is shown below.

Table 3.4
Composition of Quicklime Powder Used

Parameters	Results	Test Method
Calcium Oxide, CaO (%)	90.62	MS 850 : 2010
Magnesium Oxide, MgO (%)	2.41	MS 850 : 2010
Insoluble Matter Including Silicon Dioxide, SiO_2 (%)	0.36	MS 850 : 2010
Ferric Oxide, Fe_2O_3 (%)	0.02	MS 850 : 2010
Aluminium Oxide, Al_2O_3 (%)	0.25	MS 850 : 2010
Passing 1 mm sieve (%)	100.00	ASTM C110

3.4.2.3 Mix Design Preparation

The mix design for this research is shown in below table. Usually in road based, the normal range for lime stabilisation takes about 3% to 6% lime addition by mass of the dry soil (Zukri & Ghani, 2014). The specimens get stronger as the pH dropped. Three percent lime stabilised the soil and quadrupled the strength, but more lime resulted in a longer response with more strength (Eades et al., n.d.). The proposed mix design on the percentage of lime for this research is based on past researchers whom conducting study on the soil stabilisation using quicklime/lime on clayey soil.

Table 3.5
Mix Design Used

Batch	Sample	Type of Laboratory Tests
Control sample	Natural Soil (100%)	• Moisture Content • Specific Gravity • Particle Size Distribution • pH Test • Atterberg Limit Test • Standard Proctor Compaction test • Unconfined Compressive Strength test
1	Soil + 3% lime	• Standard Proctor Compaction test • Unconfined Compressive Strength test
2	Soil + 6% lime	
3	Soil + 9% lime	
4	Soil + 12% lime	

3.4.2.4 Physical properties of marine clay

By measuring the moisture content, specific gravity, particle size distribution and Atterberg limit test, the physical characteristics of natural soil are identified. According to the British Soil Classification System (BSCS), the soil sample is categorised as clayey silt of Intermediate Plasticity (CI) based on the physical characteristics tests that were performed (BSCS). Information about the soil index characteristics is listed above.

a) Atterberg Limit Test

The ability of a soil to deform without cracking or collapsing is known as plasticity. The lubricating effect of water films between adjacent particles is one of the primary properties of all cohesion soils. Soil engineering relies heavily on information about a soil's plasticity and standard limits tests that have been established to measure it. The shear strength and compressibility of fine soils are greatly influenced by their plasticity and as a result, this property is used to classify soils. The Atterberg Limit is the point at which soil behaves like a plastic when the water content is high. It is used to determine the moisture content at which fine-grained clay and silt soils appear in four states: solid, semisolid, plastic and liquid. The Atterberg limits are a basic measure of a fine-grained soil's critical water content. It has three limits: shrinkage, plastic and liquid. Refer Appendix 1.

Table 3.6
Atterberg Limit of Marine Clay Soil Sample

Liquid Limit (LL)	35.85 %
Plastic Limit (PL)	15.76 %
Plasticity Index (PI)	20.07%

Soil classification	**CI**

b) Standard Proctor Compaction Test

The process of altering the density of soil using mechanical means is known as compaction. It causes the particles in the soil to rearrange themselves and it leads to a decrease in the amount of space between them. When soil is compacted to a very high degree, there are relatively few voids present in the soil, which results in the soil having a larger unit weight. Because the soil in its natural state is only loosely connected, it needs to be compacted to increase its bearing capacity (Hussain & Dash, 2016). The compaction test shows the relationship between the dry density and the amount of water in a soil sample. A series of compaction tests are done on both natural soil and soil that has been stabilised by adding quicklime in varying quantities, such as 3%, 6%, 9% and 12% by weight of dry soil.

c) Unconfined Compressive Strength (UCS) test

A test of unconfined compression is carried out to establish the maximum compressive strength of marine clay after lime has been added. The unconfined compressive strength of soil is defined as the maximum value of compressive force per unit area that a soil sample can withstand. The Unconfined Compressive Strength of soil sample is gradually increase after 7 days curing period. After curing days, the increase in strength can be seen as the soil sample become harder. Furthermore, by increasing the percentages of quicklime, the Unconfined Compressive Strength of marine clay is also reinforced. The curves obtained from plotting the stress and strain value of 0% ,3%,6%,9% and 12% of quicklime as the soil stabiliser. As can be seen each curve reach peak value of strength but start to drop at certain point due to brittle structure of soil sample. The curing 7 days curing period, reducing the water content of sample and the sample will undergo brittle shear slip failure due to dry conditions of sample.

3.5 Conclusion

In conclusion, a total of four soil samples were tested through laboratory investigations conducted at UiTM and the project laboratory, where FDT and CBR tests with design mix process at UiTM and comprehensive tests including Atterberg Limits, Lime Demand Test, FDT, CBR and UCS at the project laboratory were carried out to establish design mixes using cement and lime simultaneously, with this thesis emphasizing the calculation procedures to determine the optimum binder agent percentage in accordance with ATJ,JKR requirements in Chapter 4.

CHAPTER 4
DEVELOPMENT OF CONCEPTUAL DESIGN MIX FOR SUB GRADE STABILISATION

4.1 Design Mix for Laterite Soil Using Cement

4.1.1 Introduction

Malaysia covers the road network of more than 250 thousand kilometres, with approximately 250 thousand kilometres of paved and unpaved roads. Meanwhile, expressways cover approximately 2 thousand kilometres. The most used type of pavement in Malaysia is flexible pavement. This is because it is less expensive than others. The pavement structures include subgrade, subbase, base course and asphalt finish layer. The existing material beneath a constructed pavement is known as the subgrade. The subgrade layer is vital because any pavement failure can occur if the subgrade is constructed with poor performance or quality of the soil. Thus, today's engineering methods for improving subgrade layers are to consider and include modification stabilisation methods. The addition of a binder agent to the soil to change its material characteristics is known as soil improvement. Subgrade or soil stabilisation is the processing method by treating the existing soils to improve their performance or strength as a construction material. Therefore, they become entirely fit for construction, even if they are no longer classified as such.

In 1993, according to Bell, mixing chemicals with soil to increase its volume stability, permeability, durability and strength, thereby improving the pavement structure's quality is known as soil stabilisation (Raheem et al., 2010). Then, in 1996, any procedure that improves and makes a soil material more stable is defined as stabilisation by Thagesen (Amu et al., 2011). As defined by Garber and Hoel, subgrade or soil stabilisation is an improvement of existing natural subgrade or soil to enhance its material characteristics via engineering processes throughout the twentieth century. Soil stabilisation is the method or process of introducing or improving desired features into subgrades or soil material to make it valuable and stable for construction. Cement or lime as a binder agent for the primary materials for stabilised soils. In this research, the soil that is concerned is laterite for the existing subgrade layer and cement is the material agent for the stabilising method. Laterite is a reddish-brown tropical soil that is highly in iron oxide and is formed by a range of rocks weathering processes. It grows in humid climates such as tropical and sub-tropical countries (Raheem et al., 2010). Adding cement into the existing sub grade or soil as a binding agent is known as cementitious stabilisation. To identify the amount of cement content to be added or mixed with subgrade or soil type during the mix design process.

The application of soil stabilisation in mix development and road construction is restricted to a specific purpose, possibly due to budgetary, improper design, site supervisory competency issues and uncertainty regarding the technique's efficacy for large-scale projects. Additionally, for subgrade or soil stabilisation procedures, there is a limit to the acceptable equipment. According to researchers, in Malaysia, soil stabilisation is still in its early stages and is less frequent than using more typical construction techniques like crusher run and other common materials for building roads (Razali & Che Malek, 2019). The Public Work Department of Malaysia has accepted and recognised the stabilisation technique in the construction of roads through a new specification for subgrades stabilisation for the rural areas through its public works departments in collaboration with the Ministry of Rural and Regional Development due to the advantages of having a stabilisation technique that can be increased strength for the existing subgrade at a reasonable time and cost. Therefore, this research studies and identifies the strength of laterite subgrade or soil stabilised with cement.

4.1.2 Laterite History

In 1807, a geologist named Buchanan first described the term "laterite" from the word "later" in Latin. He calls it laterite because it is a soft ferruginous mineral quarried in southeast India for building blocks (Rashid et al., 2002). Laterite is blackish brown to reddish and is rich in iron and aluminium. Lateritic soils are distinguished by their distinct colour, low fertility, high clay content and decreased cation exchange capacity. The colour of the subgrade or soil is to be determined by the number of iron oxides, hematite and goethite. If the soil sample contains a significant concentration of iron oxides, the laterite sample will be reddish to brown (Kasim et al., 2017). However, there is a solid propensity to refer to all red tropical soils as laterite, which has led to a great deal of misunderstanding. Laterite is an appropriate title for various combinations of clays, sands and gravels. According to Bridges (1970), the term laterite refers to "a huge vesicular or concretionary ironstone deposit nearly usually associated with uplifted peneplains originally associated with locations of low relief and high groundwater" (Saing et al., 2017).

In 1997, ferric known as iron-rich cemented crusts, calcrete or bauxite known as for aluminium-rich cemented crusts. Meanwhile, calcrete known as for calcium carbonate-rich crusts and silcrete known as silica-rich cemented crusts which are called laterite by Fookes based on their hardening. The contents of silica to sesquioxide show the percentage of lateralisation. The SiO2/Fe2O3/Al2O3 ratios according to Bell (2007) and Pearring (1968), ratios are less than 1.33 are laterites soils. Meanwhile, the ratio between 1.33 and 2 indicates laterite soils. However, if the ratio is more than 2 is non-lateritic soil (Sujeeth, 2015), (Bello Yamusa et al., 2017).

In conclusion, the researchers' findings demonstrate that variances in attributes depend on a location's topography, soil conditions, climate and other factors. According to Raychaudhuri, although there are some disagreements on how the topic is understood, laterite soils and their studies have significantly advanced the undisputed understanding of their origins, contents, creation, use and impact. This requires a thorough investigation into that area of interest into soil types that exhibit laterite properties.

It is essential to analyse lateritic soils and identify material characteristics. The consideration process shall start with design and then construction materials for the limitations and effects. Numerous tests and research have been carried out for this purpose and have been released throughout the period. MC, Lime Demand Test, Sieve Analysis, Proctor Compaction and AL are some of the basic physical characteristic tests performed in this study. Gidigasu stated in 1976 that the material characteristics of laterite soils in pavement structures are primarily determined by their particle-size characteristics, composition and strength of the gravel particles to increase the compaction results and environmental situations. According to Scott, the Shrinkage Limit (SL), LL and Plastic Limit (PL) tests of Atterberg consistency are the standards from which engineering characteristics of fine-grained soils can be obtained. The tests evaluate the proportion of essential water content. According to Bell, the table below displays the typical characteristics of laterite soil (Sujeeth, 2015).

The engineering properties of subgrade or soil are involved in the strength of soil stabilisation. The standard method for determining stabilised materials' strength is unconfined compressive strength (UCS). According to Paige (1998), Vorobeiff (2004), Austroads (2008) and Yeo (2011), UCS is used to assess the mix's capacity to perform effectively as a bonded sub-base and base layer (Wahab et al., 2021). The maximum axial compressive stress that a cohesive soil specimen can withstand under zero confining force is called UCS. The UCS test is the quickest and least expensive way of determining soil shear strength.

Properties	(Dabou et al., 2021)	(Alhassan & Mustapha, 2007)	(Wahab et al., 2021)	(Biswal et al., 2018)	(Ayodele et al., 2020)	(Saing et al., 2017)
Natural Moisture Content (%)	12.17	22.27	-	-	18	22.25
% Passing Through BS Sieve 75µ	30.04	77	-	-	39.1	94.89
Gravel (%)	-	-	12.79	33	-	-
Sand (%)	12.48	-	17.54	67	-	-
Silt (%)	4.93	-	61.26	0	-	-
Clay (%)	26.93	-	8.41	0	-	-
Liquid Limit (%)	41.74	49.5	70.3	47	46.2	68.73
Plastic Limit (%)	22.73	24.4	42	28	30.1	41.96
Plasticity Index (%)	19.02	20	28.3	19	16.1	26.77
ASHTO Classification	A-2-7	A-7-6	A-7-5	-	A-7-5	A-7-6
Maximum Dry Density (Mg/m^3)	-	1.482	1.39	-	1.85	1.77
Optimum Moisture Content (%)	-	18.38	28	-	16.5	20.7
pH	-	-	-	-	5.8	-
Unconfined Compressive Strength (KPa)	580	290	200.75	410	195	128.88
Location	Jomo Kenyatta University of Agriculture and Technology (JKUAT), Kenya.	Minna, Nigeria	University Technology Malaysia, Johor, Malaysia	Eastern India	Osun state, South West, Nigeria.	Buli, India

Figure 4-1 Engineering Properties of Laterite Sub Grade or Soil (Source: Wahab et al., 2021)

The shrinkage limit (SL), liquid limit (LL) and plastic limit (PL) tests of Atterberg consistency are the standards from which engineering characteristics of silt and clay soils are to be obtained. The tests evaluate the proportion of essential water content. Furthermore, LL must be less than or equal to 35 per cent and PI must be less than or equal to 12 per cent for soil to be used as subbase or base course material for road construction (Ayodele et al., 2020). LL is more than 40% and PI is greater than 16%, according to above table as attached. This provides another justification for the stabilisation of laterite soils.

Moreover, the laterite soil's strength did not match the requirements for subgrade because it did not require more than 800 kPa. Thus, the researchers can conclude that the natural soil of laterite did not exceed the minimum requirements. So, the existing subgrade or soil needs to be stabilised. UCS shall accept much attention and selection techniques for assessing to increase or improve the strength of stabilised soil. It is the primary test that is advised for measuring the requirement of identifying an additive that is required to stabilise the soil. The optimum additive might be determined from the UCS results depending on strength.

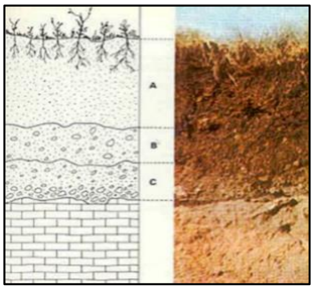

Figure 4-2 The Soil Layers

In Malaysia, the laterite soil is often located in residual soils. Soil layers, from soil down to bedrock, which is A to represent soil, while B to represents laterite, which is a regolith. Then, C represents saprolite, which is a less-weathered regolith, while below C is bedrock (See above figure). Laterite is generated by the weathering of rocks in humid tropical and subtropical climates. The geography in peninsular Malaysia has hilly terrain, consistent temperature, high humidity and rainfall. They form due to intense and extended weathering of the underlying parent rock, typically during periods of high temperatures and heavy rainfall with alternate wet and dry periods (Amadi et al., 2015). Chemical weathering removes the original soluble minerals. The fundamental result for highly leached soil type is an accumulation of hydrated iron and aluminium oxides due to water moving through the soil profile. As a result, according to Ugbe (2011), those laterite soils are insoluble and highly stable. Under the high-temperature circumstances of a humid subtropical monsoon environment, the leaching mechanism involves acid dissolving the host mineral lattice, followed by hydrolysis and precipitation of insoluble oxides and sulphates of iron, aluminium and silica.

Soil modification explains the process of altering chemical and physical characteristics in soils to achieve more strength and durability for construction purposes. When it comes reaction to increase physical soil such as shear strength and bearing capacity, soil stabilisation is a process of applying measured compaction with the addition of binder agent to generate a better soil characteristic that has required in material's engineering properties. To achieve desired soil properties, a variety of existing technologies and materials are available to choose. An operational of soil stabilisation and selection material binder agent plus existing sub grades/soil, named as stabilisation method. The soil stabilisation method is more commonly used for marine/soft clay treatment to change the material characteristics, improve strength and save time and cost. Cementitious, chemical additives and non-cementitious are all examples of binder agents, according to Reddy & Tahasildar (2015)

Table 4.1
Various Additives for Stabilisation of Soil (Source: Reddy & Tahasildar, 2015)

Group	Binder Agent
Cementitious	Lime, fly ash, ground-granulated blast furnaced (GGBS), cement, kiln dust and silica fume.
Chemical	$CaCl_2$, KCl, Na_2SiO_3, $FeCl_3$, $Mg(OH)_2$, $Na(OH)$, $NaCl$, $MgCl_2$ and $AlCl_3$
Non-Cementitious	Stone dust, quarry dust, aggregate waste, rock waste powder, crusher dust, granite sawdust and sand.

a) Chemical Stabilisation

Chemical stabilisation modifies the chemical characteristics of the soil by using admixtures. Blending natural soils with chemical agents such as lime, cement and asphalt is known as chemical stabilisation (Amu et al., 2011)—a method of enhancing ground condition for protection and dependable construction. Chemical stabilisation is one of the engineering ways of ground improvement based on treatment procedures. Other stabilisers, such as eggshell powder and sodium silicate, sometimes known as TX-85, have been utilised in several experiments to change or improve laterite soil's physical and mechanical engineering properties. According to the analysis, a requirement required at 28 days of curing time, a mixture of 12 per cent ESP and 8 per cent SS yielded a maximum strength of 570 kN/m^2.

However, it is possible to use it to stabilise the laterite soil used to construct the fill layer for light-traffic roads (Oke & Olowoyo, 2019). According to Jiang, other studies examined the effectiveness of quicklime and calcium carbide residue (CCR) on low-plasticity clay soil used as a road subgrade in China. The findings of their investigation showed that, on average, the influence of calcium carbide was more significant than that of quicklime at the exact percentages of 4 and 6 per cent (Sujeeth, 2015). The results showed that, in contrast to CCR, quicklime's effectiveness was not influenced by the degree of compaction.

In conclusion, various additives with benefits, disadvantages and effectiveness stabilise the soil. However, to stabilise the subgrade for transportation, Ordinary Portland Cement, OPC cannot replace several non-conventional, ecologically friendly stabilisers (Wahab et al., 2021). Based on research by Al-Jabban believes that cement is more successful in enhancing the engineering qualities of fine-grained soil. Additionally, the study found that using cement as a stabiliser and curing it for seven days increased UCS significantly more than the study of Bhurtel and Eisazadeh, in which lime was used to treat Bangkok clay (Marathe et al., 2015). It is clear from these comparisons that OPC has an immediate reaction and far higher effectiveness than other types of binder agents, including lime, MICP and TX-85. Consequently, OPC could still be referred to as a useful stabilising or binder agent when constructing the road infrastructure. Although various studies have been done to find the best binder agent OPC for low-volume roads following Malaysia Public Work Department regulations.

b) Cement Stabilisation

According to Thasgesen (1996), the mechanism of the stabilising agent is the cement, which is the mixing of small amounts known as cement modification into the soil. Furthermore, the cementing agent has many effects on stabilisation, such as improving shear strength, reducing moisture sensitivity in modification and considerably increasing tensile strength (Amu et al., 2011). This cement stabilisation is suitable for laterite soils, including granular. Extra amounts of cement are needed in clay-rich and poorly graded sands and are subject to moisture content. Many researchers are working to improve the laterite soil properties by using cement to stabilise the soil. Mengue researched the mechanical qualities of Cameroon's fine-grained lateritic soil in 2017 and determined that 3 per cent and 6 per cent cement are sufficient to achieve the acceptable mechanical performance of stabilised lateritic soil for usage as sub-base and base layer, respectively (Biswal et al., 2020).

Table 4.2
UCS Results After 7 Days Curing Time

Cement Content (%)	UCS Results After 7 Days Curing Time (kPa)		
	(Dabou et al., 2021)	(Wahab et al., 2021)	(Marathe et al., 2015)
0	580	200.74	556
3	1420	391.35	934
6	2880	1233.15	1858
9	3400	1737.52	2077
12	3590	1899.6	2164
Optimum (%)	6	6	6

The cement content is 3%, 6%, 9% and 12% to find the optimum content of cement as a stabiliser. Based on the graph above, the comparison between UCS results is based on the cement content via UCS test. It can be easily seen that the cement is more effective than other types of stabiliser agents (Wahab et al., 2021). From studied, it overall chosen that the optimum content of cement is 6% for use as subgrade, with values more than 800kPa, stated by Malaysian Standard. Even though the UCS of cement content is 3%, which is more than 800kPa, according to Marathe, OMC and MDD of 6 % cement content were optimum. MDD values obtained for the lateritic soil treated with cement, from both modified and standard proctor compactions, immediately after mixing, showed an increase to 6% of cement and then it decreased. Similarly, OMC values obtained for the various percentages of cement (0, 3, 6, 9 and 12%) with lateritic soil immediately after mixing decreased to 6% of cement and then increased gradually. The mix design report also stated that the minimum requirement for the strength of the stabilised layer was obtained when four per cent (4.0%) of cement was utilised as a soil stabiliser (Razali & Che Malek, 2019). Thus, this study focuses on finding 3%, 4 %, 5% and 6%.

4.1.3 Laboratory Process

This research is based on laboratory testing. Moisture Content, Particle Size Distribution, Atterberg limits, Compaction and pH Test are among the laboratory studies performed on sub-grades or soil. While the modified soil test is the same as the existing natural soil test, UCS is required for strength. The soil's characteristic material was determined according to BS 1377: Part 1-9, 1990 and the design mix tests were carried out according to BS 1924: Part 1-2, 1990. The test will be carried out with varying concentrations of cement. Then, the tests were carried out in this thesis:

Table 4.3
Mix Design to be Used.

Batch	Sample	Type of Laboratory Tests
Control sample	Natural Subgrade/Soil, i.e. Laterite Soil (100%)	• Moisture Content (MC) • Specific Gravity • Particle Size Distribution (PSD) • Atterberg Limit Test • Proctor Compaction Test • Unconfined Compressive Strength (UCS)
1	Laterite + 3% OPC Cement	• Proctor Compaction test • Unconfined Compressive Strength (UCS)
2	Laterite + 6% OPC Cement	
3	Laterite + 9% OPC Cement	
4	Laterite + 12% OPC Cement	

Most of the methods above used in the preparation of the natural soils are sieving and oven drying. However, there are different sizes of sieving and weights to use. In addition, for moisture content, use the original soil without sieving it, with crumble the soil only. The method of sample preparation is in accordance with BS 1377: Part 1-2, 1990. Meanwhile, the preparation method for mixing soil samples was according to BS 1924: Parts 1- 2, 1990. Soil samples are mixed with Ordinary Portland Cement and a specified percentage of water is added to act as a medium for the reaction process. The design mix configuration Ordinary Portland Cement (OPC) is 3%, 6%, 9% and 12%. It was done using the "dry mix procedure", according to Ndububa in 1995. Any soil stabilisation had to be well mixed with soil when it was dry and then the water had to be added gradually while mixing was still going on to reach the desired consistency. The design mix amount was 3 per cent, 6 per cent, 9 per cent and 12 per cent used in this study. To prevent segregation, the mixing was done thoroughly through the engineering process.

4.1.4 Initial Laboratory Results

Summary of Laterite Soil Properties. Refer Appendix 2.

Table 4.4
Summary of Material Properties for Laterite Soil

Physical and Mechanical Properties	Value
Moisture Content (%)	18.5
Gravel (%)	0.6
Sand (%)	1.4
Silt (%)	94.91
Clay (%)	2.91
Liquid limit (%)	40.85
Plastic limit (%)	28.84
Plasticity Index (%)	12.01
Classification	CI
Maximum Dry Density, MDD (Mg/m3)	1.62
Optimum Moisture Content, OMC (%)	25.92
Unconfined Compressive Strength After Curing 7 Days (kPa)	195

According to Table above, the moisture content for laterite soil ranges from 18.25 per cent to 18.86 per cent, with an average value of 18.50 per cent. These outcomes are consistent with research ranges, said Bell and Gidigasu. It is essential to note that the natural moisture content values are within those ranges indicated by past studies on laterite soils, even if it is unnecessary to comply with these requirements. The average moisture content around Ayodele in 2020 in Osun State, southwest Nigeria, which was 18%, can be compared to the laterite of this study. A specific area's natural moisture content can be determined. This knowledge can be used to determine the optimum way to apply it to engineers.

The American Association of State and Highway Transport Officials states that soil is considered to have finer grains if its particle size distribution is greater than 35 per cent and a coarser grain if it is less than 35 per cent (Ayodele et al., 2020). The sample had an acceptable content (P200) greater than 35 per cent, or 98.2 per cent, according to the results of the particle size distribution, making it fine-grained soil. Additionally, it is necessary to test the hydrometer test and the Atterberg limit. Consequently, the particle size distribution graph was plotted using the hydrometer test and sieving. Material of Gravel, sand, silt and clay made up 0.6, 1.4, 94.91 and 2.91 percent of the particle size distribution graph, respectively. The results shown that the laterite soil was a fine soil with a significant clay and silt content. The soil classified as well-graded clayey silt. In accordance with previous research, the laterite soil is classified as sandy clay in Kenya, sandy silt in Johor, Malaysia and gravelly sand in eastern India. The laterite soil in Selangor used in this study, which was high in silt, was similar to the laterite soil in Johor.

According to the research conducted by the earlier authors referenced, the Atterberg limit test for laterite and lateritic soils yields a noticeably wide range of values. Gidigasu states that this is affected by the various climatic and soil conditions in the vicinity of the location where the soil samples are gathered. The LL and PL values were 40.85% and 25.84%. It can be said that the kaolinite clay material is contained in the soil samples. This is because kaolinite soils have liquid limit values of between 35 and 100 percent and plastic limit values of between 20 and 40 percent (Ayodele et al., 2020). The liquid limit which was from 20 mm of the penetration to the linear line. The PI is variable between the LL and PL values which was 12.01%. The range of water concentration where the soil changes from a plastic condition to a liquid state is known as the plasticity index. The Atterberg limits or consistency limits acquired from the tests assist in describing the consistency of the fine-grained laterite soils. The name relationship is due to this. From the plasticity chart, based on the LL value and the PI, classify the laterite soil as clay of intermediate plasticity and considered as low plasticity index.

Furthermore, the laterite soil is acceptable for stabilisation because the PI is greater than 10% and the percentage of fine material is greater than 25% (Malaysia Public Work Department, 2017). From studied, cement was suitable as a binder agent for the stabilisation of laterite soil.

The sub grade or soil material become denser by compaction, increasing their dry unit weight when more water is added. This improves shear strength, reduces permeability and lessens the effect of the settlement. The MDD beyond which soil starts to lose density is the biggest restriction when water assists in maximising the dry density. The moisture level that corresponds to that is the OMC for that specific soil. From the compaction test, the MDD and OMC were 1.62 Mg/m^3 and 25.92% based on figure below. Figure 4.3, illustrates how more water was added to the soil, effectively replacing the soil particles with water and lowering the soil's density.

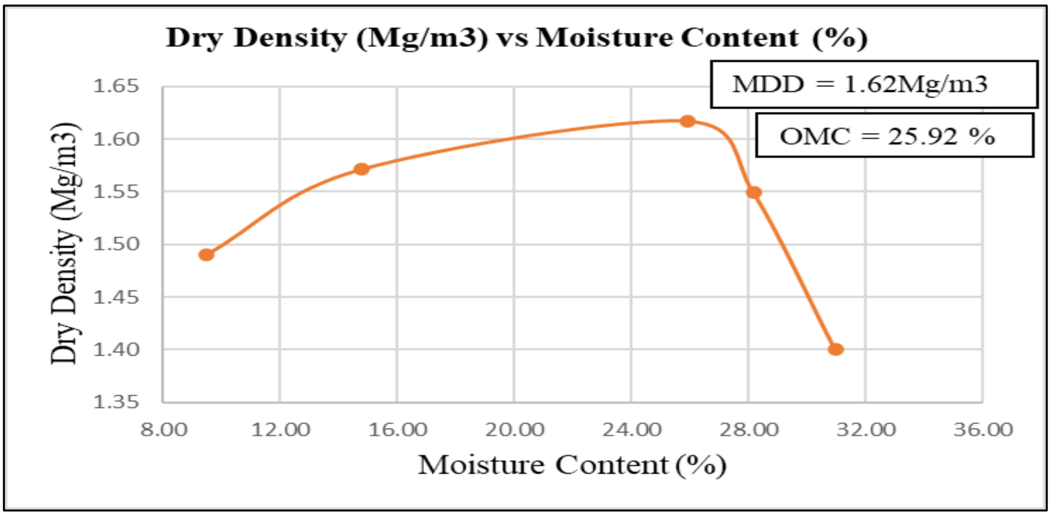

Figure 4-3 The Dry Density (Mg/m3) VS Moisture Content (%) – Refer Appendiz 2 (B)

Unconfined Compression Strength (UCS). Refer Appendix 2.

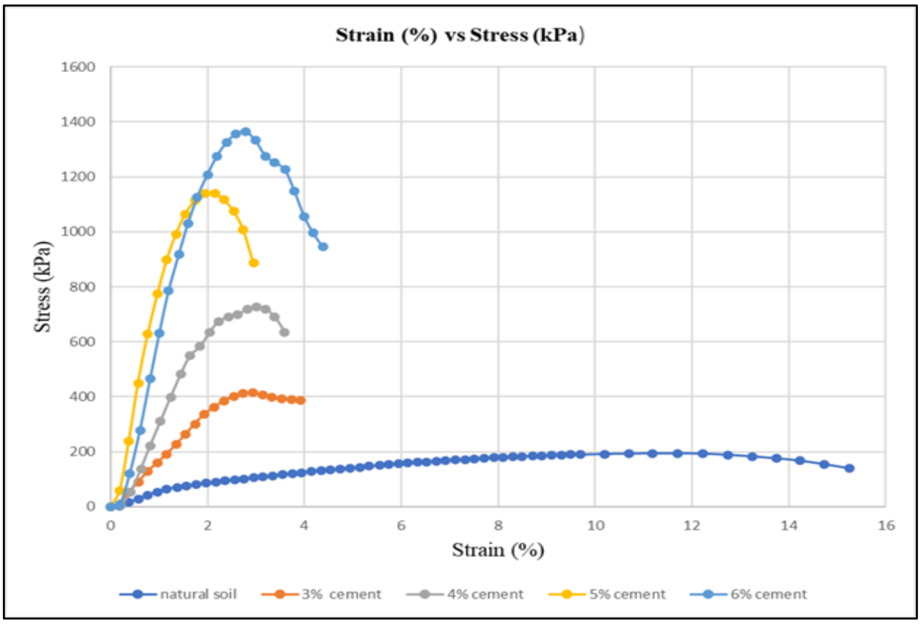

Figure 4-4 Strain (%) vs Stress (kPa)

Figure above, showed the Stress-Strain Curve in kPa of a soil sample for 0%, 3%, 4%, 5% and 6% of cement content. From the results shown above, the maximum stress was 195 kPa, 480 kPa, 728 kPa, 1140 kPa and 1365 kPa for 0%, 3%, 4%, 5% and 6% of cement content with axial strain of 11.7%, 3.35%, 3.01%, 1.95% and 2.79%. The dry density for 0%, 3%, 4%, 5% and 6% of cement content were 1.94 Mg/m^3, 1.79 Mg/m^3, 1.96 Mg/m^3, 1.92 Mg/m^3 and 1.96

Mg/m^3. The shear strengths in kPa for 0%, 3%, 4%, 5% and 6% of cement content were 97, 240, 364, 570 and 68.

Unconfined Compression Strength (UCS). Refer Appendix 2.

Table 4.5
Results of UCS After Curing 7 Days

Cement Content (%)	UCS after Curing 7 days (kPa)		Dry Density (Mg/m3)	Axial Strain (%)
	Sample 1	Sample 2		
0	161	195	1.94	11.7
3	386	416	1.93	2.93
4	659	728	1.96	3.01
5	1010	1140	1.92	1.95
6	1145	1365	1.96	2.79

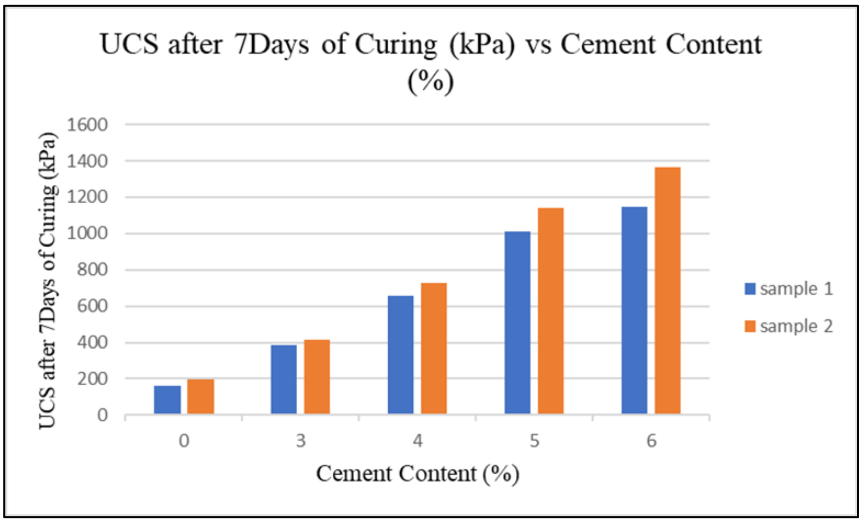

Figure 4-5 Graph of UCS After Curing 7 Days, kPa vs Cement Content (%) of 2 samples.

A cohesive soil sample, which includes clay, must undergo the unconfined compression test (UCS) to ascertain its unconfined compression strength (UCS) and failure shear strength parameter. Based on figure above, it can be shown that as the stress reaches its maximum level, it also decreases with axial strain. The UCS test was conducted using the OMC. Natural laterite soil without mixed with cement had an UCS of 195 kPa after 7 days of curing, making it unsuitable for use as a subgrade. Then, the laterite soil required to stabilise with cement. From previous studies, all laterite soil was not suitable as subgrade and needed to stabilise at a range of 128.88 kPa to 580 kPa. As the cement amount and curing period increased, the strength parameters (CBR and UCS) also rose, causing the UCS to increase from 195 kPa to 1365 kPa when stabilised (Dabou et al., 2021).

According to the Malaysia Public Work Department, the Unconfined Compression Test (UCS) for 7-day strength with moist curing at 25 degrees Celsius, for unpaved roads, needs more than or equal to 800 kPa following B.S. 1881, Part 116. According to figure 4.6, the cement content was 5 and 6 percent greater than 800 kPa, which was 1140 and 1365 kPa, respectively. However, although having 6% greater strength than 5% of cement, 5% of cement is the most acceptable optimum cement content for stabilising. As a result, the mixing plant must be equivalent to the volume of cement mix for the appropriate material used for subgrade stabilisation. This is because, in practice, the percentage of cement in the mix has a substantial impact on the cost of cement.

The compacted material has obtained the 95 percent range density for compacted fill, which is between 1.8 Mg/m^3 and 2.1 Mg/m^3 and the required UCS is 800 kPa, equivalent to 13 percent CBR. Figure 4.6 showed that all dry densities fall between 1.8 and 2.1 mg/m3. Additionally, the shear strength of laterite soil was 97 kPa, but it rose when mixed with cement. The shear strength was 570 kPa and the axial strain was 1.95 percent with a 5% cement concentration. Below is the test record for Moisture Content vs Dry Density.

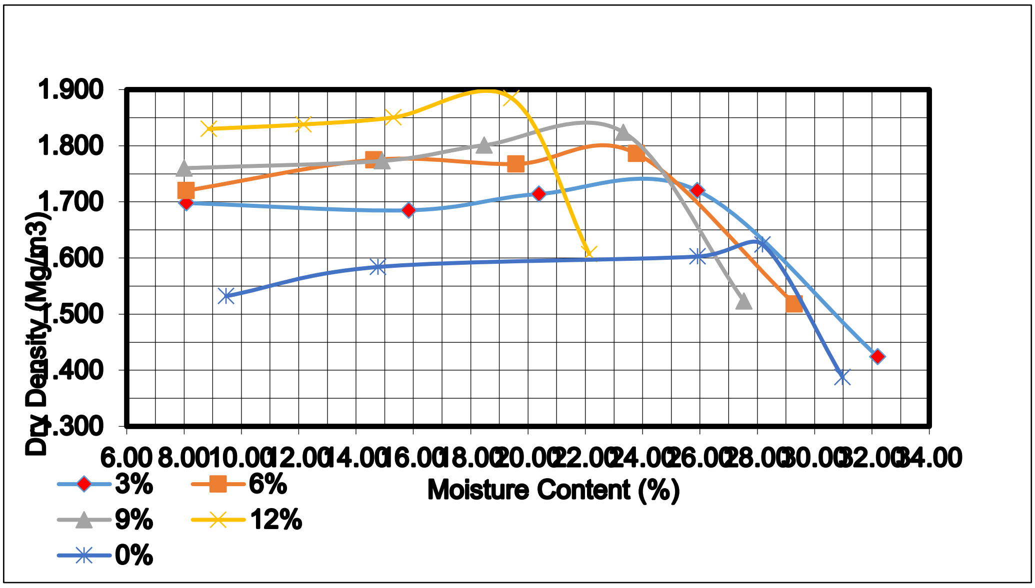

Figure 4-6 Reaction of Increased Density will Reduced Moisture Content Refer Appendix 2 (I)

4.1.5 Summary

This research is carried out to analyse the effect of sub grade/soil stabilisation using cement in altering existing material which is laterite soil properties to provide high strength and durability as a subgrade of a pavement. This study is very informative as it gives new knowledge and information on soil treatment using cement. This study is to identify the characteristic of the soil, to analysis the strength improvement of the modified laterite with selected binder agent with cement and to identify mix design for soil stabilisation. The testing procedures for the engineering properties of laterite soil including of the moisture content, particle size distribution, Atterberg limit, compaction, pH and UCS was identified. From the UCS test, the laterite soil was 195 kPa, the dry density was 1.94 Mg/m^3 and the axial strain was 11.7%. From the test result for the properties of laterite soil, the soil needs to be stabilised. This is because the PI is greater than 10%, the percentage of fine material is greater than 25%, the MDD below range 1.8 Mg/m3 to 2.1 Mg/m3 and UCS below the specification of Malaysia Public Work Department which need greater than or equal to 800kPa. The increased strength of the modified laterite soil with cement was determined with the UCS with 7 days of curing and with 4 differences percent of cement content which were 3%, 4%, 5% and 6% to modify the laterite soil. From the results of UCS, when the cement content increase, the UCS and shear strength increase too. By obtaining the optimum amount of cement content to stabilise the laterite soil, the UCS shall be done with difference percent of cement. The optimum amount of cement was identified was 5%. This is because the mixing plant must be equivalent to the volume of cement mix for the appropriate material used for subgrade stabilisation.

Other than that, the output of this study is giving knowledge on the importance to treat soil before proceeding to construct a building and roadways. Soil stabilisation is necessary in soft soil so that they can act as strong foundation before any infrastructure and commuting ways can be built. This is to ensure no fatality and economy loss happened due to damage on the construction that have been built. Furthermore, this study concentrated on building a strong foundation for laterite soil as a material for subgrade's pavement. Thus, the result of this research can influence people on how to use natural resources towards reprocessing materials for soil treatment to create innovative and cost-effective solutions for the pavement's infrastructure demands. In this study, cement can be proven as a binder agent for soil stabilisation method that can treat laterite soil to be a reliable base for any pavement's construction.

4.2 Design Mix for Marine Clay Using Lime

4.2.1 Introduction

As a developing country, Peninsular Malaysia, with a total land area of 130,268 km2, is part of Sunda land, which encompasses the islands of Borneo, Java and Sumatra, as well as the shallow oceans between them, from which several smaller islands arise (Van Bemmelen,1949). Separated into three parts of longitudinal belts, East, Central and West of Peninsular Malaysia, each belt has many different characteristics and geological evolution. The Western Belt's whole length, from the Kinta Valley to Malacca, comprises Palaeozoic rocks. The Kinta Valley is dominated by moderately thick limestones that range in age from Silurian to Permian and contain schists, amphibolite, conglomerates and other clastic as well as tiny serpentinite bodies. Rocks of Permian and Mesozoic largely coat central belts. Clay's undesirable properties include high plasticity, poverty in workability, difficult compaction and high swelling potential, which increase the construction cost.

Hence, an enhancing technique, which is stabilisation is needed before construction can be made. Over the past years, road engineers have applied many revolutions and developments to improve the parameters of cohesive soils (Cai et al., 2006). Stabilisation of subgrade/soil can increase and improve soil shear strength and soil's durability, soil shear strength and soil's durability in the overall bearing capacity of subgrade/soil. In road construction for pavement layer, with the addition of a stabilising agent in clay the pavement's subgrade can be increased. Once the subgrade/soil was stabilised, the moisture content through water absorption and permeability can be reduced in dry clay cases, in conjunction with the reaction to shrinking of materials by moisture content and temperature changes. Due to the modification, the strength development after 28 rapidly increases in the curing period during the Unconfined Compression Test (U.C.T.), which is the effect of the reaction between lime and clay (Yunus et al.,2015).

4.2.2 Marine/Soft Clay

Increasing urbanisation, to achieve the target for the development of the country and the desperation to make money, many investors are taking a brave step on exploring new locations, which area at coastal areas, to build superstructures, construction of facilities for port sports and reclamation of land. Not only buildings but coastal areas also need to construct roads to connect states. In Malaysia, marine clay is usually very thick, about 30m and is typically found in coastal area. Untreated marine clay can be a problem for geotechnical engineers due to its poverty of strength to support any load or structure in the areas. Marine/soft clay behaviour, such as high compressibility, poor drainage characteristics and low shear strength and durability, causing a primary concern in the construction near the coastal area. Marine clay's undesirable behaviour is a highly compressible soil and illustrates a mild swelling mechanism with the presence of moisture (Ruan et al., 2020).

With the inhabitancy of water, marine clay will lose its strength and become as hard as brick when it becomes dry. As it dries, which is the situation when it loses moisture, a crack will happen and based on a few studies, the cracks will go worst, which is 250mm to 500mm (Surya Karteek, 2018). These misbehaviours are due to the existence of clay minerals and organic material content in triggering marine clay's reaction activities. Keeping marine clay well drained is challenging as these soils absorb water through capillary action. In addition, due to excellent surface permeability and capillary action, these soils can become very moist and wet until the early summer months (G.S Karteek, 2018). Soft soils have high plasticity indexes with respect to liquid limit and their volume fluctuates dramatically with the changing water concentration. Based on (Zurairahetty et al., 2015), the behaviour limits of the marine clay are as follows.

Table 4.6
Index Properties of Marine/Soft Clay (Source: Zurairahetty et al., 2015).

Material Content	**Marine/Soft Clay**
Liquid Limit (%)	58
P.L. (%)	36
P.I. (%)	22
Specific gravity	2.62
Optimum Moisture Content (%)	21
Max. Dry Density (kg/m³)	1600
Organic content (%)	4.2

4.2.2.1 Quicklime

Quicklime or hydrated lime is a calcium-based stabiliser which has an influence the geotechnical and engineering properties and is often used in civil engineering applications. Quicklime is a reasonably priced substance. A chemical derivative (calcium hydroxide, of which quicklime is the basic anhydride) are significant industrial compounds. Using various stabilisation techniques and quicklime as stabilising agents an improve the soil's strength and durability. Aside from that, while lime and cement can help with the engineering properties of soils, they can also have adverse effects on other soil properties (Singh & Vasaikar, 2015). In the presence of sulphate, for example, these additives may cause undesirable expansion

(Behnood, 2018). The combination of stabilisation the method through the material's engineering process for the existing subgrade/soil and the binder agent has an impact on its properties. The selection of binder agent or cementitious materials process via soil characteristics analysis such as Particle Size Distribution (PSD) or Gradation (Sieve Analysis), chemical composition and mineralogy, Percentage of Plasticity Index, organic matter content, salt content, primarily sulphate, cation exchange capacity (C.E.C.), Lime Demand Test, specific surface area and others, all influencing the stabilisation effectiveness. In addition, the properties of stabilised soils can be affected by the type and duration of the curing condition, the construction method and quality, for example, the amount of compaction test (Eilmy et al., n.d.).

Lime is commonly used to improve subgrade soil performance by minimising volume change. Mixing with quicklime and subgrade/soil is crucial to achieve the desired results specified in the specifications. Lime also reduces soil fines by flocculating and aggregating clay particles (Little 1995). This increases the fraction of sand and silt particles in the grain size distribution (Basma and Tuncer 1991). Lime additionally reduces fine-grained soil swell potential (Kennedy et al. 1987). Soils with moisture content below optimal have a substantially greater swell potential than soils with above optimum (Sweeney et al. 1988). Soils with high montmorillonite content showed no reaction to an increase in strength via U.C.S. test. Therefore, some treatments for surface area for the cementitious chemicals have a substantial impact on the strength through mixing altogether with cement O.P.C. for strength and quicklime to reduce the moisture content.

4.2.2.2 Soil Stabilisation

Table 4.7
Various Additives for Stabilisation of Soil (Source: Reddy & Tahasildar, 2015)

Group	Binder Agent
Cementitious	Lime, fly ash, ground-granulated blast furnaced (GGBS), cement, kiln dust and silica fume.
Chemical	$CaCl_2$, KCl, Na_2SiO_3, $FeCl_3$, $Mg(OH)_2$, $Na(OH)$, $NaCl$, $MgCl_2$ and $AlCl_3$
Non-Cementitious	Stone dust, quarry dust, aggregate waste, rock waste powder, crusher dust, granite sawdust and sand.

Soil modification explains the process of altering chemical and physical characteristics in soils to achieve more strength and durability for construction purposes. When it comes to reaction to increase physical soil, such as shear strength and bearing capacity. Soil stabilisation is a process of applying measured compaction with the addition of a binder agent to generate a better soil characteristic that is required in the material's engineering properties. To achieve desired soil properties, a variety of existing technologies and materials are available to choose. An operational of soil stabilisation and selection material binder agent plus existing sub grades/soil, named as stabilisation method. The soil stabilisation method is more commonly the method being used for marine/soft clay treatment to change the material characteristic to improve the strength and save time and cost. The cementitious, chemical additives and non-cementitious are all examples of the binder agents, according to Reddy & Tahasildar (2015).

4.2.2.3 Quicklime Treatment

Soil engineering qualities may be greatly improved by applying lime. Enhancement may be divided into two broad categories: soil modification and soil stabilisation (Mohammed Al-Bared & Marto, 2017). Quicklime able to enhance almost all fine-grained soils, although it has the greatest impact on clay soils with a moderate to high ductility. For most parts, modification happens primarily to the substitution of calcium cations provided by hydrolysed lime that is already present on the clay mineral's surface. Cementitious products are formed in high-pH environments when lime combines with clay minerals to generate hydrated lime. This reaction alters the clay mineral properties. Plasticity and swelling are decreased, moisture-holding capacity is lowered and stability is increased.

This method of soil stabilisation involves mixing lime into the soil to increase its strength and resiliency (Babu & Poulose, 2008). The proportion of lime added/mixed into the existing soil varies depending on the characteristics of the native soil. The higher the plasticity, the more lime is required/usually mixed in. Quicklime is a naturally occurring material in nature. Although treating soil with lime is a popular method of soil stabilisation, it is most common on paved roads. Typically, treating unpaved roads with lime is prohibitive. Geographical regions frequently dictate whether it can be used to stabilise soil. The subgrade of pavement can be strengthened and stiffened by mixing/adding quicklime to existing subgrade/soil.

4.2.3 Laboratory Process

The Soil Mechanics, Laboratory of the School of Civil Engineering, test was conducted to identify the fundamental properties of natural soil. There were determined that several initial tests were based on BS 1377: Part 2:1990, specifically for moisture content, specific gravity, particle size distribution and Atterberg limit test. The soil's chemical content test (pH test was conducted using BS 1377: Part 3:1990 at the Environmental Laboratory of the School of Civil Engineering using a pH meter and its buffer solutions. Furthermore, the density compaction of the soil sample was identified through the Proctor Compaction test by BS 1377: Part 4:1990.

Following the initial test for physical properties, a series of Unconfined Compressive Strength (U.C.S.) tests are conducted by BS 1377: Part 7:1990 to determine the improvement of material characteristics and the increased strength properties of soil sample before and after reinforced through mixing with different proportions of binder agents.

Figure 4-7 Example Laboratory Test Equipment. Source: MMTS Laboratory

4.2.3.1 Study Area

The soil used in this research was marine clay. The soil was taken at a landfill area in Nibong Tebal, Pulau Pinang. Clayey soil was selected as the testing soil. It is regarded as problematic because it is particularly vulnerable to water logging because of its high mineral content, such as montmorillite, which causes structure settlement. The soil sample should be more than 30 kg for laboratory testing to conduct all the proposed testing.

Figure 4-8 The Site Location of the Case Study

The quicklime being used was obtained from RCI Lime Sdn. Bhd. The certificate and analysis for this quicklime powder are shown below.

Table 4.8
A Proportion of Quicklime Powder Is Being Used.

Parameters	Results	Test Method
Calcium Oxide, CaO (%)	90.62	MS 850: 2010
Magnesium Oxide, MgO (%)	2.41	MS 850: 2010
Insoluble Matter, Including Silicon Dioxide, SiO_2 (%)	0.36	MS 850: 2010
Ferric Oxide, Fe_2O_3 (%)	0.02	MS 850: 2010
Aluminium Oxide, Al_2O_3 (%)	0.25	MS 850: 2010
Passing 1 mm sieve (%)	100.00	ASTM C110

4.2.3.2 Design Mix Process.

The design mix process for this research is shown in Table below. Usually, in the pavement layer, the normal range for lime stabilisation takes about 3% to 6% lime addition by mass of the dry soil (Zukri & Ghani, 2014). The specimens get stronger as the pH drops. Three percent of the lime stabilised with soil and quadrupled its strength, but more lime resulted in a more extended response with more strength (Eades et al., n.d.). The proposed mix design on the percentage of lime for this research is based on past researchers who conducted a study on soil stabilisation using quicklime/hydrated lime on clayey soil.

Table 4.9
Mix Design Used in This Study.

Batch	Sample	Type of Laboratory Tests
Control sample	Natural Subgrade/Soil i.e. Marine Clay (100%)	• Moisture Content (MC) • Specific Gravity • Particle Size Distribution (PSD) • pH Test • Atterberg Limit Test • Proctor Compaction Test • Unconfined Compressive Strength
1	Marine Clay + 3% lime	• Proctor Compaction test • Unconfined Compressive Strength (U.C.S.) test
2	Marine Clay + 6% lime	
3	Marine Clay + 9% lime	
4	Marine Clay + 12% lime	

4.2.4 Initial Laboratory Results

4.2.4.1 Material Characteristic of Marine Clay

By measuring material characteristics via the moisture content, specific gravity, particle size distribution and Atterberg limit test, the physical characteristics of natural soil are identified based on the British Soil Classification System. The subgrade/soil samples are categorised as clayey silt of Intermediate Plasticity (CI) based on the physical characteristics tests that were performed. Information about the soil index characteristics.

Table 4.10
Material Characteristic of Soil Sample.

Soil Properties	Marine Clay (Nibong Tebal, P. Pinang)
Colour	Dark brown
Natural moisture content (%)	16.72
pH	3.302

Specific gravity (Mg/m_3)	2.605
Particle size distribution (%)	
a) Gravel	0.58
b) Sand	2.42
c) Silt	70.04
d) Clay	26.96
Plastic limit, P.L. (%)	15.76
Liquid limit, L.L. (%)	35.85
Plasticity index, P.I. (%)	20.09
Soil Classification	**CI**

4.2.4.2 Atterberg Limit Test

Sub grade/soil subgrade is to deform without cracking or collapsing, known as plasticity. The lubricating effect of water films between adjacent particles is one of the primary properties of all cohesion soils. Soil engineering relies heavily on information about a soil's plasticity and standard limits tests established to measure it. The shear strength and compressibility of fine soils are greatly influenced by their plasticity and as a result, this property is used to classify soils. The Atterberg Limit is the point at which soil behaves similarly to plastic when water is high. It determines the moisture content at which fine-grained clay and marine clay appear in four states: solid, semi-solid, plastic and liquid. The Atterberg limits are used to measure fine-grained soil for critical water content. It has three limits, including shrinkage, plastic and liquid.

Table 4.11
Atterberg Limit of Soil Sample for Marine Clay at Nibong Tebal, Penang

Liquid Limit (L.L.)	35.85 %
Plastic Limit (P.L.)	15.76 %
Plasticity Index (P.I.)	20.07%
Soil classification	**CI**

4.2.4.3 Proctor Compaction Test

The process of identifying soil density using an engineering process known as compaction. It causes the particles in the soil to rearrange themselves, leading to a decrease in the amount of space between them. When soil is compacted to a very high degree, there are relatively few voids existing in the soil, which may result in the soil having a more extensive unit weight. Because the soil in its natural state is only loosely connected, it needs to be compacted to increase its bearing capacity (Hussain & Dash, 2016). The compaction test has shown the relationship between the dry density and moisture content. A few compaction tests are done on both natural soil/marine clay and subgrade/soil that has been stabilised by adding quicklime in varying quantities, such as 3%, 6%, 9% and 12% by weight of dry soil.

Table 4.12
Optimum Moisture Content and Maximum Dry Density Soil Sample.

Sample	**O.M.C. (%)**	**M.D.D. (Mg/m³)**
Natural soil	55.8	1.129
Soil (Marine Clay) + 3% Quicklime	55.9	1.210
Soil (Marine Clay) + 6% Quicklime	50.83	1.266
Soil (Marine Clay) + 9% Quicklime	47.19	1.270
Soil (Marine Clay) + 12% Quicklime	40.23	1.347

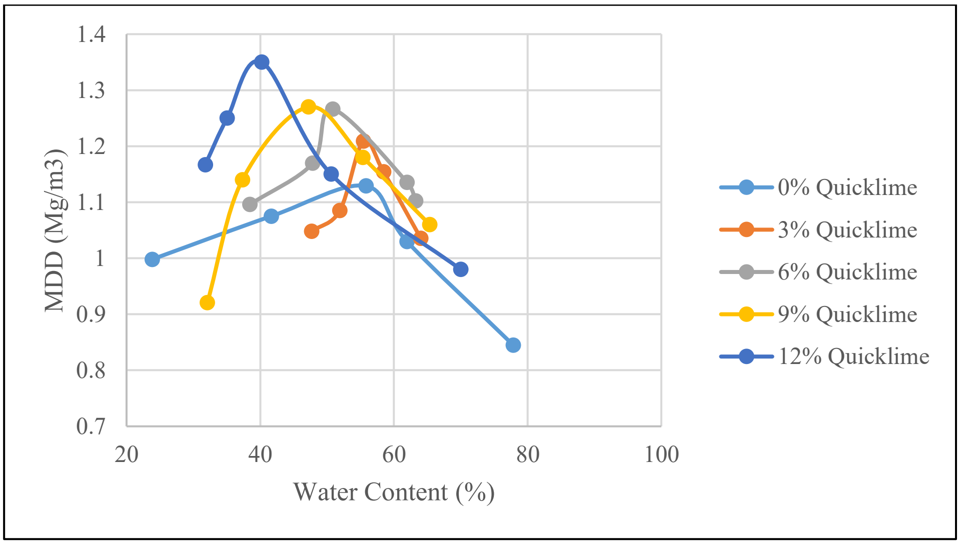

Figure 4-9 Density Curves for Sub Grade/Soil Stabilised with Quicklime.

With an increase in the amount of quicklime from 0 per cent to 12 per cent by weight of the soil, the Maximum Dry Density (M.D.D.) increased from 1.129 Mg/m3 to 1.347Mg/m3, as indicated in the graph that refers to Figure above, the results displayed to demonstrate that the Optimum Moisture Content (O.M.C.) gradually decreased from 55.8 per cent for natural soil to 40.23 per cent for soil stabilised with 12% quicklime. According to the findings, one conclusion that can be drawn is that the decrease in water content, which relates to an increase of per cent in the amount of quicklime in the subgrades/soil, caused the increasing the subgrades/soil's dry density, as a result, the addition of lime had the effect of lessening the compacted soil's total mass and volume. In addition, lime treatment has resulted in flattening compaction curves, indicating the modification in density is due to changes in moisture content. In other words, the required density of subgrade/soil can be defined with lime treatment over a broad range of controlling moisture content.

4.2.4.4 Unconfined Compressive Strength (U.C.S.) Test

A test of the material's strength through the Unconfined Compression Strength Test (U.C.S.) is carried out to establish the maximum compressive strength of marine clay after lime has been added. The unconfined compressive strength of subgrades/soil is defined as the maximum compressive force per unit area that a subgrade/soil sample can withstand.

The summarizes the results obtained from UCS conducted after 7 days of curing with varying lime content.

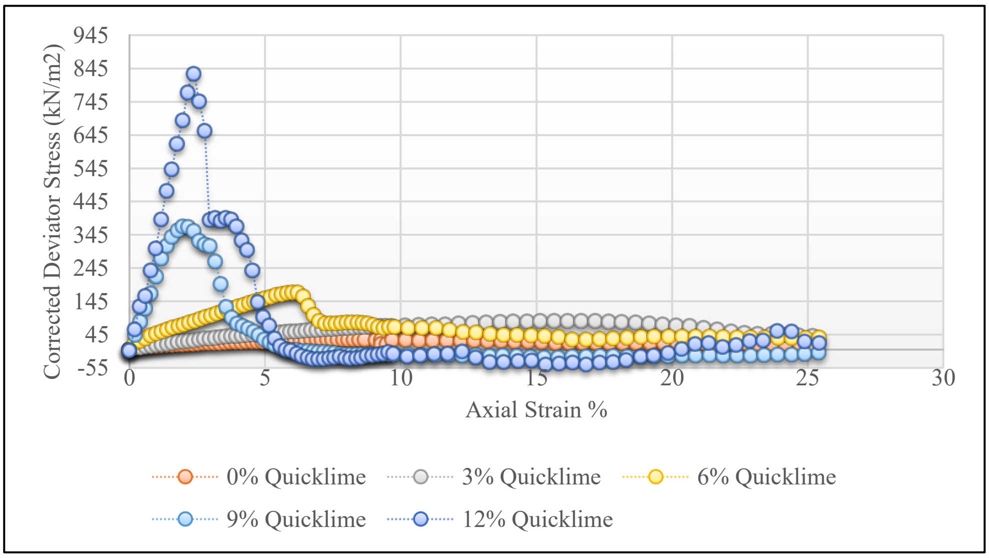

Figure 4-10 Corrected Deviator Stress vs Strain Curves of Soil Sample with Quicklime.

The Unconfined Compressive Strength of the grade/soil sample is gradually increased after 7 days of curing time. After curing days, an increase strength can be seen as the soil material samples become harder. The small rise in strength between natural soils and the subgrade/soil added with 3% quicklime is due to an adaption of soil to stabiliser added. Furthermore, by increasing the percentages of quicklime, the Unconfined Compressive Strength test of marine clay is also reinforced. Figure 6 shown the curves obtained from plotting the stress and strain value of 0%, 3%, 6%, 9% and 12% of quicklime as the soil stabiliser. As can be seen each curve reach peak value of strength but start to drop at certain point due to brittle structure of soil sample. The duration of 7 days curing time, reducing the moisture content of sample and the sample will undergo brittle shear slip failure due to dry conditions of sample.

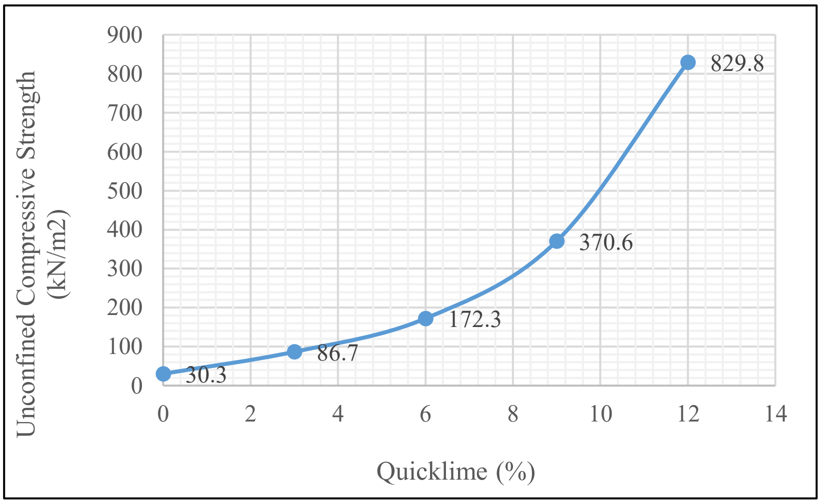

Figure 4-11 UCS vs Strain Curves of Soil Sample Added with Different Amount of Quicklime

In this research study, the utilisation of lime as stabilising material in marine clay are investigated and analysed. The physical test was done on the soil before conducting the mechanical test on the marine clay. Based on the British Soil Classification System (BSCS) and the Plasticity Chart, the sample of soil from the study area is classified as clayey SILT with Intermediate Plasticity (CI). The effect of adding quicklime to a soil sample at different rates of 3%, 6%, 9% and 12% by weight of soil caused the Maximum Dry Density (MDD) increased while the Optimum Moisture Content (OMC) decreased. This is because the quicklime behaviour that absorb the moisture in soil through pozzolanic activities. With more lime, cations near negatively charged clay surfaces grow. This charge imbalance in pore water causes osmosis. Charge on the clay surface prevents ion diffusion. Free water molecules diffuse towards the clay surface to equalise charge. In such a system, hydration and osmotic forces offset long-range attractive forces. The overall force becomes repulsive (Mitchell and Soga, 2005). It separates clay particles, creating a scattered structure that allows particles to slide over each other during compaction. Other than that, through this study, the strength of modified soil is determined. Through a series of Unconfined Compressive Strength test, the strength of soil sample increased with the increasing percentages of lime. The maximum compressive strength obtained is 829.8 kPa when marine clay is added with 12% of quicklime by the weight of the soil. Lime treatment flattens compaction curves, indicating density variation with gradually decreasing of moisture content. With lime treatment, soil density can be achieved over a larger moisture range. Overall, it can be concluded that quicklime can be used for altering the problematic soil, this can be proved as the characteristic of soil such as strength increase gradually. The untreated soil shows significant changes when added with lime. This shows a big difference between the strength of soil sample before and after being treated.

Table 4.13
Physical Properties of Marine Clay Soil Sample

Soil Properties	**Marine Clay**
Colour	Dark brown
Natural moisture content (%)	16.72
pH	3.302
Specific gravity (Mg/m_3)	2.605
Particle size distribution (%)	
Gravel	0.58
Sand	2.42
Silt	70.04
Clay	26.96
Plastic Limit, PL (%)	15.76
Liquid Limit, LL (%)	35.85
Plasticity Index, PI (%)	20.09
Soil Classification	**CI**

Table 4.14
Optimum Moisture Content and Maximum Dry Density Marine Clay Soil Sample

Sample	**OMC (%)**	**MDD (Mg/m^3)**
Natural soil	55.8	1.129
Soil + 3% Quicklime	55.9	1.210
Soil + 6% Quicklime	50.83	1.266
Soil + 9% Quicklime	47.19	1.270
Soil + 12% Quicklime	40.23	1.347

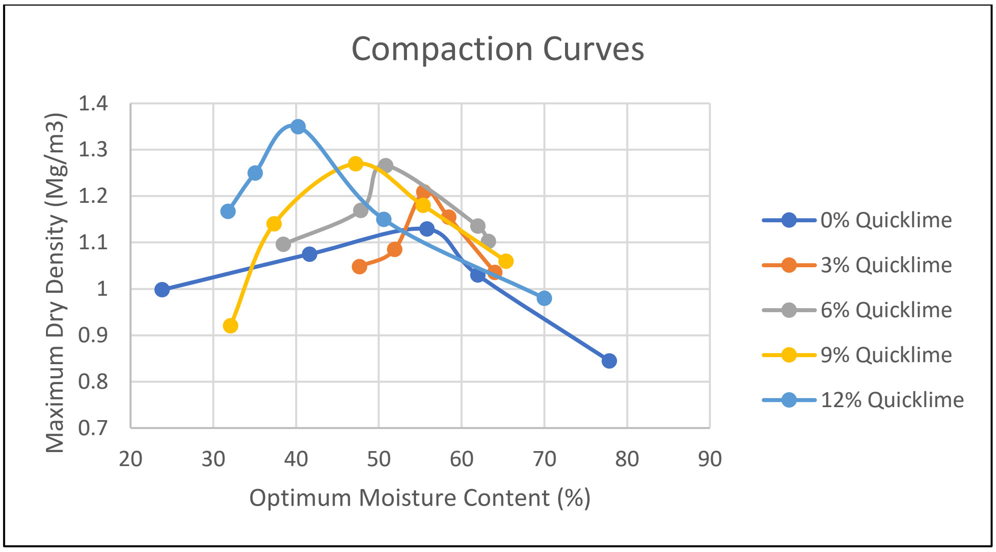

Figure 4-12 Compaction Curves for Marine Clay Soil Stabilised with Quicklime

With an increase in the amount of quicklime from 0 percent to 12 percent by weight of the soil, the Maximum Dry Density (MDD) increased from 1.129 Mg/m3 to 1.347Mg/m3, as indicated in the graph. On the other hand, the results that are displayed is to demonstrate that the Optimum Moisture Content (OMC) gradually decreased from 55.8 percent for natural soil to 40.23 percent for soil that has been stabilised with 12% quicklime. According to the findings, one conclusion that can be drawn is that the decrease in soil content, which corresponds to an increase of percent in the amount of quicklime in the soil, is what caused the increased in the soil's dry density. It was clear that the addition of lime had the effect of lessening the compacted soil's total mass and volume. In addition, lime treatment has resulted in flattening compaction curves, which indicates that the modification in density due to changes in moisture content. In other words, the required density of a soil can be obtained with the use of lime treatment over a broad range of moisture content.

Furthermore, continued research should also aim to refine the existing empirical formula to account for variables such as curing time, soil mineralogy and environmental conditions. This would enhance the formula's applicability across diverse marine clay profiles in Malaysia. The potential integration of advanced techniques such as machine learning could also provide innovative ways to model the complex interactions between lime and marine clay, further improving the accuracy and usability of predictive models.

By expanding on these findings and addressing the identified gaps, future developments in this field could significantly improve the efficiency, reliability and cost-effectiveness of stabilising marine clays, particularly in Malaysia. This ongoing effort will contribute to sustainable infrastructure development and long-term geotechnical stability in coastal and marine environments.

4.3 Comprehensive Test Results from Landfill Project

4.3.1 Introduction

Expansive clay soils, which undergo significant volume changes with moisture, contain a diverse mix of minerals. These include various clay minerals like kaolinite, illite, mixed-layer clays and montmorillonite, as well as non-clay minerals such as quartz and organic or colloidal matter. Even small amounts of certain clay minerals can greatly influence the physical properties of a clay deposit. Various efforts have been made to address the negative characteristics of these soils such as excessive swelling, shrinkage and reduced load-bearing capacity. One common approach to mitigate these challenges involves replacing expansive clays with materials that are less sensitive to moisture fluctuations. Another approach is to stabilise/strengthen of the soil through addition of various supplementary cementitious material like cement, fly ash, lime, bagasse, rice husk, coconut fibres, polypropylene fibres, recycled carpet fibres and various waste materials (David Boardman, 2001) (Bell, 1996). These materials have been chosen based on their availability, cost-effectiveness and performance. An example of a chemical method for modifying the physical and chemical properties of soils is stabilisation with lime. Lime, whether in the form of quicklime ($CaO(s)$) or hydrated lime ($Ca(OH)_2(s)$), has been used to enhance soil properties, particularly in clayey soils. Lime stabilisation is commonly used in road construction projects across Europe, United States, Australia, where its ability to increase MDD, reduce OMC and increase strength makes it ideal for stabilising weak subgrades (C.D.F. Rogers, 1996).

Boardman et al (2001) noted that stabilisation and solidification in lime-clay mixes likely occur due to a thinner water layer in the clay. When lime ($CaO(s)$ or $Ca(OH)_2(s)$) is added to a soil-water system, it initiates several chemical reactions that profoundly alter the physico-chemical properties of clay soils. These reactions bring about immediate and long-term changes and have been extensively documented in the literature. Adding $CaO(s)$ to a soil-water system triggers an exothermic hydration reaction that produces $Ca(OH)_2(s)$, releasing significant heat energy (approximately 173 × 10^9 J per kilogram of $CaO(s)$ added). This reaction not only removes water through hydration and evaporation but also improves soil workability by reducing water content. Importantly, the hydration reaction leads to high concentrations of $Ca^{2+}(aq)$ and $OH^-(aq)$ ions in the soil pore water. These ions dissociate and initiate a sequence of reactions influenced by soil composition, mineralogy and pore water chemistry. Initially, cation exchange occurs where Ca^{2+} ions replace existing cations like sodium or potassium on the clay mineral surfaces. This exchange, coupled with an increase in pH from OH^- ions, enhances mineral particle flocculation causing the clay particles to aggregate. As a result, the soil's plasticity properties decreases and workability improves. Physically, the soil becomes drier and more friable to allow for facilitating construction and compaction.

Adding lime, particularly quicklime, has significant effects on soil properties according to Bell (1996). Lime rapidly reduces soil moisture by absorbing it and causing an exothermic reaction that evaporates water. Initially, lime bonds with clay minerals, enhancing soil workability without immediately increasing strength. However, beyond saturation, excess lime initiates a cementing process crucially dependent on its reaction with clay minerals. This process increases the plasticity of soils, particularly montmorillonite clays, while minimally affecting kaolinitic clays and fine quartz are less affected. Lime also alters liquid limits, decreasing them for montmorillonite and thereby reducing its volume variability. Moreover, lime enhances the plasticity and liquid limit of soils and improves their strength and California Bearing Ratio (CBR). Optimal strength gains are typically achieved with 4-6% lime content, are influenced by factors such as mixing water content, curing duration and temperature, with higher temperatures above 30°C notably accelerating these improvements. (Bell, 1996)

Rogers (1996) found that within three to four days, the plasticity of lime-modified soil decreases, making it less prone to swelling and shrinking compared to untreated soil. This early plasticity reduction enhances the soil’s stability, particularly in applications requiring fast-track stabilisation solutions.

In conclusion, when lime is mixed with clay soils, the chemical and physical reactions described earlier eventually lead to the development of cementitious bonds. Over time, pozzolanic reactions occur, wherein silica (SiO_2) and alumina (Al_2O_3) from the clay minerals react with calcium hydroxide to form calcium silicate hydrate (C-S-H) and calcium aluminate hydrate (C-A-H) compounds. These cementitious products bond the soil particles together, significantly increasing its strength and stability.

In the Malaysian context, the availability of materials such as lime and cement plays a critical role in soil stabilisation practices. Lime and cement are widely available in Malaysia, making them practical choices for addressing the long-standing issue of weak and problematic soils. Considering the prevalence of marine clay in areas like Klang, Langkawi, Kedah and other coastal cities, research into the use of lime for stabilising marine clay is particularly relevant. Marine clay is characterized by high water content, low shear strength and sensitivity to salinity, all of which pose significant challenges for construction and infrastructure development in these regions. A targeted study focusing on the properties and stabilisation of marine clay with lime is essential to develop cost-effective and locally applicable solutions.

Given the widespread occurrence of marine clay in Malaysia and its challenging properties, a continued focus on finding combination of stabilisation material is critical. A long-term research agenda aimed at optimizing the use of lime and lime-cement combinations for marine clay would provide valuable insights and practical solutions. Furthermore, developing an empirical formula that correlates lime content with the maximum dry density (MDD) specific to Malaysia's marine clay would be a significant step forward to help engineers and road site supervisory s decide the percentage of dosage. The developed formula should be accurate enough to enable engineers and practitioners to determine the required lime content for achieving optimal soil compaction and strength based on regional soil characteristics. This approach would not only enhance the efficiency and precision of soil stabilisation practices but also ensure their economic viability and applicability across Malaysia's coastal regions.

4.3.2 Laboratory Test Report for Design Mix Marine Clay and Quicklime

This research has identified a location or project named as Landfill Sanitary Waste (Tempat Pelupusan Pepejal dan Sampah) at Nibong Tebal, Seberang Perai, Penang. Samples are taken dated 09th May 2022, Tested on 10th May 2022. The existing material is MARINE CLAY SOIL. The activities scope of test is Proctor Test, Sieve Analysis, Atterberg Limit (Plasticity Index), Moisture content of natural soil, Hydrometer sedimentation method, UCS (Unconfined Compression Strength), Lime Demand (Eades & Grim), Particle Density test and Soaked CBR 4 days penetration test.

4.3.3 Proctor Test

The Proctor compaction test is a laboratory method of experimentally determining the optimal moisture content at which a given soil type will become most dense and achieve its maximum dry density. The first is a light compaction test using a 2.5 kg rammer (Standard Proctor). The second is a heavy compaction test using a 4.5 kg rammer with a greater drop on thinner layers of soil (Modified Proctor). For both tests a compaction mould of 1 litre internal volume is used for soil in which all particles pass a 20 mm test sieve.

Method using 4.5 kg rammer (BS Heavy) Existing Materials. In this test the comp-active effort is greater than described in the above test. The mass of the rammer is increased to 4.5 kg, the height of the drop to 450 mm and the number of compacted layers is increased from three to five. The test is performed on material passing the 20 mm test sieve using the 1 L mould, The sample preparation and the testing procedure is identical to the above test apart from the rammer used is 4.5 kg in weight with a drop of 450 mm. The moist soil shall be compacted in 5 equal layers into the mould. The number of blows applied to each layer shall be 27 blows. Use Vibration rammer machine every layer must vibration 1 minute.

Result from Proctor Test we make for get a value MDD Max Dry Density and OMC Optimum Moisture Content for Sample S1 Marine Clay, found that the original MDD is 1.467

Mg/mm³ and OMC is 24.19 %. Then for Sample S1 Marine Clay added with 3% Lime, result for MDD is 1.504 Mg/mm³ and OMC is 22.20 %. For Sample S1 Marine Clay added with 6% Lime, result shown MDD is 1.601 Mg/mm³ and OMC : 20.11 %. Sample S1 Marine Clay added with 9% Lime, result shown at MDD is 1.682 Mg/mm³ and OMC is 19.10 % and finally Sample S1 Marine Clay added with 12% Lime shown MDD is 1.750 Mg/mm³ and OMC is 18.45 %.

4.3.4 Sieve Analysis

A particle size distribution analysis is a necessary classification test for soils & gradually materials especially coarse soils, in that it presents the relative portions of different sizes of particles. From this, it is possible to determine whether the soil consists of predominantly gravel, sand, silt or clay sizes and, to a limited extent, which of these size-ranges is likely to control the engineer-ring properties of the soil.

Particle size determinations on large samples of aggregate are necessary to ensure that aggregates perform as intended for their specified use. A sieve analysis or gradation test determines the distribution of aggregate particles by size within a given sample. Result of sieve: light black gravelly, sandy and silty clay.

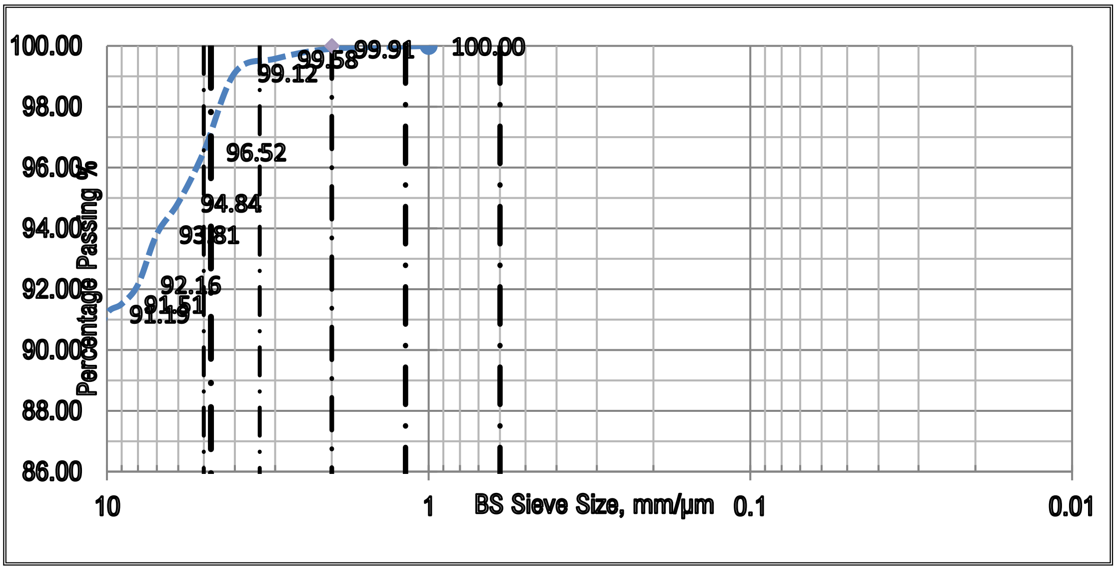

Figure 4-13 Particle Size Distribution Curve for top layer 300mm thick

Refer Appendix 1 (F)

4.3.5 Atterberg Limit Test

Atterberg limits test is a basic measure of critical water contents for fine-grained soil: its shrinkage limit, plastic limit and liquid limit. Depends on its water content, soil may appear in one of four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behaviour of a soil is different and consequently so its engineering properties.

Thus, the boundary between each state can be defined based on a change in the soil's behaviour. Atterberg limits test can be used to distinguish between silt and clay and to distinguish between different types of silts and clays. The water content at which the soils change from one state to the other are known as consistency limits or Atterberg's limit. The result shown for Sample 1, for Plastics limit is 35.73, the Liquid limit: 57.83 and the Plasticity Index is 22.09 %. (Refer Appendix 1 (C))

So, we can name as unsuitable material; shall include running silt, peat, logs, stumps, perishable or toxic material, slurry or mud, or any material consisting of highly organic clay and silt. Which is clay having a liquid limit exceeding 80% and / or a plasticity index exceeding 55%. Which is susceptible to spontaneous ignition. Which has a loss of weight greater than 2.5% on ignition. And containing large amounts of roots, grass and other vegetable matter.

4.3.6 Natural Moisture Content

This method covers the laboratory determination of the moisture content of a soil as a percentage of its oven-dried weight. The method may be applied to fine, medium and coarse-grained soils for particle sizes from 2 mm to >10 mm.

The method is based on removing soil moisture by oven-drying a soil sample until the weight remains constant. The moisture content (%) is calculated from the sample weight before and after drying. Result for natural moisture content value is 65.82%. (Refer Appendix 1 (A) Bottom depth > 300mm thick).

4.3.7 Hydrometer Test – Sedimentation Method

The hydrometer analysis of soil, based on Stokes' law, calculates the size of soil particles from the speed at which they settle out of suspension from a liquid. Results from the test show the grain size distribution for soils finer than the No. 200 (75µm) sieve. However, when combined with a sieve analysis, offer a complete gradation profile of soils containing coarser materials.

This method determines particle size distribution in a soil from the coarse sand size down to clay size (about 2 µm) and measuring the particle size distribution of fine-grained soils like clay and silt is best performed using the soil hydrometer test. This sedimentation method is familiar to all geotechnical laboratories. Result may show in Hydrometer Calibration Curve below. Refer Appendix 1 (D).

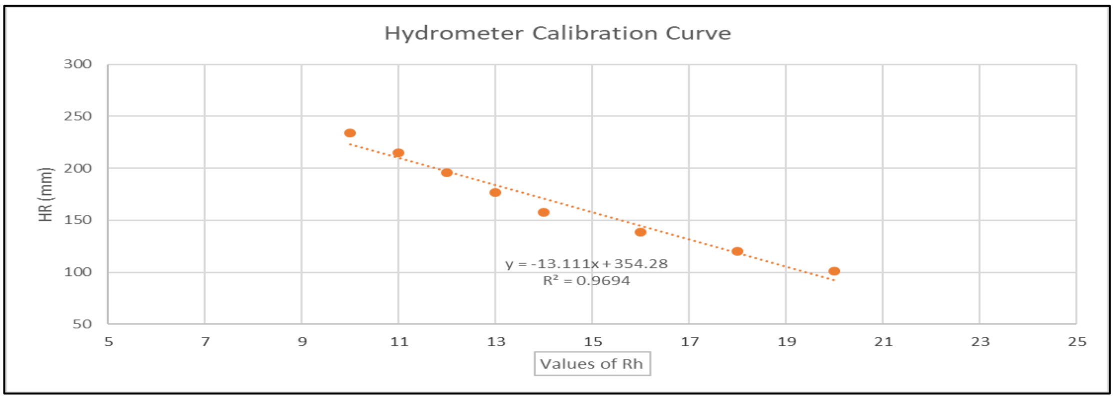

Figure 4-14 Hydrometer Calibration Curve

4.3.8 Unconfined Compression Strength (UCS) Test

The Unconfined Compressive Strength (UCS) is a simple laboratory testing method to assess the mechanical properties of rocks and fine-grained soils. It provides a measure of the undrained strength characteristics of the rock or soil. The unconfined compression test is often included in the laboratory testing program of geotechnical investigations, especially when dealing with rocks.

UCS test for design mix Specification requirement is must more than 0.8 until 5 N/mm² for strength with age a cube curing is 7 days. Requirement for strength is more than 1 until 5 N/mm² for 7 days. An age a sample within 3 and 7 curing condition. The Agent Mix Proportions are using Lime for 3%, 6%, 9% and 5%. Result UCS Test as below. Refer Appendix 1 (I).

Table 4.15
Result UCS Test in N/mm2

Day Age	LIME 3%	LIME 6%	LIME 9%	LIME 12%	SOIL
3	0.60	1.13	2.15	4.25	0.11
7	0.93	2.06	2.73	5.14	0.13

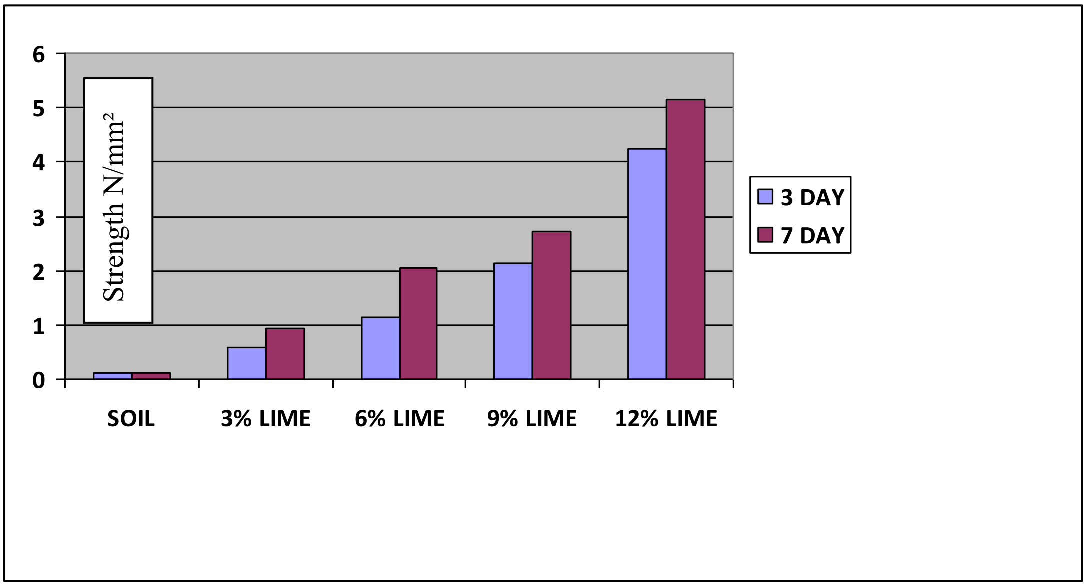

Figure 4-15 The Increased of Strength Prior to Addition of Lime Content.

4.3.9 Lime Demand Test (Eades & Grim)

This process is used to screen the soil for potential for reactivity with lime. The U.S. Air Force Soil Stabilisation Index System (SSIS, 1976) determines a soil to be a candidate for lime stabilisation if the soil has at least 25% passing the 75-micron sieve and has a plasticity index (PI) of at least 10. The screening criteria also limit organics to less than 1% by weight and soluble sulphates to less than 0.3% unless special precautions are taken (Little, 1996).

Approximate Lime Demand: Perform the Eades and Grim pH test (ASTM D 6276) to determine lime demand. This protocol approximates the amount of lime required to provide a long-term pozzolanic reaction that will maximize the probability of providing acceptable long-term strength, resilient properties and durability. Result using pH Test below.

Table 4.16
pH Test Result

Weight Of Lime (%)	2%	3%	4%	5%	6%	SOIL
Reading Of pH 1hr	12.3	12.4	12.6	12.7	12.83	9.5

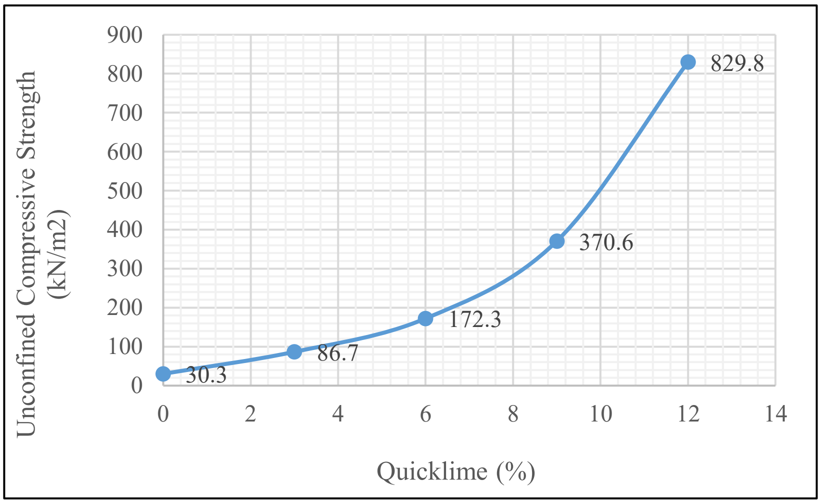

Figure 4-16 Lime Demand Test pH vs Lime Content

Refer Appendix 1 (G)

4.3.10 Particle Density Test/ Specific Gravity

Particle density or specific gravity is a measure of the actual particles which make up the soil mass and is defined as the ratio of the mass of the particles to the mass of the water they displace. A knowledge of the particle density is essential in relation to other soil tests. It is used when calculating porosity and voids ratio and is particularly important when compaction and consolidation properties are being investigated. The particle density is a function of both the specific density of the material and of the void content of the particles. The particle density depends mainly on the void content of the particles. For some aggregates, the particle density is higher for the smaller particles than for the larger ones, because the larger particles are more expanded. The Result shown in an average value ρs (Mg/m3): 2.26. Refer Appendix 1 (E).

4.3.11 Soak 4 Day CBR & Penetration Test

The California bearing ratio (CBR) is a penetration test for evaluation of the mechanical strength of natural ground, subgrades and basecourses beneath new carriageway construction. The CBR rating was developed for measuring the load-bearing capacity of soils used for building roads. The CBR can also be used for measuring the load-bearing capacity of unimproved airstrips or for soils under paved airstrips.

The strength of the subgrade is the main factor in determining the required thickness of flexible pavements for roads and airfields. The strength of a subgrade, subbase and base course materials are expressed in terms of their California Bearing Ratio (CBR) value.

Table 4.17
Result Penetration CBR Test

MARINE CLAY (%)	MARINE CLAY + 3% LIME	MARINE CLAY + 6% LIME
2.41	11.79	16.29

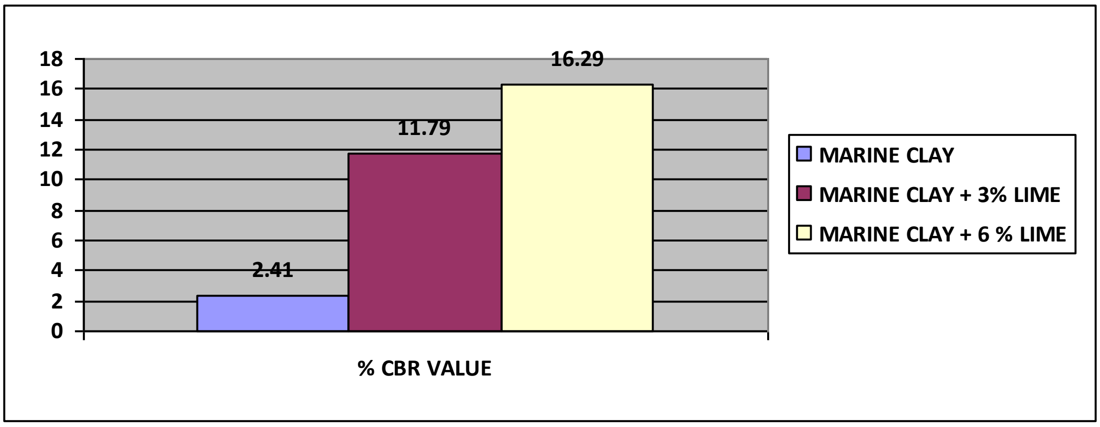

Figure 4-17 The increased CBR value of additional Lime content

4.3.12 Laboratory Test Results and Discussions

Samples of marine clay were collected from a designated site, then subjected to air-drying and sieving to eliminate coarse particles. Subsequently, five soil samples were prepared with varying lime concentrations (0%, 3%, 6%, 9% and 12% by dry weight). Standard laboratory compaction tests were performed to ascertain the maximum dry density (MDD) and optimum moisture content (OMC) for each sample. Data analysis encompassed a comparative study of compaction characteristics among samples and assessing the impact of lime on soil stabilisation.

The results from field density tests and lab compaction tests reveal a notable trend in maximum dry density (MDD) with increasing percentages of lime in marine clay stabilisation. As the percentage of lime increased from 0% to 12%, there was a consistent increase in the maximum dry density of the stabilised soil samples. This positive correlation suggests that lime addition has a significant impact that enhances the soil's compaction characteristics, resulting in higher densities at higher lime concentrations. The observed increase in MDD is attributed to the cementitious reactions between lime and clay particles. These reactions reduce soil plasticity and improve particle bonding. Consequently, the soil structure becomes denser and more compact under the same compaction effort.

Conversely, the optimum moisture content (OMC) showed a decreasing trend as the percentage of lime increased. This reduction in PMC can be explained by lime's ability to trap water within the fine pores of the soil. Additionally, the hydration reaction of the lime removes water through chemical binding and evaporation, further reduce the moisture content required to achieve MDD. This observation suggests that lime facilitates better soil compaction efficiency by lowering the amount of moisture demand.

Higher maximum dry densities achieved with lime addition imply improved load-bearing capacity and making less susceptibility to moisture-induced volume changes. Simultaneously, the decreased optimum moisture content enhances the ease of compaction and reduces construction time and costs associated with soil stabilisation efforts. These advantages are particularly beneficial in geotechnical applications such as road embankments, foundations and other infrastructure developments on weak marine clay subgrades.

This findings regarding the effect of lime addition on marine clay stabilisation diverge from those reported in previous studies. Contrary to the findings of Bell (1996), who observed an increase in optimum moisture content (OMC) and a decrease in maximum dry density (MDD) with increasing lime content, our results indicate a consistent increase in MDD and a decrease in OMC as lime content increases from 0% to 12%. This discrepancy suggests potential variations in soil composition, lime quality, or testing methodologies between studies. The observed increase in MDD can be attributed to improved particle bonding and reduced soil plasticity due to lime-induced cementitious products, facilitating greater compaction efficiency. However, our results challenge the notion that higher lime content necessarily leads to lower MDD and higher OMC. These findings underscore the complex interplay of lime-soil interactions and highlight the need for further research to elucidate the underlying mechanisms governing lime's impact on soil compaction characteristics.

Table 4.18
Values of Compaction Test of Marine Clay Treated with Various Amount of Lime

Amount Lime Added (%)	Optimum Moisture Content (%)	Maximum Dry Density (Mg/m3)	Percentage decrease (%) in OMC	Percentage increase (%) in MDD
0	24.187	1.467		
3	22.198	1.536	8.224	4.707
6	20.111	1.601	16.853	9.090
9	19.101	1.750	21.027	19.282
12	18.452	1.750	23.709	19.282

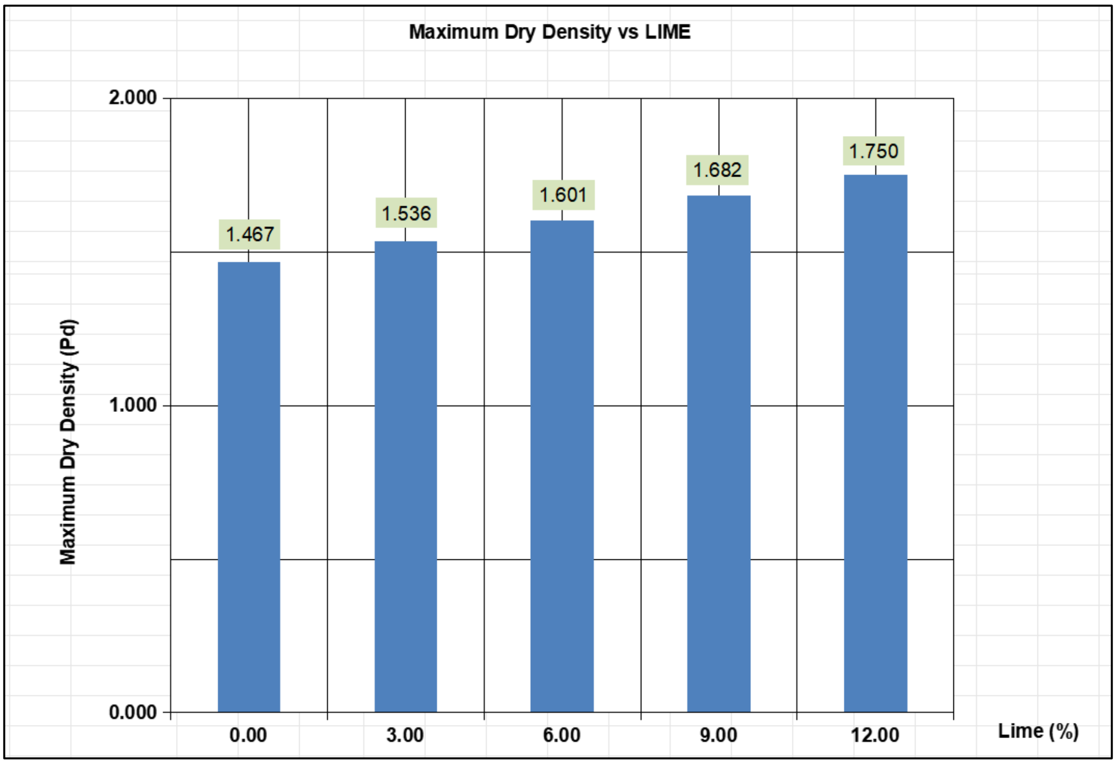

Figure 4-18 Maximum Dry Density of Marine Clay with Various Additions of Lime.

Refer Appendix 1 (L)

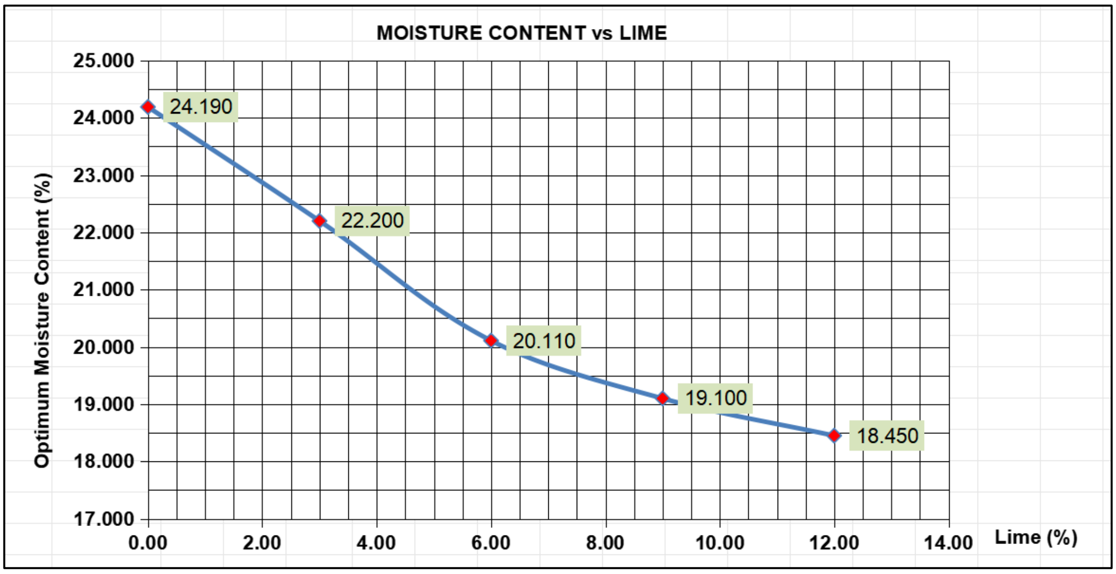

Figure 4-19 Optimum Moisture Content of Marine Clay with Lime.

Refer Appendix 1 (L)

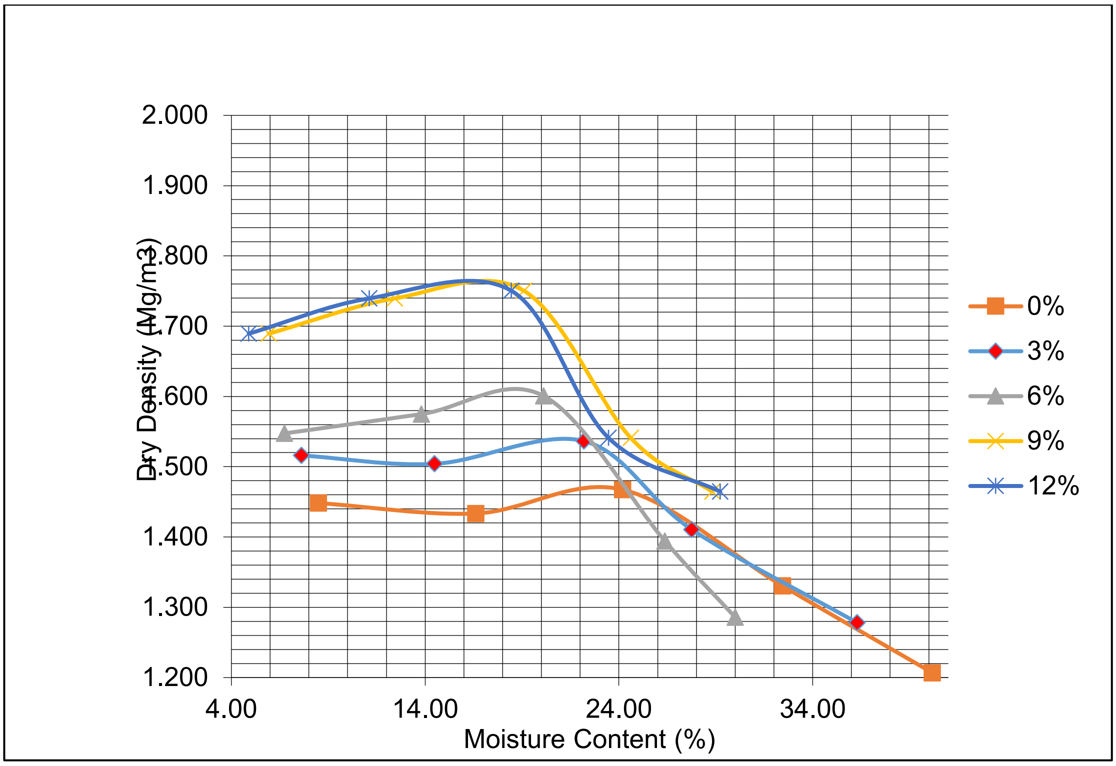

Figure 4-20 Maximum Dry Density of Marine Clay with Various Additions of Lime. Refer Appendix 1 (L)

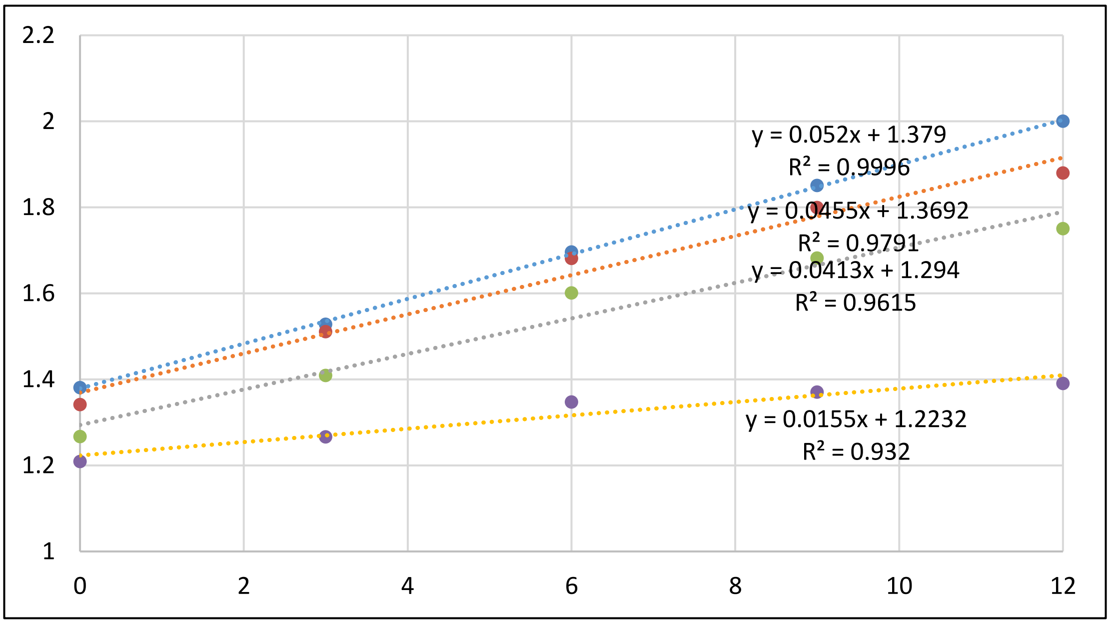

Figure 4-21 Maximum Dry Density Increased Simultaneously with Addition of Lime

4.3.13 Theoretical Results and Finding

Those resources are to identify the binder contents based on the Maximum Dry Density and Plasticity Index which fall into 4 categories starting from PI between 20 %, 20 % to 25 %, 25 % to 30 % and PI more than 30 %. Below table shown to formula be used to identify the lime quantity in percentage (%).

Percentage of Lime, L_p

For PI, 20% use : $P_{dry} = 0.0527L_p + 1.379$

For PI, 20% < PI < 25% use : $P_{dry} = 0.0455L_p + 1.369$

For PI, 25% < PI < 30% use : $P_{dry} = 0.0413L_p + 1.294$

For PI, >30% use : $P_{dry} = 0.0155L_p + 1.294$

Table 4.19
Formula Be Used to Identify the Lime Quantity in Percentage (%).

Criteria PI	Equation	% Confidence
20%	$P_{dry} = 0.0527L_p + 1.379$	99.96%
20% < PI < 25%	$P_{dry} = 0.0455L_p + 1.369$	97.91%
25% < PI < 30%	$P_{dry} = 0.0413L_p + 1.294$	96.15%
>30%	$P_{dry} = 0.0155L_p + 1.294$	93.2%

X-axis represents the percentage of lime content, ranging from 0 to 12. Therefore, Y-axis represents the Moisture Density (MDD), with values ranging from 1.4 to 2.0. The chart shows multiple data points, likely representing different samples or experimental conditions. These data points are connected by lines to visualize the trend. The chart includes four linear regression lines, each with its own equation in the form of y = mx + b. These lines represent the best fit for the data points, allowing us to predict the MDD for a given lime content. The positive slope of the regression lines confirms the direct relationship between MDD and lime content. As the lime content increases, the MDD also increases. This is likely due to the lime's ability to absorb water and fill voids in the soil, leading to increased density. Each regression line has its own equation, where represents the MDD and x represents the lime content. The slope of the line, indicating the rate of change in MDD with respect to lime content. The y-intercept, indicating the MDD value when the lime content is zero. The slope of the regression lines varies slightly, suggesting differences in the rate of MDD increase with lime content under different conditions. The y-intercept also varies, indicating different initial MDD values before lime addition. While not explicitly shown in the chart, an R-squared value would quantify the goodness of fit of the regression lines. A higher R-squared value indicates a better fit, suggesting that the linear model is a good representation of the relationship between MDD and lime content.

This chart is likely relevant to fields like civil engineering or soil mechanics. It could be used to optimize Soil Compaction. Determine the optimal lime content to achieve a desired MDD for soil compaction purposes. To study the effects of lime addition on soil stability and strength and to optimize the lime content in road base materials to achieve desired compaction and strength. To gain a deeper understanding, additional information would be helpful for knowing the type of soil, its initial moisture content and other relevant factors would provide context for the observed behaviour. Understanding the compaction method, curing time and other experimental variables would help interpret the results. Last but not lease, conducting a statistical analysis would provide confidence intervals for the regression lines, allowing for more accurate predictions.

Those resources are to identify the binder contents based on the Maximum Dry Density and Plasticity Index which fall into 4 categories starting from PI less than 10 %, 10 % to 12 %, 12 % to 18 % and PI more than 18 %. Below table shown to formula be used to identify the cement quantity in percentage (%).

In summary, this study demonstrates that lime addition has a significant and positive impact on the compaction characteristics of marine clay. The consistent increase in Maximum Dry Density (MDD) and reduction in Optimum Moisture Content (OMC) observed across lime-treated samples highlight lime's potential as an effective stabilising agent. These findings contribute to a deeper understanding of lime-soil interactions and provide a solid foundation for improving soil stabilisation practices for marine clays in practical applications.

A key outcome of this study is the development of an empirical formula to predict lime content in correlation with MDD, achieving a strong correlation with an R-squared value exceeding 0.9. This formula offers a practical tool for engineers and practitioners to determine the optimal lime content required to achieve maximum compaction efficiency for marine clays under various conditions. However, the study did not extend to developing a similar predictive model for OMC. Future research should focus on addressing this gap by exploring the relationship between lime content and OMC to create a comprehensive suite of predictive tools for soil stabilisation.

4.4 Comprehensive Test Results from Road Project at Sungai Buloh, Selangor.

4.4.1 Introduction

Lateritic soils are highly weathered natural materials characterized by a high concentration of hydrated oxides of iron or aluminium. This composition results from the residual accumulation or absolute enrichment of elements through processes such as the solution, movement and chemical precipitation of aluminium, iron and manganese (Oluyemi-Ayibiowu, B., 2019). These soils are significantly affected by weathering due to the presence of weathered materials enriched with minerals of low solubility, such as iron and aluminium oxides, known as laterite gravel. Due to their mineral composition and physical properties, laterite soils often fail to meet the standards required by road agencies for high-traffic road pavements. In some cases, there are unsuitable even for medium to light traffic applications. This deficiency can be attributed to their particle-size characteristics, the type and strength of gravel particles, the degree of compaction achievable, the volume of traffic, the climatic and hydrological conditions of the construction site and the geographical factors of the area. Due to its swelling nature, lateritic soil presents challenges for engineering projects. It contracts when dry and expands when wet. Leading to issues such as instability and cracking. These properties make lateritic soils a less favourable material for use in road construction and other geotechnical applications.

Various methods are employed to enhance the geotechnical characteristics of laterites to meet the criteria for subbase and base course materials. These methods include preloading, soil replacement, using recycled concrete aggregates and applying soil-stabilising chemicals. Chemical stabilisation is used to enhance the geotechnical qualities of natural soil by adding additives such as lime, cement, fly ash and bitumen to the soil. In this study, cement is used as stabilisation agent.

4.4.2 Methods for Improving Laterite Soils

To address these challenges, various methods have been developed to enhance the geotechnical characteristics of laterites to meet the criteria for subbase and base course materials. These methods include preloading, soil replacement, using recycled concrete aggregates and applying soil-stabilising chemicals. Chemical stabilisation is another widely employed method used to enhance the geotechnical qualities of natural soil by adding additives such as lime, cement, fly ash and bitumen to the soil.

Chemical stabilisation modifies the physical and chemical characteristics of the soil by introducing additives such as lime, cement, fly ash and bitumen. These additives work by interacting with the soil's mineral components, leading to improved strength, reduced plasticity and increased resistance to moisture-induced volume changes. Cement has proven to be an effective stabilising agent for lateritic soils. When mixed with the soil, cement undergoes hydration reactions, forming calcium silicate hydrate (C-S-H) and calcium aluminate hydrate (C-A-H) compounds. These compounds bind the soil particles together, reducing porosity and increasing strength.

In this study, cement was chosen as the stabilisation agent due to its widespread availability, proven performances and cost-effectiveness. The application of cement not only enhances the engineering properties of lateritic soils but also addresses their swelling behaviour, making them suitable for use in a variety of geotechnical and road construction projects.

4.4.3 Results And Discussions

For materials and methodology as explained. Samples of laterite soil were collected from a designated site. To examine the response of soils to varied percentages of additives, the soil samples were mixed with 3%, 6%, 9% and 12% cement contents by weight. Comprehensive standard laboratory compaction tests were conducted to determine both the maximum dry density (MDD) and the optimum moisture content (OMC) for each sample. Following this, compaction tests were performed on the stabilised sample. The results are tabulated below.

Table 4.20
Values of Compaction Test of Laterite Soil Treated with Various Amount of Cement

Amount Cement Added (%)	Optimum Moisture Content (%)	Maximum Dry Density (Mg/m3)	Percentage decrease (%) in OMC	Percentage increase (%) in MDD
0	25.92	1.603		
3	20.38	1.714	21.39	6.948
6	19.58	1.767	24.47	10.281
9	18.47	1.801	28.74	12.370
12	15.30	1.850	40.96	15.457

4.4.4 Result and Observation

i) Maximum Dry Density (MDD)

The findings from field density tests and laboratory compaction tests revealed a notable trend in MDD with increasing cement concentrations. The results of the compaction tests on the samples show that the Maximum Dry Density (MDD) for the soils ranged from 1.603 Mg/m^3 to 1.85 Mg/m^3, demonstrating a significant improvement in compaction characteristic as cement content increased. As the cement content increased, the MDD exhibited a consistent rise, indicating the beneficial impact of cement addition on soil compaction. This observation is in line with Wahab et al (2021), where they found that the maximum dry density of the treated soil improved as the cement concentration increased. Refer Appendix 2 (B).

ii) Optimum Moisture Content (OMC)

While the Optimum Moisture Content (OMC) exhibited a decreasing trend with increasing cement concentrations ranged from 25.92% to 15.3% a representing of reduction of 40.96%. As the lime cementitious content increased from 0% to 12%, there was a consistent increase in the MDD of the stabilised soil samples.

As per Appendix 2 (B), for treated soil at 12% cement content, the OMC decreased 40.96% from its natural value from 25.92% to 15.3%, while the MDD increased 39.93% from 1.74 Mg/m^3 to 6.948 Mg/m^3. This happen due to water been absorbed by cement and may be due to soil sample grading. This observation suggests that cement effectively reduces the required moisture content for achieving maximum density, thereby improving soil compaction efficiency. This indicates that cement addition significantly enhances compaction, leading to higher densities at higher cement concentrations. These findings diverge from those reported in previous studies, where Wahab et al (2021) observed an increase in optimum moisture content (OMC).

4.4.5 Practical Explanation

The findings of this study highlight the significant potential of cement as a stabilising agent for lateritic soils. The observed increase in MDD and reduction in OMC indicate that cement improves the compaction efficiency and load-bearing capacity of the soil, making it suitable for geotechnical applications such as road subbase layers, embankments and foundation beds.

The substantial reduction in moisture requirements also has practical benefits, such as shorter construction times, reduced material handling and cost savings. These advantages make cement stabilisation a cost-effective and efficient solution for enhancing the geotechnical properties of problematic soils.

The divergent results between this study and Wahab et al. (2021) likely stem from differences in experimental conditions, soil characteristics and testing methodologies. In this study, increasing cement concentrations led to a decrease in optimum moisture content (OMC) and a substantial increase in maximum dry density (MDD), indicating improved soil compaction efficiency. Factors such as variations in soil composition, binder types, compaction methods and environmental conditions can significantly influence how cement interacts with soil particles, affecting compaction behaviour differently across studies. These differences underscore the complexity of soil stabilisation processes and highlight the need for context-specific analysis when interpreting and applying research findings in geotechnical engineering and construction practices.

4.4.6 Laboratory Test Report for Design Mix Soil and Cement

Another location selected for this research is located at borrow pit earthwork project for highway infrastructure at Sungai Buloh. Date of Sample on 03rd July 2022 and date of testing on 04th July 2022. Material identified as laterite soil.

The scope of testing including of Proctor Test, Sieve Analysis & hydrometer, Atterberg Limit (Plasticity Index), Moisture content of natural soil, UCS (Unconfined Compression Strength), PH Test, Particle Density test and Soak CBR 4 days penetration test.

4.4.7 Proctor Test

The Proctor compaction test is a laboratory method of experimentally determining the optimal moisture content at which a given soil type will become most dense and achieve its maximum dry density.

The first is a light compaction test using a 2.5 kg rammer (Standard Proctor). The second is a heavy compaction test using a 4.5 kg rammer with a greater drop on thinner layers of soil (Modified Proctor). For both tests a compaction mould of 1 litre internal volume is used for soil in which all particles pass a 20 mm test sieve.

Method using 4.5 kg rammer (BS Heavy) Existing Materials. In this test the comp-active effort is greater than described in the above test. The mass of the rammer is increased to 4.5 kg, the height of the drop to 450 mm and the number of compacted layers will be increased from three to five.

The rammer used is 4.5 kg in weight with a drop of 450 mm. The moist soil shall be compacted in 5 equal layers into the mould. The number of blows applied to each layer shall be 27 blows. Use Vibration rammer machine every layer must vibration 1 minute.

Result Proctor Test that we make for a value MDD Max Dry Density and OMC Optimum. Moisture Content as Sample S1 Laterite Only the MDD is 1.603 Mg/mm^3 and OMC is 25.92%. The same Sample S2 with Laterite added with 3% Cement, the MDD is 1.714 Mg/mm^3, OMC is 20.38 %. For Sample S3 Laterite added 6% Cement, result for MDD is 1.767 Mg/mm³ and OMC is 19.58 %. Sample S4 Laterite added with 9% Cement, result for MDD is 1.801Mg/mm³ and OMC is 18.47 %. And finally Sample S5 for Laterite added with 12% Cement, result shown MDD is 1.850 Mg/mm³ and OMC is 15.30 %. Refer Appendix 2 (B)

4.4.8 Sieve Analysis & Hydrometer

A particle size distribution analysis is a necessary classification test for soils & gradually materials especially coarse soils, in that it presents the relative portions of different sizes of particles. From this, it is possible to determine whether the soil consists of predominantly gravel, sand, silt or clay sizes and, to a limited extent, which of these size-ranges is likely to control the engineering properties of the soil.

Particle size determinations on large samples of aggregate are necessary to ensure that aggregates perform as intended for their specified use. A sieve analysis or gradation test determines the distribution of aggregate particles by size within a given sample. Result of sieve: Brownish gravelly, Sandy & Silty Clay as below. Refer Appendix 2 (E).

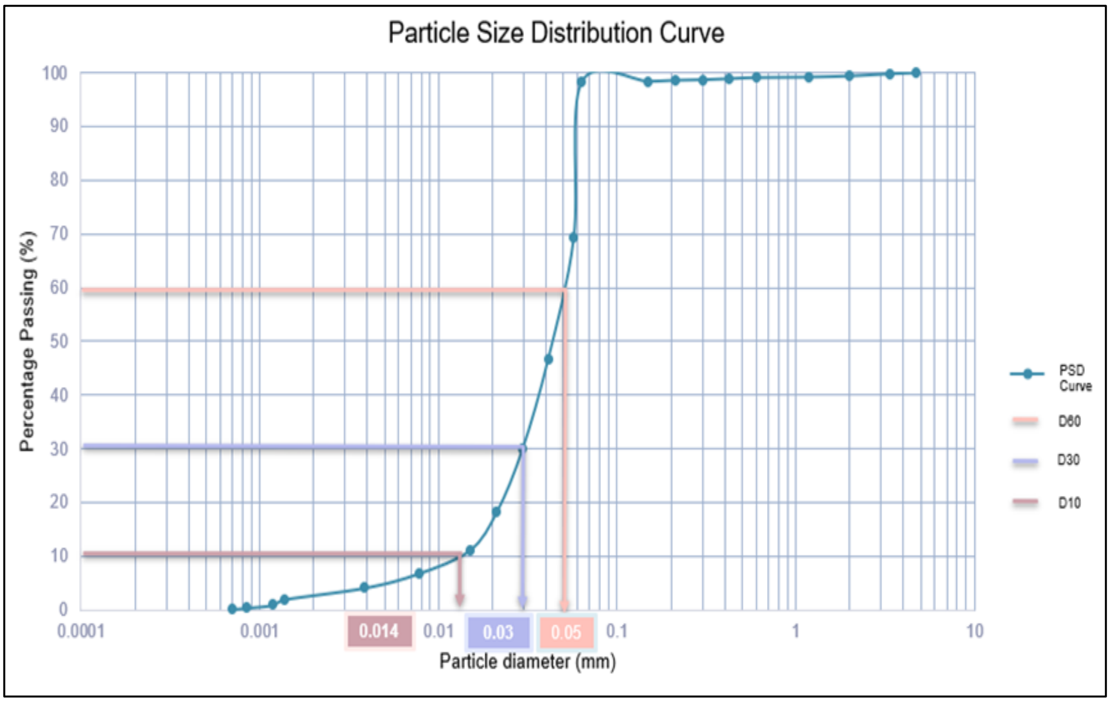

Figure 4-22 Particle Distribution Curve

Gravelly : 0.60%	D60mm : 0.05	Cu : 3.57
Sandy : 1.40%	D30mm : 0.03	Cg : 1.29
Silty : 94.90%	D10mm : 0.014	
Clay : 2.91%		

4.4.9 Atterberg Limit Test

Atterberg limits test is a basic measure of critical water contents for fine-grained soil: its shrinkage limit, plastic limit and liquid limit. Depends on its water content, soil may appear in one of four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behaviour of a soil is different and consequently so its engineering properties.

Thus, the boundary between each state can be defined based on a change in the soil's behaviour. Atterberg limits test can be used to distinguish between silt and clay and to distinguish between different types of silts and clays. The water content at which the soils change from one state to the other are known as consistency limits or Atterberg's limit.

As a result for Plastics limit is 39.96, Liquid limit is 28.74 and Plasticity Index is 11.22 % and categorise as unsuitable material; shall include running silt, peat, logs, stumps, perishable or toxic material, slurry or mud or any material consisting of highly organic clay and silt which is clay having a liquid limit exceeding 80% and / or a plasticity index exceeding 55%, susceptible to spontaneous ignition has a loss of weight greater than 2.5% on ignition and containing large amounts of roots, grass and other vegetable matter. Refer Appendix 2 (C).

4.4.10 Natural Moisture Content

This method covers the laboratory determination of the moisture content of a soil as a percentage of its oven-dried weight. The method may be applied to fine, medium and coarse-grained soils for particle sizes from 2 mm to >10 mm.

Main principle of the method is based on removing soil moisture by oven-drying a soil sample until the weight remains constant. The moisture content (%) is calculated from the sample weight before and after drying. And the result for natural moisture content value of 18.5%. Refer Appendix 2 (A).

4.4.11 Unconfined Compression Strength (UCS) Test

The Unconfined Compressive Strength (UCS) is a simple laboratory testing method to assess the mechanical properties of rocks and fine-grained soils. It provides a measure of the undrained strength characteristics of the rock or soil. The unconfined compression test is often included in the laboratory testing program of geotechnical investigations, especially when dealing with rocks.

For UCS test, design mix Spec requirement is must more than 0.8 until 5 N/mm^2 for strength with age a cube curing is 7 days. The requirement for strength is more than 1 until 5

N/mm² for 7 days. An age of samples within 3 and 7 days curing condition with agent Mix Proportion for Cement are 3%, 6%, 9% and 12%.

Table 4.21
Result UCS Test

Day Age	Cement 3%	Cement 6%	Cement 9%	Cement 12%	SOIL
3	3.52	4.40	6.58	8.15	1.71
7	4.29	5.64	7.39	10.30	2.04

Source: Refer Appendix 2 (H)

4.4.12 Soak 4 Day CBR & Penetration Test

The California bearing ratio (CBR) is a penetration test for evaluation of the mechanical strength of natural ground, subgrades and basecourses beneath new carriageway construction. The CBR rating was developed for measuring the load-bearing capacity of soils used for building roads. The CBR can also be used for measuring the load-bearing capacity of unimproved airstrips or for soils under paved airstrips.

The strength of the subgrade is the main factor in determining the required thickness of flexible pavements for roads and airfields. The strength of a subgrade, subbase and base course materials are expressed in terms of their California Bearing Ratio (CBR) value.

Table 4.22
Result Penetration CBR Test

LATERITE (%)	**LATERITE + 3% Cement**	**LATERITE + 6% Cement**
8.74	14.04	27.28

Source: Appendix 2 (G).

According to the Malaysia Public Work Department, the Unconfined Compression Test (UCS) for 7-day strength with moist curing at 25 degrees Celsius, for unpaved roads, needs more than or equal to 800 kPa following B.S. 1881, Part 116. The cement content was 5 and 6 percent greater than 800 kPa, which was 1140 and 1365 kPa, respectively. However, although having 6% greater strength than 5% of cement, 5% of cement is the most acceptable optimum cement content for stabilising. As a result, the mixing plant must be equivalent to the volume of cement mix for the appropriate material used for subgrade stabilisation. This is because, in practice, the percentage of cement in the mix has a substantial impact on the cost of cement. The compacted material has obtained the 95 percent range density for compacted fill, which is between 1.8 Mg/m3 and 2.1 Mg/m3 and the required UCS is 800 kPa, equivalent to 13 percent CBR. Results showed that all dry densities fall between 1.8 and 2.1 mg/m3. Additionally, the shear strength of laterite soil was 97 kPa, but it rose when mixed with cement. The shear strength was 570 kPa and the axial strain was 1.95 percent with a 5% cement concentration.

Furthermore, this research is carried out to find the effect of soil stabilisation using lime in altering organic material which is clay minerals and non-clay materials in marine clay properties to provide high strength and durability as a subgrade of a pavement. This study is very informative as it gives new knowledge and information on soil treatment using quicklime. Moreover, this study gives an idea to researcher and probably new researcher on the use of lime as a material in soil stabilisation to reduce the variability of thick subgrade beneath the pavement.

Other than that, the output of this study is giving knowledge on the importance to treat soil before proceeding to construct a building and roadways. Soil stabilisation is necessary in soft soil so that they can act as strong foundation before any infrastructure and commuting ways can be built. This is to ensure no fatality and economy loss happened due to damage on the construction that have been built. This study may help any public and private sector which are involved in developing new infrastructure at coastal areas. Therefore, this study concentrated on building a strong foundation using marine clay as a subgrade of pavement. Thus, the output of this research can influence and convince people to use natural resources towards reprocessing materials for soil development to create innovative and cost-effective solutions for the infrastructure demands of an expanding population.

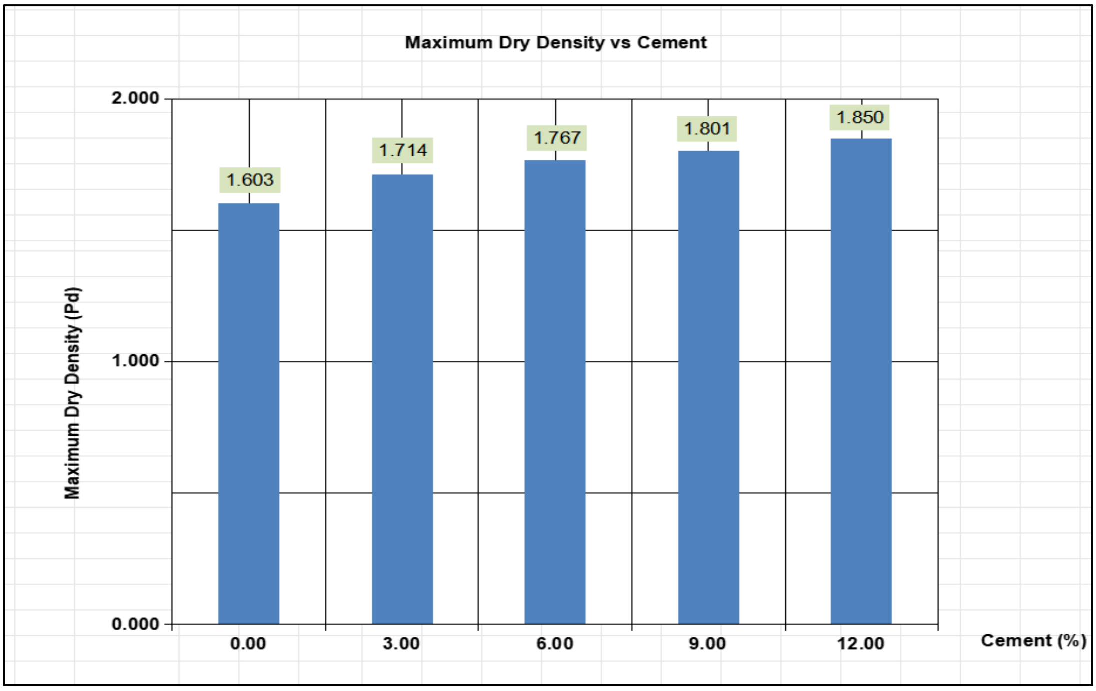

Figure 4-23 Maximum Dry Density of Laterite with Various Additions of Cement

Refer Appendix 2 (K)

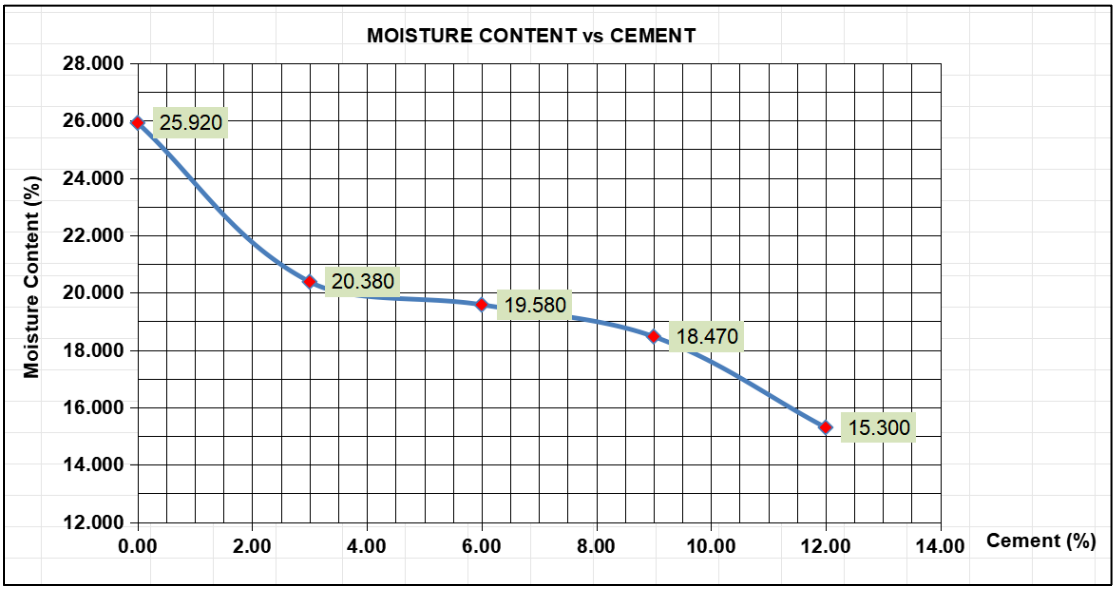

Figure 4-24 Optimum Moisture Content of Laterite with Various Additions of Cement

Refer Appendix 2 (K)

4.4.13 Maximum Dry Density vs Cement Content

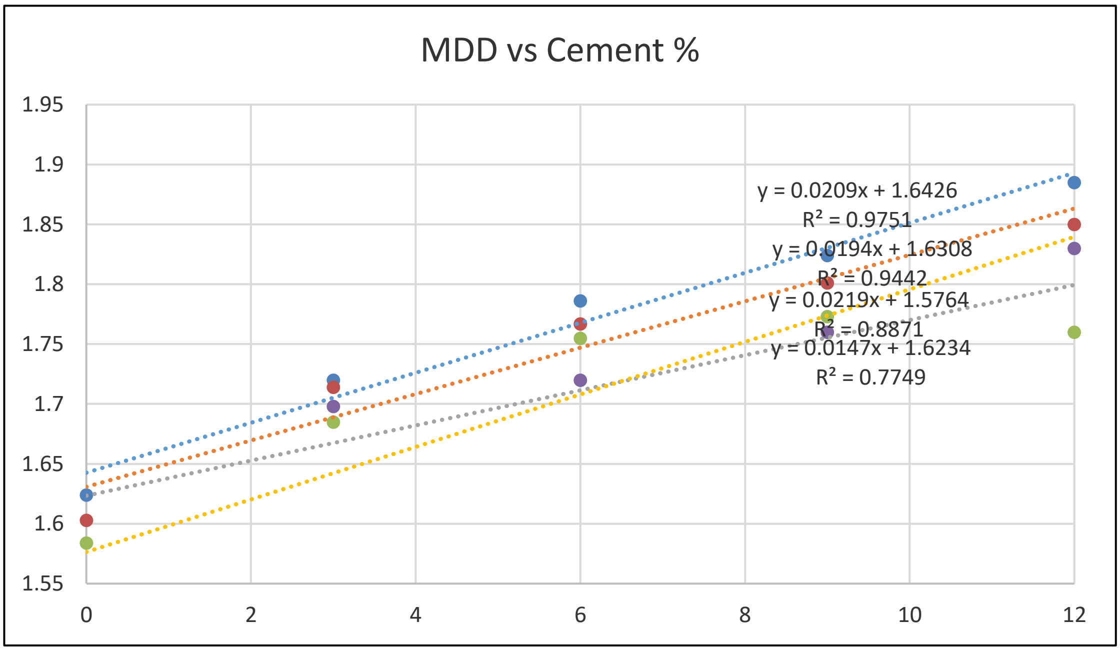

Figure 4-25 Maximum Dry Density Increased Stimulus with Addition of Cement

Refer Appendix 2 (L)

4.4.14 Theoretical Results and Finding

Those resources are to identify the binder contents based on the Maximum Dry Density and Plasticity Index which fall into 4 categories starting from PI less than 10 %, 10 % to 12 %, 12 % to 18 % and PI more than 18 %. Below table shown to formula be used to identify the cement quantity in percentage (%). Refer Appendix 2 (L).

Percentage of Cement, C_p

For PI, 10 % use : $P_{dry} = 0.0209C_p + 1.6426$

For PI, 10 % < PI < 12 % use : $P_{dry} = 0.0219C_p + 1.5764$

For PI, 12 % < PI < 18 % use : $P_{dry} = 0.0194C_p + 1.6308$

For PI, >18 % use : $P_{dry} = 0.0147C_p + 1.6234$

Table 4.23
Formula Be Used to Identify the Cement Quantity in Percentage (%)

Criteria Plasticity Index (PI)	Equation	% Confidence
10%	$P_{dry} = 0.0209C_p + 1.6426$	97.51%
10% < PI < 12%	$P_{dry} = 0.0194C_p + 1.6308$	88.71%
12% < PI < 18%	$P_{dry} = 0.0219C_p + 1.5764$	94.42%
>18%	$P_{dry} = 0.0147C_p + 1.6234$	77.49%

Those resources are to identify the binder contents based on the Maximum Dry Density and Plasticity Index which fall into 4 categories starting from PI less than 10 %, 10 % to 12 %, 12 % to 18 % and PI more than 18 %. Above table shown to formula be used to identify the cement quantity in percentage (%). The provided graph illustrates the relationship between the Maximum Dry Density (MDD) of a material and its cement content. The MDD is a measure of how tightly soil particles can be compacted under standard laboratory conditions. Cement content, on the other hand, is the percentage of cement present in the material. The graph shows a general upward trend, indicating that as the cement content increases, the MDD also tends to increase. This is consistent with the understanding that cement acts as a binding agent, increasing the cohesion and strength of the material, which in turn allows for tighter compaction.

The four linear regression lines overlaid on the graph represent different models fitted to the data. Each line has an equation of the form y = mx + b, where; y is equal to MDD, x is the Cement content and y is an intercept (representing the MDD when the cement content is zero). The different slopes and intercepts of the lines suggest variations in the relationship between MDD and cement content under different conditions or for different materials. In geotechnical engineering, soil compaction is a critical process to increase the soil's strength and stability. The graph helps engineers understand how the addition of cement can improve the compaction characteristics of a soil. The MDD is a key parameter used in the design of concrete mixes. The graph can be used to optimize the cement content in a mix to achieve the desired MDD, which is important for the strength and durability of the concrete.

The graph provides valuable information about the properties of the material being tested. The slope and intercept of the regression lines can be used to characterize the material's response to changes in cement content. Overall, the graph provides valuable insights into the effect of cement content on the MDD of a material. It can be a useful tool for engineers in various fields to make informed decisions about material selection, mix design and construction practices.

4.5 Conclusion

This study provides valuable insights into the relationship between cement content and the geotechnical properties of lateritic soils. By analysing the trends in Maximum Dry Density (MDD) and Optimum Moisture Content (OMC), engineers and practitioners can make informed decisions regarding the use of cement for soil stabilisation in road construction and other infrastructure projects.

In summary, the results demonstrate that cement addition has a significant and positive impact on the compaction characteristics of lateritic soils. The consistent increase in MDD and substantial reduction in OMC observed across cement-treated samples highlight the effectiveness of cement as a stabilising agent. These findings contribute to a deeper understanding of cement-soil interactions and provide a practical framework for improving soil stabilisation practices for lateritic soils in tropical regions.

A key outcome of this study is the identification of a strong correlation between cement content and MDD, with significant improvements observed as cement concentration increased. While this research focused on MDD and OMC, the development of an empirical model correlating cement content with other geotechnical parameters, such as strength and California Bearing Ratio (CBR), would further enhance the practical application of cement stabilisation.

Such a model would enable practitioners to predict and optimize soil behaviour under varying conditions, ensuring efficient and cost-effective stabilisation solutions.

Future research should also focus on addressing variables such as curing duration, environmental factors and soil mineralogy, which can influence the interaction between cement and lateritic soils. Advanced techniques, including machine learning, could be explored to model the complex relationships between cement content and soil properties, providing innovative tools for soil stabilisation practices.

By expanding on these findings and addressing the identified gaps, future developments could significantly enhance the efficiency, reliability and sustainability of lateritic soil stabilisation. This is particularly important in regions like Malaysia, where lateritic soils are prevalent and play a critical role in infrastructure development. The ongoing effort to refine stabilisation methods will contribute to the construction of durable and cost-effective infrastructure, ensuring long-term geotechnical stability in challenging environments.

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 Introduction

Cementitious (Cement/Lime) stabilisation of subgrade materials in Malaysia is growing in acceptance with asset owners and designers recognising the benefits of improving the engineering properties of the untreated soil. Benefits include enhanced working platforms, reduced pavement thickness, reduced overall construction cost and construction time, reduced risk of premature failure from expansive and reactive subgrade materials and the ability to minimise the volume of imported materials.

The process of using in situ stabilisation as an alternative to traditional remove and replace options provides greater benefits in a number of areas, such as; recycling pavement materials, minimising use of quarried products, minimising disposal, reduced construction cost, less construction time, reduced energy use, less trucks on road network which means less fuel & pollution and reduced CO2 emissions by up to 40% over conventional methods (Vorobieff and Murphy, 2003).

It is recommended that local authorities especially JKR and/or KPLB to fully implement and execute design and construction standards suitable for successful implementation of subgrade stabilisation in Malaysia especially for rural road.

5.2 Conclusion

This research is carried out to analyse the effect of sub grade/soil stabilisation using cement in altering existing material which is laterite soil properties to provide high strength and durability as a subgrade of a pavement. This study is very informative as it gives new knowledge and information on soil treatment using cement. This study is to identify the characteristic of the soil, to analysis the strength improvement of the modified laterite with selected binder agent with cement and to identify mix design for soil stabilisation. The testing procedures for the engineering properties of laterite soil including of the moisture content, particle size distribution, Atterberg limit, compaction, pH and UCS was identified. From the UCS test, the laterite soil was 195 kPa, the dry density was 1.94 Mg/m^3 and the axial strain was 11.7%. From the test result for the properties of laterite soil, the soil needs to be stabilised. This is because the PI is greater than 10%, the percentage of fine material is greater than 25%, the MDD below range 1.8 Mg/m3 to 2.1 Mg/m3 and UCS below the specification of Malaysia Public Work Department which need greater than or equal to 800kPa. The increased strength of the modified laterite soil with cement was determined with the UCS with 7 days of curing and with 4 differences percent of cement content which were 3%, 4%, 5% and 6% to modify the laterite soil. From the results of UCS, when the cement content increase, the UCS and shear strength increase too. By obtaining the optimum amount of cement content to stabilise the laterite soil, the UCS shall be done with difference percent of cement. The optimum amount of cement was identified was 5%. This is because the mixing plant must be equivalent to the volume of cement mix for the appropriate material used for subgrade stabilisation.

Other than that, the output of this study is giving knowledge on the importance to treat soil before proceeding to construct a building and roadways. Soil stabilisation is necessary in soft soil so that they can act as strong foundation before any infrastructure and commuting ways can be built. This is to ensure no fatality and economy loss happened due to damage on the construction that have been built. Furthermore, this study concentrated on building a strong foundation for laterite soil as a material for subgrade's pavement. Thus, the result of this research can influence people on how to use natural resources towards reprocessing materials for soil treatment to create innovative and cost-effective solutions for the pavement's infrastructure demands. In this study, cement can be proven as a binder agent for soil stabilisation method that can treat laterite soil to be a reliable base for any pavement's construction.

In this research study, the utilisation of lime as stabilising material in marine clay is investigated and analysed. The physical engineering test was prepared on the subgrade/soil before conducting the mechanical test on the marine clay. Based on the British Soil Classification System (BSCS) and the Plasticity Test, the samples of soil from the marine clay area are classified as clayey SILT with Intermediate Plasticity (CI). The reaction of mixing quicklime to a soil sample at different rates of 3%, 6%, 9% and 12% by weight of sub grade/soil caused the Maximum Dry Density (M.D.D.) increased while the Optimum Moisture Content (O.M.C.) is decreased. This is because the quicklime behaviour that absorb the moisture in soil through pozzolanic activities. With more lime, cations near negatively charged clay surfaces grow. This reaction imbalance in pore water causes osmosis. Charge on the clay surface prevents ion diffusion. Free water molecules diffuse towards the clay surface to equalise charge. In such a system, hydration and osmotic forces offset long-range attractive forces and the overall force becomes repulsive (Mitchell and Soga, 2005). It separates clay particles, creating a scattered structure that allows particles to slide over each other during compaction. Other than that, through this laboratory testing, the strength of modified sub grade/soil is determined. Through a series of U.C.S. test, the strength of soil sample increased with the increasing percentages of lime content. The highest compressive strength obtained is 829.8 kPa when marine clay is added with 12% of quicklime. Soil stabilisation's treatment flattens compaction curves, increased density variation with gradually decreasing of moisture content. With soil treatment, soil density can be reached over a lower down a moisture content. Overall, it can be concluded that quicklime can be used for altering the problematic soil, this can be proved as the characteristic of soil such as strength increase gradually. The untreated soil shows significant changes when added with lime. This shows a big difference between the strength of soil sample before and after being treated.

5.3 Recommendations

The design engineers should fully be aware on proposed for stabilisation methods for existing soil characteristics by providing a concise overview of main outcomes for both existing soils i.e. laterite and marine clay at an optimal binder types and combinations, strength gain especially for UCS and CBR.

The novelties in the thesis have been identified and analysed to calculate the best binders and provide the percentage of the binder as required to ensure meet and achieved the specifications.

Therefore, from the design until operational shall be taken into consideration for sustainable design performance of cementitious binder for construction of stabilised soil or subgrade.

Table 5.1
Simplified Recommendation as Follows.

Type of Soil	Recommendation Process	Requirements
Laterite	Selection of Binder Agent	Cement
	Optimum Binder Content	4-8%
	OMC	+/- 2%
	Curing Time	7 days
	Expected UCS	>1.5MPa
	Expected CBR	>20%
Marine Clay	Selection of Binder Agent	Lime
	Optimum Binder Content	>6%
	OMC	+/- 2%
	Curing Time	7 days
	Expected UCS	>1.5MPa
	Expected CBR	>20%
	Expected Performance	Improvement in shear strength
		Reduction in Plasticity Index (PI)
		Improved Bearing Capacity
Laterite or Marine Clay	Construction Process	
	Site Preparation	Site Cleaning
		Conduct in-situ density/MC
	Mixing	Spread Manual with Spreader
		Mixing with Water Based on machine's weight 250-300mm thick
		Compact Vibro Compactor >95% MDD
		Grading Motor Grader
		Curing Water Tanker
Quality Control	In-situ test	Moisture Content
		MDD
		Binder Content
		CBR
		UCS

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APPENDICES

APPENDIX 1

(LABORATORY TEST FOR MARINE CLAY @ LANDFILL PROJECT)

A.	MOISTURE CONTENT
B.	PROCTOR TEST
	SAMPLES: ORIGINAL Top < 300mm Depth, 3%, 6%, 9% AND 12% LIME)
	SAMPLES: ORIGINAL Bottom >300mm Depth, 3%, 6%, 9% AND 12% LIME)
C.	ATTERBERG LIMIT (Top and Bottom)
D.	HYDROMETER SEDIMETATION TEST (Top and Bottom)
E.	PARTICLE DENSITY TEST
F.	SIEVE ANALYSIS (Top and Bottom)
G.	LIME DEMAND TEST/pH TEST (ORIGINAL, 2%, 3%, 4%, 5% and 6% LIME)
H.	SOAKED CBR (ORIGINAL, 3% and 6% LIME)
I.	UNCONFINED COMPRESSIVE STRENGTH (ORI., 3%, 6%, 9% AND 12%)
J.	DRY DENSITY VS MOINTURE CONTENT
K.	MAXIMUM DRY DENSITY VS % LIME (ORI., 3%, 6%, 9% AND 12%)
L.	MOISTURE CONTENT VS LIME CONTENT (ORI., 3%, 6%, 9% AND 12%)
M.	MAXIMUM DRY DENSITY VS PI (ORI., 3%, 6%, 9% AND 12%) – **NOVELTY**

APPENDIX A

Soil Description : MARINE CLAY - TOP LAYER
Drying Method: HOT PLAT/ SAND BATCH

Date Sampl 9/05/2022
Date Testing 10/05/2022
Date Completed : 10/05/2022

NATURAL MOISTURE CONTENT

MOISTURE CONTENT

Container No.		T1	T2				
Wt. of wet Soil + Container (M2)	gm	142.45	135.00				
Wt. of dry Soil + Container (M3)	gm	110.90	105.58				
Wt. of Container (M1)	gm	33.10	33.05				
Wt. of Moisture (M2-M3)	gm	31.55	29.42				
Wt. of dry Soil (M3-M1)	gm	77.80	72.53				
Moisture Content W =100(M2-M3)/(M3-M1)%		40.55	40.56				
NATURAL MOISTURE CONTENT	%	**40.55**	**40.56**				
AVERAGE		**40.56**					

Soil Description : MARINE CLAY > 300mm thick (BOTTOM LAYER) Drying Method: HOT PLAT/ SAND BATCH				APPENDIX A Date Sample :9/5/2022 Date Testing: 10/5/2022 Date Completed : 10/5/2022		
NATURAL MOISTURE CONTENT						
MOISTURE CONTENT						
Container No.		T4	T5	T6		
Wt. of wet Soil + Container (M2)	gm	48.16	47.83	59.41		
Wt. of dry Soil + Container (M3)	gm	35.88	33.57	41.92		
Wt. of Container (M1)	gm	16.77	13.48	13.81		
Wt. of Moisture (M2-M3)	gm	12.28	14.26	17.49		
Wt. of dry Soil (M3-M1)	gm	19.11	20.09	28.11		
Moisture Content W =100(M2-M3)/(M3-M1)%		64.26	70.98	62.22		
NATURAL MOISTURE CONTENT	%	**64.26**	**70.98**	**62.22**		
AVERAGE			**65.82**			

LABORATORY WORKSHEET

APPENDIX 1 (B)

Description of material : **MARINE CLAY ONLY (> 300 mm thick)**
Source : **PULAU BURUNG LANDFILL, PULAU PINANG**

Date sample : **09/05/2022**
Date test : **11/05/2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number			1	2	3	4	5
Mass of mould + base + compacted specimen (M2)		gm	5630	5730	5880	5820	5750
Mass of mould + base (M1)		gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)		gm	1565	1665	1815	1755	1685
Bulk density =	$\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.571**	**1.672**	**1.822**	**1.762**	**1.692**
Dry density =	$\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.448**	**1.433**	**1.467**	**1.330**	**1.207**
Moisture content container No.			A21	A22	A23	A24	A25
Mass of container + bulk specimen (m1)		gm	65.30	92.10	75.40	54.60	76.30
Mass of container (m2)		gm	14.20	15.60	14.30	14.60	14.20
Mass of container + dried specimen (m3)		gm	61.30	81.20	63.50	44.80	58.50
Moisture content =	$\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**8.49**	**16.62**	**24.19**	**32.45**	**40.18**

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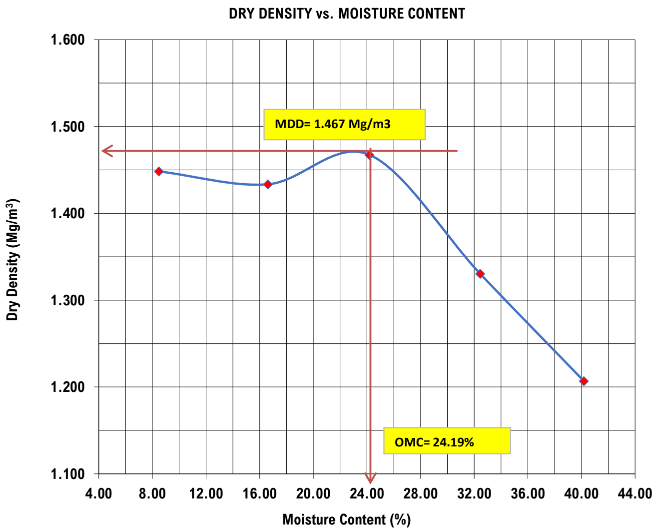


Test Method : 2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.467**	Mg/m^3
Opt. Moisture Content	**24.19**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (B)

Description of material : **> 300mm thick MARINE CLAY + 3 % LIME**
Source : **PULAU BURUNG LANDFILL, PULAU PINANG**

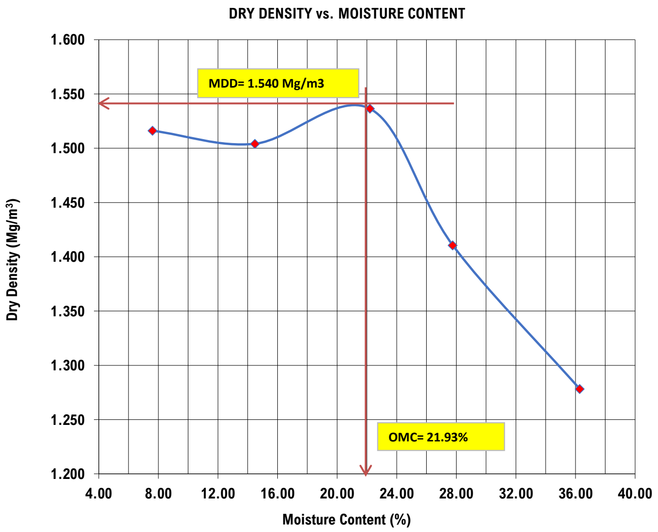
Date sample : **09/05/2022**
Date test : **11/05/2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number			1	2	3	4	5
Mass of mould + base + compacted specimen (M2)		gm	5690	5780	5935	5860	5800
Mass of mould + base (M1)		gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)		gm	1625	1715	1870	1795	1735
Bulk density =	$\frac{M2 - M1}{(3.142 \times d^2 / 4) \times h}$	Mg/m^3	**1.632**	**1.722**	**1.878**	**1.802**	**1.742**
Dry density =	$\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.516**	**1.504**	**1.536**	**1.411**	**1.278**
Moisture content container No.			A11	A12	A13	A14	A15
Mass of container + bulk specimen (m1)		gm	66.30	82.10	70.10	59.30	63.30
Mass of container (m2)		gm	14.00	14.10	14.50	14.20	14.10
Mass of container + dried specimen (m3)		gm	62.60	73.50	60.00	49.50	50.20
Moisture content =	$\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**7.61**	**14.48**	**22.20**	**27.76**	**36.29**

** Delete as appropriate*



Test Method :		
2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.536**	Mg/m^3
Opt. Moisture Content	**22.20**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (B)

Description of material : **> 300mm thick MARINE CLAY + 6% LIME**

Source : **PULAU BURUNG LANDFILL, PULAU PINANG**

Date sample : **09/05/2022**

Date test : **11/05/2022**

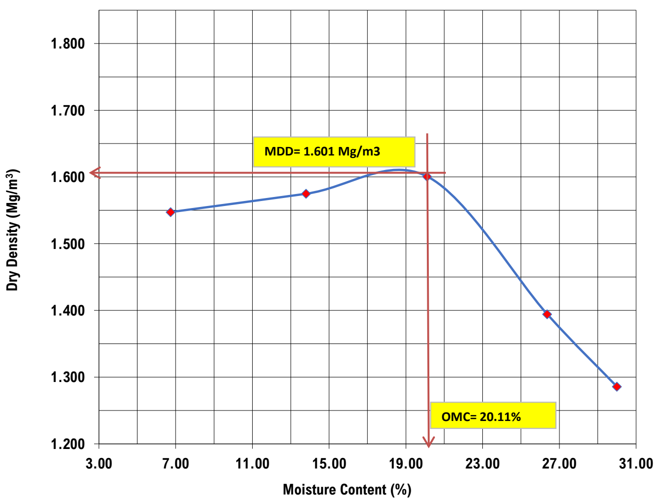
PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number		1	2	3	4	5
Mass of mould + base + compacted specimen (M2)	gm	5710	5850	5980	5820	5730
Mass of mould + base (M1)	gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)	gm	1645	1785	1915	1755	1665
Bulk density = $\frac{M2 - M1}{(3.142 \times d^2/4) \times h}$	Mg/m^3	**1.652**	**1.792**	**1.923**	**1.762**	**1.672**
Dry density = $\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.547**	**1.575**	**1.601**	**1.394**	**1.286**
Moisture content container No.		A1	A2	A3	A4	A5
Mass of container + bulk specimen (m1)	gm	66.50	84.10	79.10	62.50	68.80
Mass of container (m2)	gm	14.20	14.00	14.00	14.10	14.20
Mass of container + dried specimen (m3)	gm	63.20	75.60	68.20	52.40	56.20
Moisture content = $\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**6.73**	**13.80**	**20.11**	**26.37**	**30.00**

** Delete as appropriate*

DRY DENSITY vs. MOISTURE CONTENT



Test Method :		
2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.601**	Mg/m^3
Opt. Moisture Content	**20.11**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX (1 (B)

Description of material : **> 300mm thick MARINE CLAY + 9% LIME**
Source : **PULAU BURUNG LANDFILL, PULAU PINANG**

Date sample : **09/05/2022**
Date test : **11/05/2022**

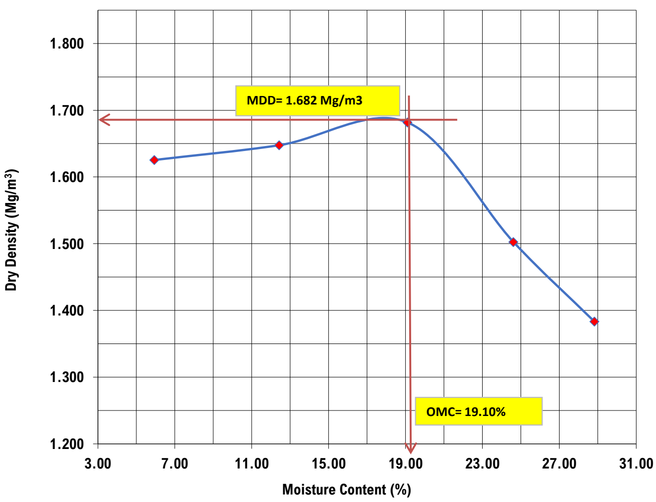
PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number			1	2	3	4	5
Mass of mould + base + compacted specimen (M2)		gm	5780	5910	6060	5930	5840
Mass of mould + base (M1)		gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)		gm	1715	1845	1995	1865	1775
Bulk density =	$\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.722**	**1.852**	**2.003**	**1.872**	**1.782**
Dry density =	$\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.625**	**1.648**	**1.682**	**1.503**	**1.383**
Moisture content container No.			A6	A7	A8	A9	A10
Mass of container + bulk specimen (m1)		gm	76.50	94.50	88.20	78.50	72.60
Mass of container (m2)		gm	14.00	14.00	14.00	14.20	14.05
Mass of container + dried specimen (m3)		gm	73.00	85.60	76.30	65.80	59.50
Moisture content =	$\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**5.93**	**12.43**	**19.10**	**24.61**	**28.82**

** Delete as appropriate*

DRY DENSITY vs. MOISTURE CONTENT



Test Method :		
2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.682**	Mg/m^3
Opt. Moisture Content	**19.10**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (B)

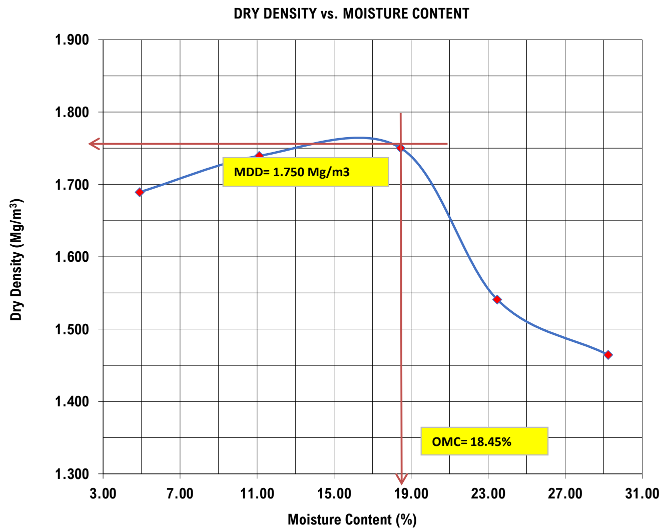
Description of material :	> 300mm thick MARINE CLAY + 12% LIME	Date sample :	09/05/2022
Source :	PULAU BURUNG LANDFILL, PULAU PINANG	Date test :	11/05/2022

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number			1	2	3	4	5
Mass of mould + base + compacted specimen (M2)		gm	5830	5990	6130	5960	5950
Mass of mould + base (M1)		gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)		gm	1765	1925	2065	1895	1885
Bulk density =	$\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.772**	**1.933**	**2.073**	**1.903**	**1.893**
Dry density =	$\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.689**	**1.739**	**1.750**	**1.541**	**1.464**
Moisture content container No.			A16	A17	A18	A19	A20
Mass of container + bulk specimen (m1)		gm	80.50	90.10	93.60	84.60	81.40
Mass of container (m2)		gm	14.10	14.10	14.00	14.10	14.20
Mass of container + dried specimen (m3)		gm	77.40	82.50	81.20	71.20	66.20
Moisture content =	$\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**4.90**	**11.11**	**18.45**	**23.47**	**29.23**

** Delete as appropriate*



Test Method :		
2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.750**	Mg/m^3
Opt. Moisture Content	**18.45**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (B)

Description of material : **MARINE CLAY ONLY (300mm thick - TOP LAYER)**
Source : **PULAU BURUNG LANDFILL, PULAU PINANG**

Date sample : **09/05/2022**
Date test : **11/05/2022**

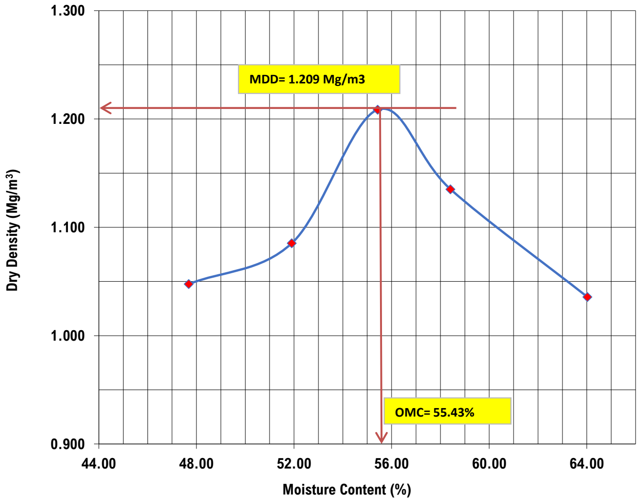
PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number		1	2	3	4	5
Mass of mould + base + compacted specimen (M2)	gm	5606	5707	5936	5856	5757
Mass of mould + base (M1)	gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)	gm	1541	1642	1871	1791	1692
Bulk density = $\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.547**	**1.649**	**1.879**	**1.798**	**1.699**
Dry density = $\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.048**	**1.085**	**1.209**	**1.135**	**1.036**
Moisture content container No.		A21	A22	A23	A24	A25
Mass of container + bulk specimen (m1)	gm	65.30	92.10	75.40	54.60	76.30
Mass of container (m2)	gm	14.20	15.60	14.30	14.60	14.20
Mass of container + dried specimen (m3)	gm	48.80	65.96	53.61	39.85	52.06
Moisture content = $\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**47.69**	**51.91**	**55.43**	**58.42**	**64.03**

** Delete as appropriate*

DRY DENSITY vs. MOISTURE CONTENT



Test Method :		
2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.209**	Mg/m^3
Opt. Moisture Content	**55.43**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (B)

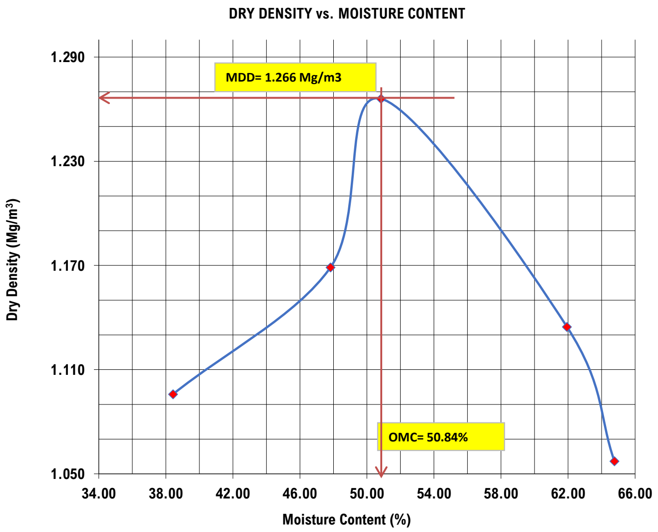
Description of material :	**300mm thick MARINE CLAY + 3 % LIME (TOP LAYER)**	Date sample :	**09/05/2022**
Source :	**PULAU BURUNG LANDFILL, PULAU PINANG**	Date test :	**11/05/2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number		1	2	3	4	5
Mass of mould + base + compacted specimen (M2)	gm	5576	5786	5967	5895	5800
Mass of mould + base (M1)	gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)	gm	1511	1721	1902	1830	1735
Bulk density = $\frac{M2 - M1}{(3.142 \times d^2/4) \times h}$	Mg/m^3	**1.517**	**1.728**	**1.910**	**1.837**	**1.742**
Dry density = $\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.096**	**1.169**	**1.266**	**1.135**	**1.057**
Moisture content container No.		A11	A12	A13	A14	A15
Mass of container + bulk specimen (m1)	gm	66.30	82.10	70.10	59.30	63.30
Mass of container (m2)	gm	14.00	14.10	14.50	14.20	14.10
Mass of container + dried specimen (m3)	gm	51.78	60.10	51.36	42.05	43.96
Moisture content = $\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**38.43**	**47.83**	**50.84**	**61.94**	**64.77**

** Delete as appropriate*



Test Method : 2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.266**	Mg/m^3
Opt. Moisture Content	**50.84**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (B)

Description of material :	**300mm thick MARINE CLAY + 6% LIME (TOP LAYER)**	Date sample :	**09/05/2022**
Source :	**PULAU BURUNG LANDFILL, PULAU PINANG**	Date test :	**11/05/2022**

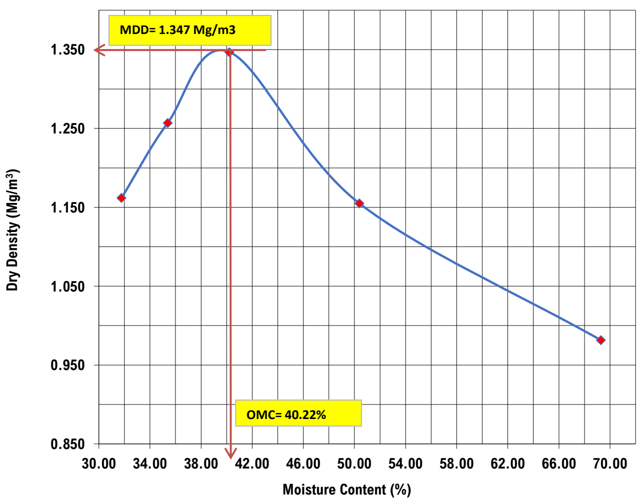
PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number		1	2	3	4	5
Mass of mould + base + compacted specimen (M2)	gm	5590	5760	5946	5795	5720
Mass of mould + base (M1)	gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)	gm	1525	1695	1881	1730	1655
Bulk density = $\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.531**	**1.702**	**1.889**	**1.737**	**1.662**
Dry density = $\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.162**	**1.257**	**1.347**	**1.155**	**0.982**
Moisture content container No.		A1	A2	A3	A4	A5
Mass of container + bulk specimen (m1)	gm	78.77	97.40	90.00	71.70	85.30
Mass of container (m2)	gm	14.20	14.00	14.00	14.10	14.20
Mass of container + dried specimen (m3)	gm	63.20	75.60	68.20	52.40	56.20
Moisture content = $\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**31.78**	**35.39**	**40.22**	**50.39**	**69.29**

** Delete as appropriate*

DRY DENSITY vs. MOISTURE CONTENT



Test Method :		
2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.347**	Mg/m^3
Opt. Moisture Content	**40.22**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (B)

Description of material : **300mm thick MARINE CLAY + 9% LIME (TOP LAYER)**
Source : **PULAU BURUNG LANDFILL, PULAU PINANG**

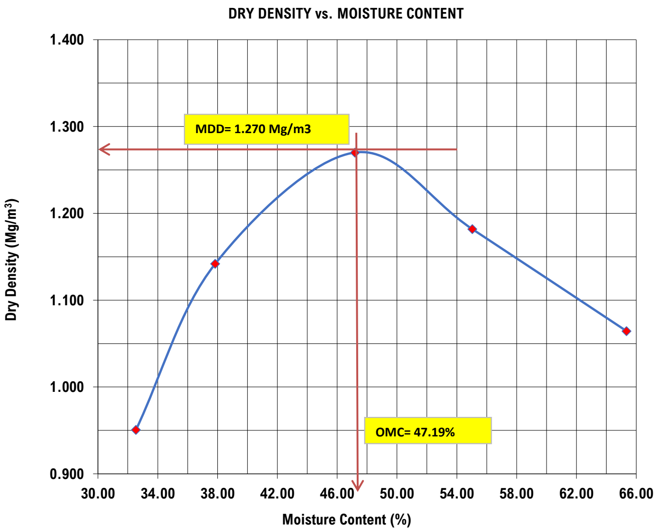
Date sample : **09/05/2022**
Date test : **11/05/2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number			1	2	3	4	5
Mass of mould + base + compacted specimen (M2)		gm	5320	5633	5927	5890	5818
Mass of mould + base (M1)		gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)		gm	1255	1568	1862	1825	1753
Bulk density =	$\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.260**	**1.574**	**1.869**	**1.832**	**1.760**
Dry density =	$\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**0.951**	**1.142**	**1.270**	**1.182**	**1.064**
Moisture content container No.			A6	A7	A8	A9	A10
Mass of container + bulk specimen (m1)		gm	92.20	112.70	105.70	94.20	89.20
Mass of container (m2)		gm	14.00	14.00	14.00	14.20	14.05
Mass of container + dried specimen (m3)		gm	73.00	85.60	76.30	65.80	59.50
Moisture content =	$\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**32.54**	**37.85**	**47.19**	**55.04**	**65.35**

** Delete as appropriate*



Test Method :		
2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.270**	Mg/m^3
Opt. Moisture Content	**47.19**	(%)

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (B)

Description of material : **300mm thick MARINE CLAY + 12% LIME (TOP LAYER)**
Source : **PULAU BURUNG LANDFILL, PULAU PINANG**

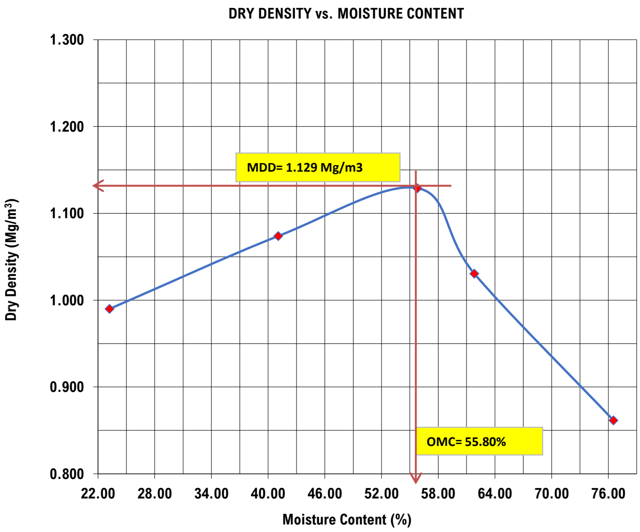
Date sample : **09/05/2022**
Date test : **11/05/2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number			1	2	3	4	5
Mass of mould + base + compacted specimen (M2)		gm	5280	5574	5817	5726	5580
Mass of mould + base (M1)		gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)		gm	1215	1509	1752	1661	1515
Bulk density =	$\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.220**	**1.515**	**1.759**	**1.668**	**1.521**
Dry density =	$\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**0.990**	**1.074**	**1.129**	**1.031**	**0.862**
Moisture content container No.			A16	A17	A18	A19	A20
Mass of container + bulk specimen (m1)		gm	92.10	110.60	118.70	106.50	106.00
Mass of container (m2)		gm	14.10	14.10	14.00	14.10	14.20
Mass of container + dried specimen (m3)		gm	77.40	82.50	81.20	71.20	66.20
Moisture content =	$\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**23.22**	**41.08**	**55.80**	**61.82**	**76.54**

** Delete as appropriate*



Test Method : 2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.129**	Mg/m^3
Opt. Moisture Content	**55.80**	(%)

Remarks :

APPENDIX 1 (C)

LABORATORY WORKSHEET

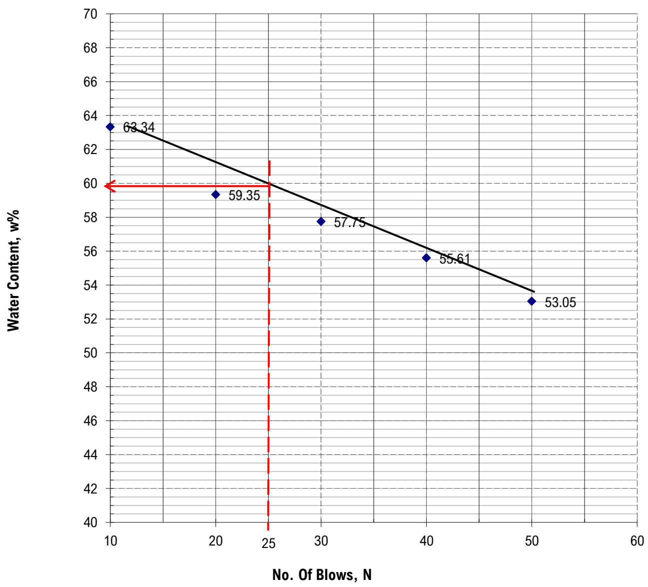
Description of material :	MARINE CLAY 300 mm thick (TOP LAYER)	Date sample :	09/05/2022
Source :	PULAU BURUNG LANDFILL, PULAU PINANG	Date test :	12/05/2022

ATTERBERG LIMIT

(Casagrande Method) / BS 1377: Part 2: 1990: 4.5/4.6* / JKR/SPJ/2008-S4

PLASTIC LIMIT								Average Moisture Content
Test no.		1	2	3	4	5		
Container no.		T4	T5	T7				
Mass of wet soil + container	gm	44.35	37.72	38.91				
Mass of dry soil + container	gm	39.04	34.61	32.53				
Mass of container	gm	23.67	24.67	17.1	no reading	no reading		35.73
Mass of moisture	gm	5.31	3.11	6.38				
Mass of dry soil	gm	15.37	9.94	15.43				
Moisture content	%	**34.55**	**31.29**	**41.35**				

LIQUID LIMIT							Average Moisture Content
Test no.		1	2	3	4	5	
Container no.		A9	A10	A11	A12	A13	
Number of bumps / Blows		50	40	30	20	10	
Mass of wet soil + container	gm	61.42	66.80	67.77	65.20	62.50	
Mass of dry soil + container	gm	46.01	48.95	49.07	47.23	44.74	
Mass of container	gm	16.96	16.85	16.69	16.95	16.70	57.82
Mass of moisture	gm	15.41	17.85	18.70	17.97	17.76	
Mass of dry soil	gm	29.05	32.10	32.38	30.28	28.04	
Moisture content	%	**53.05**	**55.61**	**57.75**	**59.35**	**63.34**	



Sample Preparation*		
As received washed on 425μm sieve		
*Air Dried	-	°C
*Oven Dried	-	°C
Proportion retained on 425μm sieve	-	°C
(*Delete as appropriate)		

RESULTS	(%)
LIQUID LIMIT	57.82
PLASTIC LIMIT	35.73
PLASTICITY INDEX	**22.09**

Remarks : Method : Casagrande / ~~Penetration Cone~~

APPENDIX 1 (C)

LABORATORY WORKSHEET

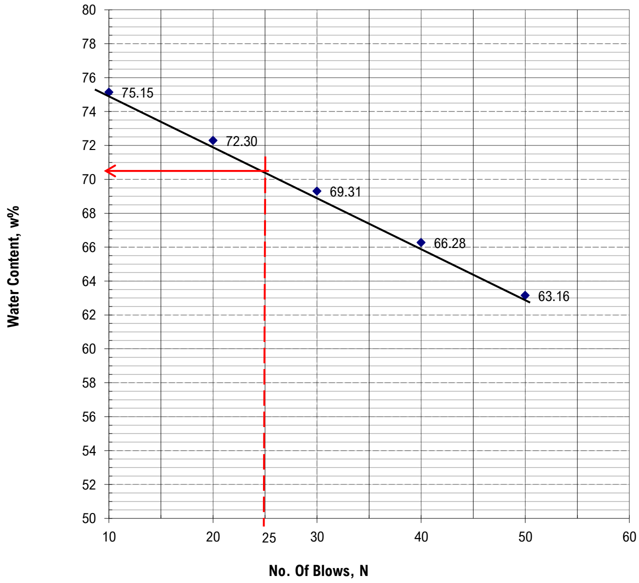
Description of material :	**MARINE CLAY > 300mm thick BOTTOM LAYER**	Date sample :	**09/05/2022**
Source :	**PULAU BURUNG LANDFILL, PULAU PINANG**	Date test :	**12/05/2022**

ATTERBERG LIMIT

(Casagrande Method) / BS 1377: Part 2: 1990: 4.5/4.6* / JKR/SPJ/2008-S4

PLASTIC LIMIT			1	2	3	4	5	Average Moisture Content
	Test no.		1	2	3	4	5	
	Container no.		T4	T5	T7	T3	T8	
	Mass of wet soil + container	gm	55.60	57.60	58.60	64.20	75.20	
	Mass of dry soil + container	gm	51.30	51.90	52.10	55.60	62.85	
	Mass of container	gm	33.60	32.50	33.10	32.80	32.95	**33.38**
	Mass of moisture	gm	4.30	5.70	6.50	8.60	12.35	
	Mass of dry soil	gm	17.70	19.40	19.00	22.80	29.90	
	Moisture content	%	**24.29**	**29.38**	**34.21**	**37.72**	**41.30**	

LIQUID LIMIT								Average Moisture Content
	Test no.		1	2	3	4	5	
	Container no.		A9	A10	A11	A12	A13	
	Number of bumps / Blows		50	40	30	20	10	
	Mass of wet soil + container	gm	48.60	47.50	58.60	65.30	62.50	
	Mass of dry soil + container	gm	42.60	41.80	48.10	51.60	49.80	
	Mass of container	gm	33.10	33.20	32.95	32.65	32.90	**69.24**
	Mass of moisture	gm	6.00	5.70	10.50	13.70	12.70	
	Mass of dry soil	gm	9.50	8.60	15.15	18.95	16.90	
	Moisture content	%	**63.16**	**66.28**	**69.31**	**72.30**	**75.15**	



Sample Preparation*		
As received washed on 425μm sieve		
*Air Dried	-	°C
*Oven Dried	-	°C
Proportion retained on 425μm sieve	-	°C
(*Delete as appropriate)		

RESULTS	**(%)**
LIQUID LIMIT	69.24
PLASTIC LIMIT	33.38
PLASTICITY INDEX	**35.86**

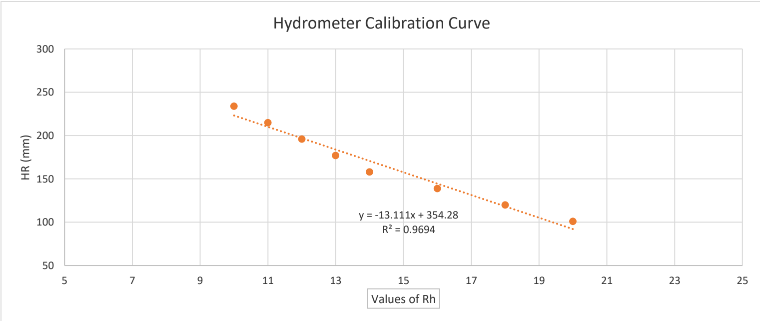
Remarks : Method : Casagrande / ~~Penetration Cone~~

HYDROMETER SEDIMETATION TEST

APPENDIX 1 (D)

Location/Source	PULAU BURUNG LANDFILL, PULAU PINANG / MARINE CLAY 300mm thick BOTTOM	Sample no :	S1
		Date sample :	09.-5.2022
		Date test :	14.05.2022

Scale mark g/cm^3	Reading R_h	Distance from lowest mark d mm	$H = d + N$ mm	H_R mm
1.02	20	0	30	101
1.018	18	19	49	120
1.016	16	38	68	139
1.014	14	57	87	158
1.013	13	76	106	177
1.012	12	95	125	196
1.011	11	114	144	215
1.01	10	133	163	234
0.9985	-1.5	152	182	253

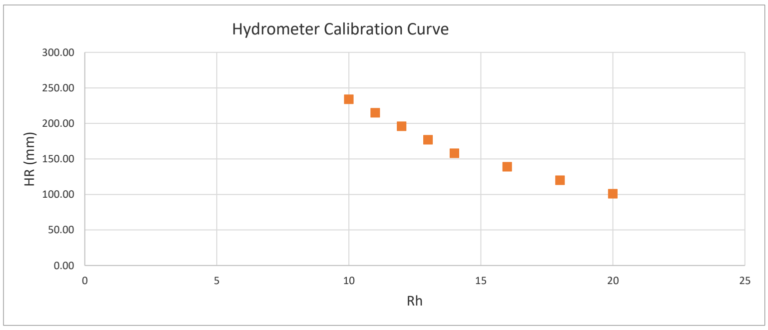


HYDROMETER SEDIMETATION TEST

APPENDIX 1 (D)

Location/Source	PULAU BURUNG LANDFILL, PULAU PINANG / MARINE CLAY > 300mm BOTTOP	Sample no :	S2
		Date sample :	09.-5.2022
		Date test :	14.05.2022

Scale mark g/cm^3	Reading R_h	Distance from lowest mark d mm	$H = d + N$ mm	H_R mm
1.015	20	0	30	100.94
1.015	18	19	49	119.94
1.015	16	38	68	138.94
1.014	14	57	87	157.94
1.013	13	76	106	176.94
1.012	12	95	125	195.94
1.011	11	114	144	214.94
1.01	10	133	163	233.94



PARTICLE DENSITY TEST

APPENDIX 1 (E)

Location/Source	PULAU BURUNG LANDFILL, PULAU PINANG / MARINE CLAY	Sample no :	S1
		Date sample :	09.05.2022
		Date test :	14.05.2022

Sample no.	1	2	3
Mass of bottle, m_1 (g)	42.28	48.06	38.7
Mass of bottle + soil, m_2 (g)	62.59	67.9	58.63
Mass of bottle + soil + water, m_3 (g)	152.76	161.15	149.5
Mass of bottle full of water, m_4 (g)	141.07	150.64	138.25
Particle density, ρ_s (Mg/m^3)	2.356	2.126	2.296
Average value ρ_s (Mg/m^3)	**2.260**		

LABORATORY WORKSHEET

Description of material :	**MARINE CLAY 300mm thick TOP LAYER**	Date sample :	**09/05/2022**
Source :	**PULAU BURUNG LANDFILL, PULAU PINANG**	Date test :	**14.05.2022**
Sample preparation :	air dried **/oven dried/** others *	Page :	**1 of 2**

SIEVE ANALYSIS

JKR /SPJ/2008 - S4

B.S. Sieve (mm)	Weight Retained (g)	Retained (%)	Passing (%)	MS 30: PART 4: 1995 BS 1377 Part 2 :1990 : 9.2 / 9.3 / 9.4 / 9.6 / 9.7*					
75.00									
63.00			100						
50.00									
37.50									
28.00									
25.00									
20.00	0.00	0.00	**100.00**						
14.00									
10.00	0.00	0.00	**100.00**						
6.30									
5.00	0.00	0.00	**100.00**						
4.75	0.94	0.09	**99.91**						
3.35	3.29	0.33	**99.58**						
2.00	4.56	0.46	**99.12**						
1.18	6.52	0.65	**98.47**						
(μm)									
0.600	19.50	1.95	**96.52**						
0.425	16.76	1.68	**94.84**						
0.300	10.36	1.04	**93.81**						
0.212	16.50	1.65	**92.16**						
0.150	6.45	0.65	**91.51**						
0.063	3.20	0.32	**91.19**						
Pan (Silt&Clay)-1	11.9	1.19	**90.00**						
Total (g)	**100.0**								

Weight of sample before oven	(g)	**1000.0**
Weight of sample after oven	(g)	**100.0**
Silt & Clay after wet sieving (g)		**900.0**

Remarks :

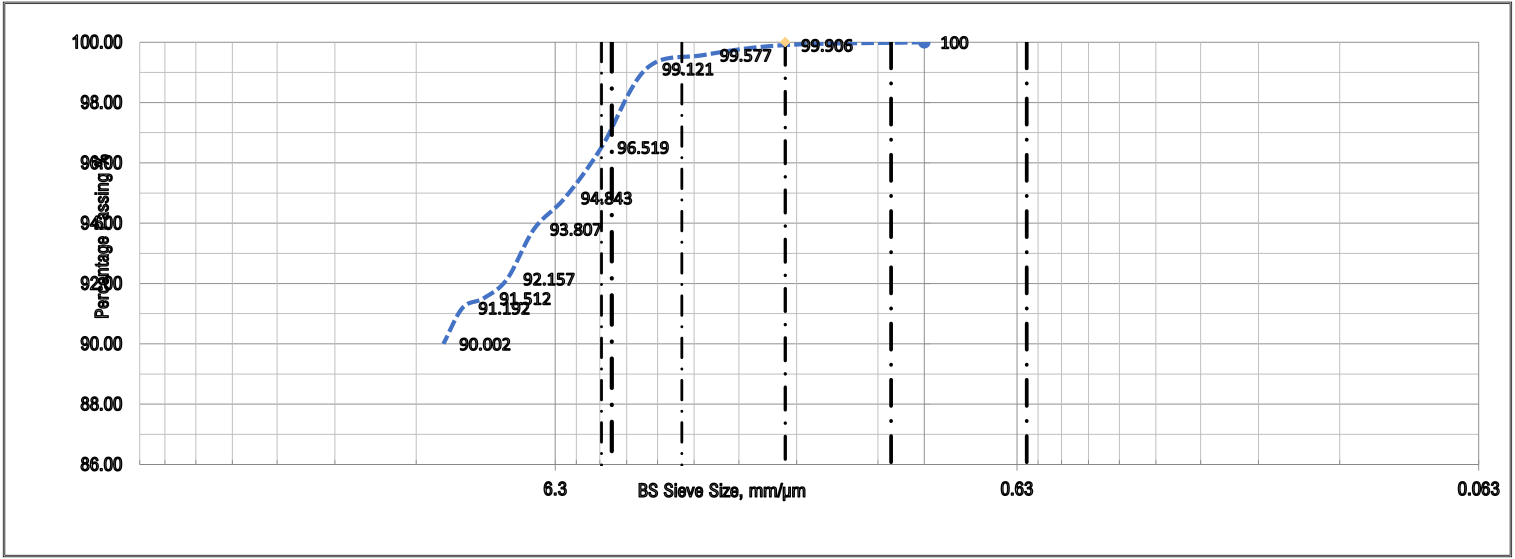
LABORATORY WORKSHEET

APPENDIX 1 (F)

SOIL	**PULAU BURUNG LANDFILL, PULAU PINANG / MARINE CLAY (300mm thick TOP LAYER)**	Date sample :	**09/05/2022**
Source :	**Brownish gravelly, Silty Clay & SANDY**	Date test :	**14.05.2022**
Sample preparation :	air dried /**oven dried**/ others *	Page :	**2 of 2**

SIEVE ANALYSIS - DRY SIEVING (CHART)

JKR/SPJ/2008-S4



Remarks :

Gravelly	0.00	%
Sandy	100.00	%
Silty Clay	0.00	%

LABORATORY WORKSHEET

APPENDIX 1 (F)

Description of material :	**> 300mm MARINE CLAY + LIME (Bottom Layer)**	Date sample :	**09/05/2022**
Source :	**PULAU BURUNG LANDFILL, PULAU PINANG**	Date test :	**14.05.2022**
Sample preparation :	air dried /**oven dried**/ others *	Page :	**1 of 2**

SIEVE ANALYSIS

JKR

/SPJ/2008 - S4

B.S. Sieve (mm)	Weight Retained (g)	Retained (%)	Passing (%)	MS 30: PART 4: 1995 BS 1377 Part 2 :1990 : 9.2 / 9.3 / 9.4 / 9.6 / 9.7*					
75.00									
63.00			100						
50.00									
37.50									
28.00									
25.00									
20.00									
14.00									
10.00									
5.00	0.00	0.00	**100.00**						
4.75	0.50	0.05	**99.95**						
3.35	1.60	0.16	**99.79**						
2.36									
2.00	3.60	0.36	**99.43**						
1.18	3.80	0.38	**99.05**						
(μm)									
0.600	4.40	0.44	**98.61**						
0.425	5.90	0.59	**98.02**						
0.300	5.20	0.52	**97.50**						
0.212	2.70	0.27	**97.23**						
0.150	1.20	0.12	**97.11**						
0.075									
0.063	1.20	0.12	**97.0**						
Pan (Silt&Clay)-1	17.10	1.71	**95.28**						
Total (g)	**47.2**								
Weight of sample before oven		(g)	**1000.0**						
Weight of sample after oven		(g)	**47.2**						
Silt & Clay after wet sieving (g)			**952.8**						

Remarks :

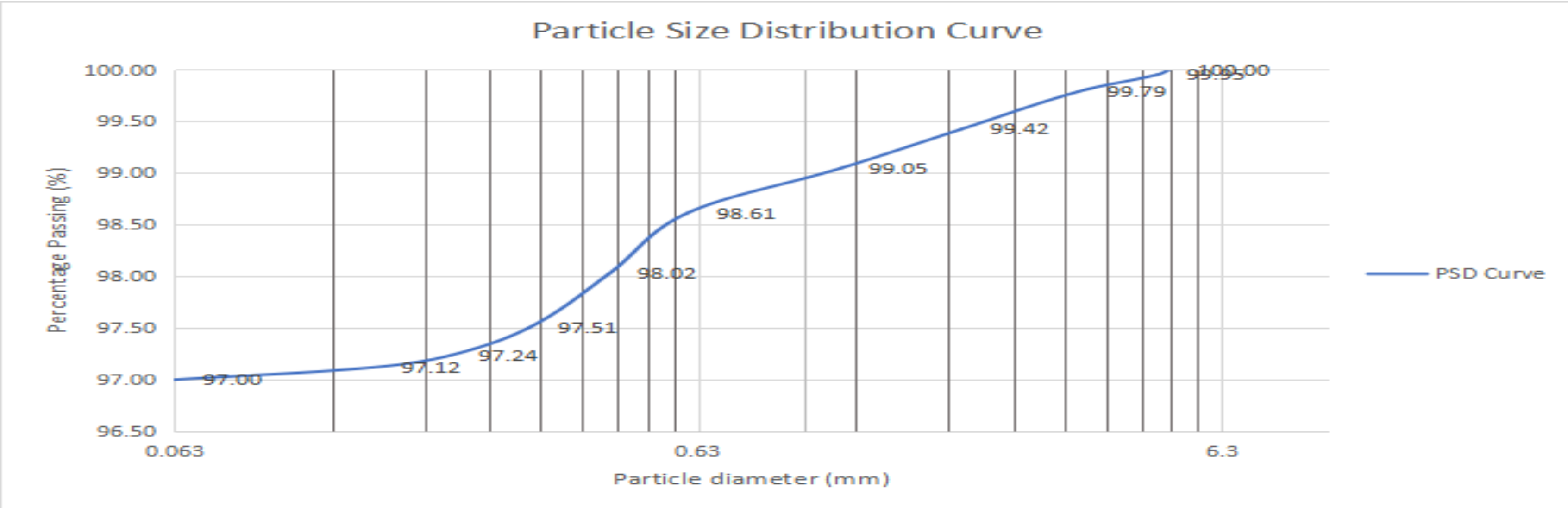
LABORATORY WORKSHEET

APPENDIX 1 (F)

Source :	PULAU BURUNG LANDFILL, PULAU PINANG / MARINE CLAY (>300mm thick - Bottom Layer)	Date sample :	**09/05/2022**
		Date test :	**14.05.2022**
Sample preparation :	air dried /**oven dried**/ others *	Page :	**2 of 2**

SIEVE ANALYSIS - DRY SIEVING (CHART)

JKR/SPJ/2008-S4



Remarks :										
	Gravelly	**0.58**	%	**Clay**	**29.96**	%	**D_{60} (mm)**	0.013	C_u	10.83
	Sandy	**2.42**	%				**D_{30} (mm)**	0.002	C_g	0.34
	Silty	**70.04**	%				**D_{10} (mm)**	0.001		

LIME DEMAND TEST

DISCRIPTION	NO 1	NO 2	NO 3	NO 4	NO 5	NO 6
Weight Of Speciment (gm)	20.00	20.00	20.00	20.00	20.00	20.00
Weight Of Container (gm)	37.00	38.00	38.00	38.00	38.00	38.00
Weight Of Speciment + Container (gm)	57.00	59.00	59.00	58.00	58.00	59.00
Water 100mℓ (gm)	100.0	100.0	100.0	100.0	100.0	100.0
Weight Of Lime (%)	**2%**	**3%**	**4%**	**5%**	**6%**	0%
	4	6	8	10	12	SOIL
Reading Of pH 1hr	**12.3**	**12.4**	**12.6**	**12.5**	**12.5**	**9.5**
Temperature 0 °c	32	32	32.5	32.8	32.9	32

moisture content : 14.79%

Description of material :	MARINE CLAY	Date sample :	09.05.2022
Source:	PULAU BURUNG LANDFILL, PULAU PINANG	Date test :	20.05.2022

APPENDIX 1 (H)

CALIFORNIA BEARING RATIO (CBR) SOAKED 95% MDD

BS 1377 PART 4 / JKR/SPJ/2008 - S4

A. Preparation Procedure :

[ ]	Compresion with tamping
[ ]	Compresion in layers
[ ]	Rammer compaction to a spec. density
[X]	Vibrating compaction to a spec. density
[ ]	Rammer compaction with specified effort

B. Compaction Data :

1.Max Dry Density (MDD)	1.3390	Mg/m^3
2.Optimum Moisture Content	39.81	%
3.Specified Density at 95%	1.2721	Mg/m^3
4.Volume of Mould	2305	cm^3
5.Specified Effort	5	layers
6.Blow per layer By Vibrating Hammer	1	minit
7.Calculated mass of soil required (m) = $\frac{V}{100}$ * (100 + W) pd	4099	g
8.Soaking Time	4	days
9.Final Swelling	9.0	mm
10.Water to be added	1632	gm

Note: V = Vol Of Mould
W = Moisture Content Of Sample
pd = Selected Density Of 95%

Description of material :	**MARINE CLAY**	Date test :	**20.05.2022**
Source:	PULAU BURUNG LANDFILL, PULAU PINANG	Page :	**1 of 3**

SOAKED CBR TEST

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

Density Before Soaking		
Mould no		7
Wt Of mould + wet sample	gm	11205
Wt of mould,	gm	7295
Wt of sample	gm	3910
Volume of mould	cm^3	2305
Wet density	mg/m^3	1.696
Average moisture content	%	32.98
Dry density	mg/m^3	1.3390

Moisture Content (Before Soaked)			
Container No		**X2**	**Z3**
Wt of cont. + wet sample	gm	**114.30**	**102.60**
Wt of cont. + dry sample	gm	**90.30**	**80.80**
Wt. of container	gm	**17.10**	**15.10**
Wt. of water	gm	**24.00**	**21.80**
Wt. of dry sample	gm	**73.20**	**65.70**
Moisture content	%	**32.79**	**33.18**
Av.Moisture content	%	**32.98**	
Proving ring constant		0.02649 (Div) kNf	

Penetration	Dial Gauge		Force		Corrected	
	Top	**Bottom**	**Top**	**Bottom**	Top	Bottom
0.00	0	0	0.00	0.00		
0.50	2	3	0.05	0.08		
1.00	3	4	0.08	0.11		
1.50	5	6	0.13	0.16		
2.00	7	8	0.19	0.21		
2.50	**10**	**12**	**0.26**	**0.32**		
3.00	12	13	0.32	0.34		
3.50	13	15	0.34	0.40		
4.00	13	15	0.34	0.40		
4.50	14	15	0.37	0.40		
5.00	**14**	**16**	**0.37**	**0.42**		
5.50						
6.00						
6.50						
7.00						
7.50						

Expansion & Absorption Determination		
Surcharge	kg	**4.5**
Gauge reading after soaking	Div	**9.00**
Gauge reading before soaking	Div	**0.0000**
Total expansion	mm	**9.0**
Wt. Of mould + sample after soaking	gm	**11630**
Wt. Of mould + sample before soaking	gm	**11205**
Wt. of water absortion	gm	**425.0**

Moisture Content (After Soaked)		**Top**	**Bottom**
Container No		**Z4**	**A43**
Wt of cont. + wet sample	gm	**100.70**	**103.60**
Wt of cont. + dry sample	gm	**79.60**	**79.70**
Wt. of container	gm	**18.60**	**14.80**
Wt. of water	gm	**21.10**	**23.90**
Wt. of dry sample	gm	**61.00**	**64.90**
Moisture content	%	**34.59**	**36.83**

Period of soaking	4 days
Method of campaction	-

5 layers Method Of Compaction : Vabrating Hammer Time/Duratoin 1 min

Remarks :

LABORATORY WORKSHEET

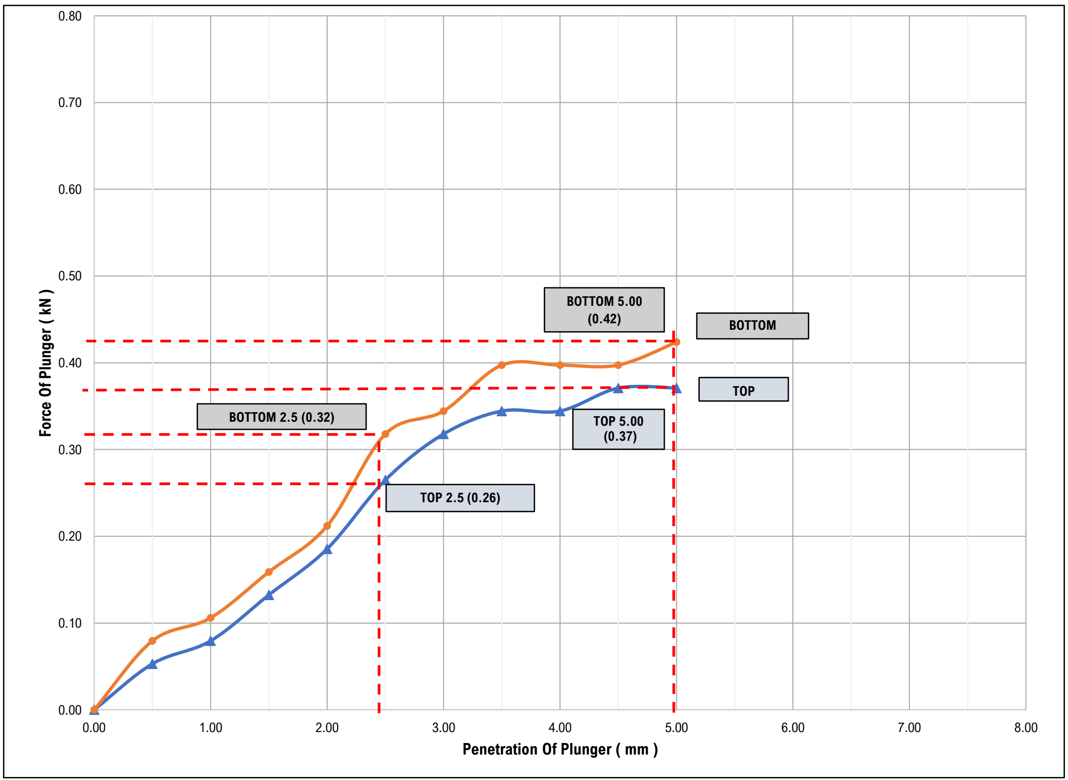
APPENDIX 1 (H)

Description of material : **MARINE CLAY**
Source: **PULAU BURUNG LANDFILL, PULAU PINANG**

Date test : **20.05.2022**
Page : **2 of 3**

SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4



Plunger Penetration (mm)	Top C.B.R (%)	Top M/C (%)
2.50	**2.01**	**34.59**
5.00	**1.85**	

Plunger Penetration (mm)	Bottom C.B.R (%)	Bottom M/C (%)
2.50	**2.41**	**36.83**
5.00	**2.12**	

Remarks : * Specification Limits from JKR's Design Specification is not less than 12%

Highest Value in this material 2.41%

Remarks :

APPENDIX 1 (H)

LABORATORY WORKSHEET

Description of material : **MARINE CLAY**
Source : **PULAU BURUNG LANDFILL, PULAU PINANG**

Date test : **20.05.2022**
Page : **3 of 3**

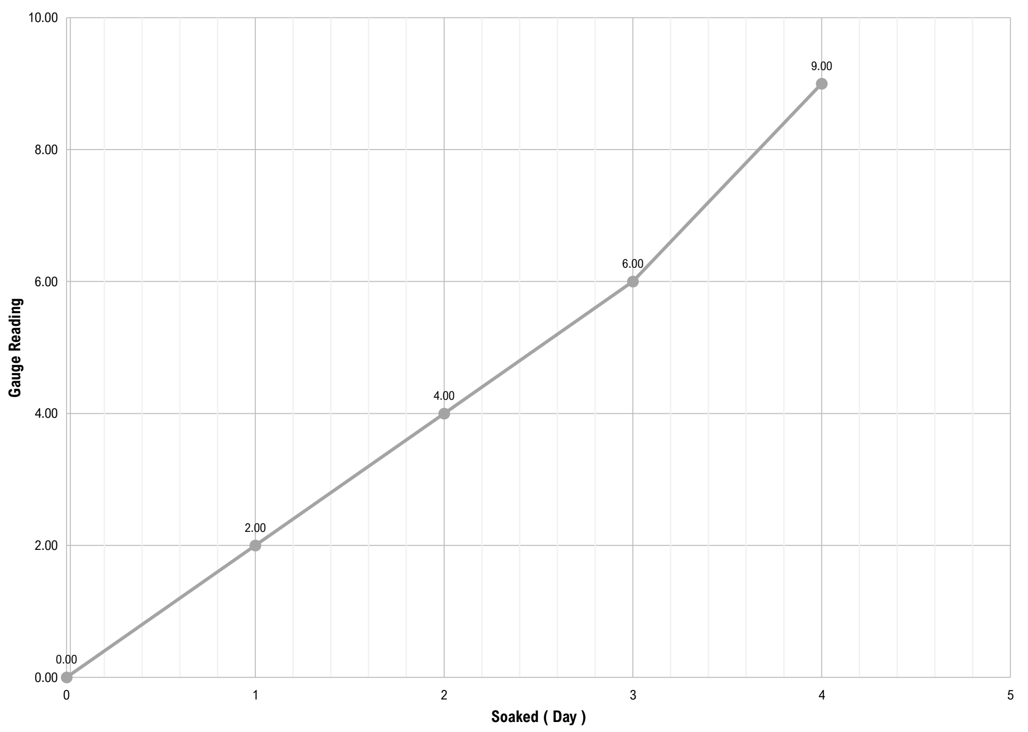
SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

DATE	DAY	TIME	PERIOD		Reading (mm)	REMARKS
20.05.2022	THURSDAY	4.40 PM	0		0.00	FULLY WATER
21.05.2022	FRIDAY	4.40 PM	1		2.00	-
22.05.2022	SATURDAY	4.40 PM	2		4.00	-
23.05.2022	SUNDAY	4.40 PM	3		6.00	-
24.05.2022	MONDAY	4.40 PM	4		9.00	-

Remarks:

C.B.R.(SWELL CHART)



Description of material :	**MARINE CLAY + 3 % LIME**	Date sample :	**09.05.2022**
Sources:	**PULAU BURUNG LANDFILL, PULAU PINANG**	Date test :	**20.05.2022**

APPENDIX 1 (H)

CALIFORNIA BEARING RATIO (CBR) SOAKED 95% MDD

BS 1377 PART 4 /

JKR/SPJ/2008 - S4

A. Preparation Procedure :

	Compresion with tamping
	Compresion in layers
	Rammer compaction to a spec. density
X	Vibrating compaction to a spec. density
	Rammer compaction with specified effort

B. Compaction Data :

1.Max Dry Density (MDD)	1.4010	Mg/m^3
2.Optimum Moisture Content	36.52	%
3.Specified Density at 95%	1.3310	Mg/m^3
4.Volume of Mould	2305	cm^3
5.Specified Effort	5	layers
6.Blow per layer By Vibrating Hammer	1	minit
7.Calculated mass of soil required (m) = $\frac{V * (100 + W)\ pd}{100}$	4188	g
8.Soaking Time	4	days
9.Final Swelling	5.0	mm
10.Water to be added	1530	gm

Note: V = Vol Of Mould
W = Moisture Content Of Sample
pd = Selected Density Of 95%

Description of material :	MARINE CLAY + 3% LIME	Date test :	20/05/2022
SOURCES:	PULAU BURUNG LANDFILL, PULAU PINANG	Page :	1 of 3

APPENDIX 1 (H)

SOAKED CBR TEST

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

Density Before Soaking		
Mould no		6
Wt Of mould + wet sample	gm	11260
Wt of mould,	gm	7295
Wt of sample	gm	3965
Volume of mould	cm^3	2305
Wet density	mg/m^3	1.720
Average moisture content	%	36.16
Dry density	mg/m^3	1.4010

Moisture Content (Before Soaked)			
Container No		**P2**	**P3**
Wt of cont. + wet sample	gm	**122.70**	**112.50**
Wt of cont. + dry sample	gm	**94.60**	**86.90**
Wt. of container	gm	**17.00**	**16.00**
Wt. of water	gm	**28.10**	**25.60**
Wt. of dry sample	gm	**77.60**	**70.90**
Moisture content	%	**36.21**	**36.11**
Av.Moisture content	%	**36.16**	
Proving ring constant		0.02649 (Div) kNf	

Penetration	Dial Gauge		Force		Corrected	
	Top	Bottom	Top	Bottom	Top	Bottom
0.00	0	0	0.00	0.00		
0.50	9	15	0.24	0.40		
1.00	18	25	0.48	0.66		
1.50	26	34	0.69	0.90		
2.00	38	47	1.01	1.25		
2.50	**46**	**59**	**1.22**	**1.56**		
3.00	57	67	1.51	1.77		
3.50	67	77	1.77	2.04		
4.00	70	80	1.85	2.12		
4.50	74	84	1.96	2.23		
5.00	**78**	**89**	**2.07**	**2.36**		
5.50						
6.00						
6.50						
7.00						
7.50						

Expansion & Absorption Determination		
Surcharge	kg	**4.5**
Gauge reading after soaking	Div	**5.00**
Gauge reading before soaking	Div	**0.0000**
Total expansion	mm	**5.0**
Wt. Of mould + sample after soaking	gm	**11310**
Wt. Of mould + sample before soaking	gm	**11260**
Wt. of water absortion	gm	**50.0**

Moisture Content (After Soaked)		**Top**	**Bottom**
Container No		**P4**	**P5**
Wt of cont. + wet sample	gm	**114.90**	**108.20**
Wt of cont. + dry sample	gm	**86.70**	**81.60**
Wt. of container	gm	**16.20**	**16.00**
Wt. of water	gm	**28.20**	**26.60**
Wt. of dry sample	gm	**70.50**	**65.60**
Moisture content	%	**40.00**	**40.55**

Period of soaking	4 days
Method of campaction	-

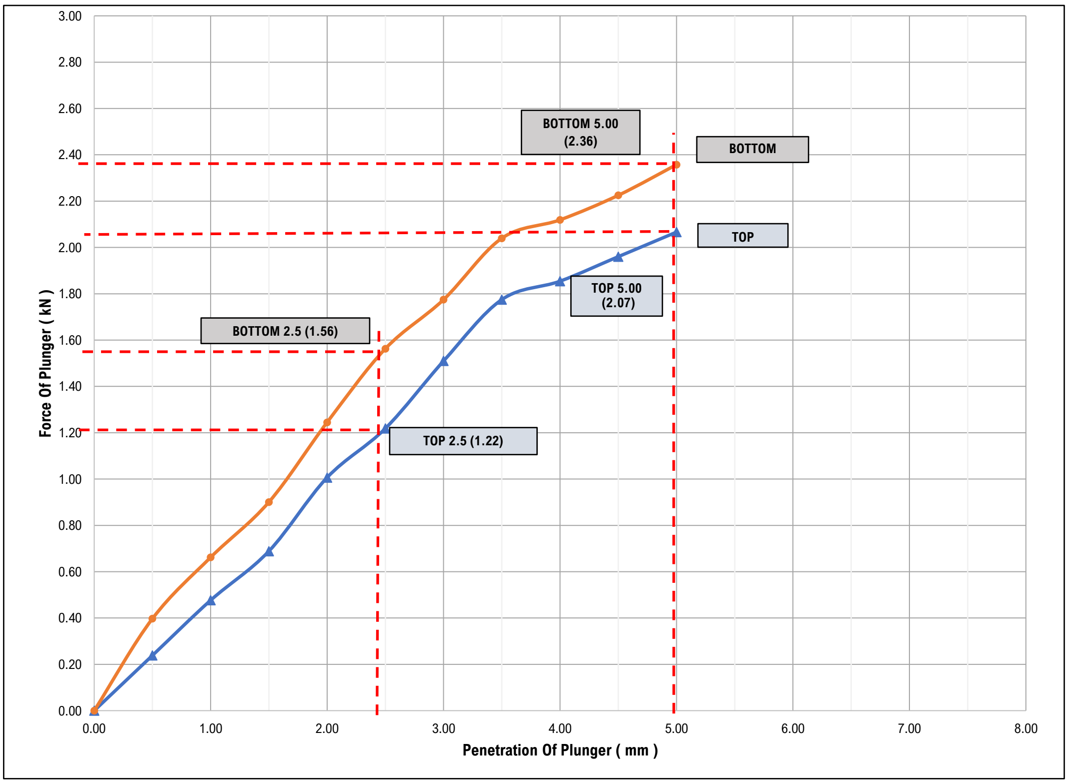
5 layers Method Of Compaction : Vabrating Hammer Time/Duratoin 1 min

Remarks :

Description of material :	**MARINE CLAY + 3% LIME**	Date test :	**20/05/2022**
Sources:	**PULAU BURUNG LANDFILL, PULAU PINANG**	Page :	**2 of 3**

SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4



Plunger Penetration (mm)	Top	
	C.B.R (%)	M/C (%)
2.50	**9.23**	**40.00**
5.00	**10.33**	

Plunger Penetration (mm)	Bottom	
	C.B.R (%)	M/C (%)
2.50	**11.84**	**40.55**
5.00	**11.79**	

Remarks : * Specification Limits from JKR's Design Specification is not less than 12%

Highest Value in this material 11.79%

Remarks :

APPENDIX 1 (H)

LABORATORY WORKSHEET

Description of material :	**MARINE CLAY + 3% LIME**	Date test :	**20/05/2022**	
Source :	**PULAU BURUNG LANDFILL, PULAU PINANG**	Page :	**3 of 3**	

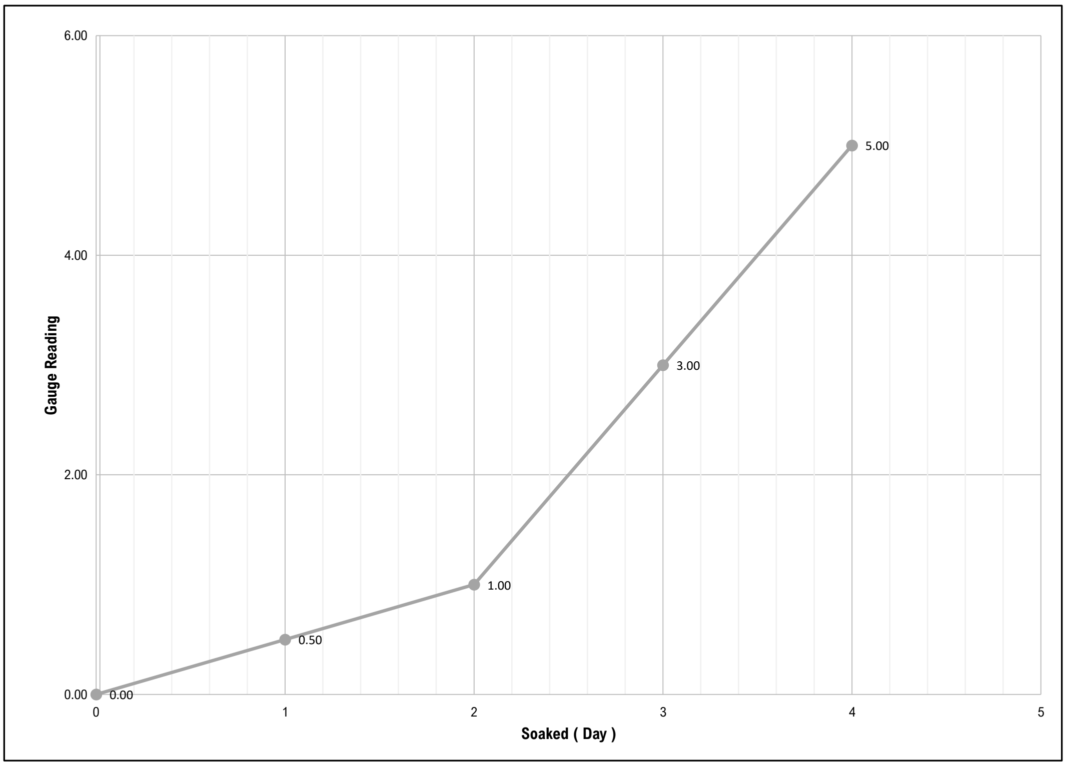
SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

DATE	DAY	TIME	PERIOD		Reading (mm)	REMARKS
20.05.2022	FRIDAY	4.40 PM	0		0.00	FULLY WATER
21.05.2022	SATURDAY	4.40 PM	1		0.50	-
22.05.2022	SUNDAY	4.40 PM	2		1.00	-
23.05.2022	MONDAY	4.40 PM	3		3.00	-
24.05.2022	TUESDAY	4.40 PM	4		5.00	-

Remarks:

C.B.R.(SWELL CHART)



Description of material :	**MARINE CLAY + 6 % LIME**	Date sample :	**09.05.2022**
Sources:	**PULAU BURUNG LANDFILL, PULAU PINANG**	Date test :	**20.05.2022**

APPENDIX 1 (H)

CALIFORNIA BEARING RATIO (CBR) SOAKED 95% MDD

BS 1377 PART 4 /

JKR/SPJ/2008 - S4

A. Preparation Procedure :

	Compresion with tamping
	Compresion in layers
	Rammer compaction to a spec. density
X	Vibrating compaction to a spec. density
	Rammer compaction with specified effort

B. Compaction Data :

1.Max Dry Density (MDD)	1.4740	Mg/m³
2.Optimum Moisture Content	30.17	%
3.Specified Density at 95%	1.4003	Mg/m³
4.Volume of Mould	2305	cm³
5.Specified Effort	5	layers
6.Blow per layer By Vibrating Hammer	1	minit
7.Calculated mass of soil required (m) = $\frac{V}{100}$ * (100 + W) pd	4201	g
8.Soaking Time	4	days
9.Final Swelling	3.5	mm
10.Water to be added	1268	gm

Note: V = Vol Of Mould
W = Moisture Content Of Sample
pd = Selected Density Of 95%

Description of material : **MARINE CLAY + 6% LIME**
SOURCES: **PULAU BURUNG LANDFILL, PULAU PINANG**

Date test : **20/05/2022**

APPENDIX 1 (H)

SOAKED CBR TEST

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

Density Before Soaking		
Mould no		5
Wt Of mould + wet sample	gm	11310
Wt of mould,	gm	7295
Wt of sample	gm	4015
Volume of mould	cm^3	2305
Wet density	mg/m^3	1.742
Average moisture content	%	30.13
Dry density	mg/m^3	1.4740

Moisture Content (Before Soaked)			
Container No		**P6**	**P7**
Wt of cont. + wet sample	gm	**111.20**	**100.70**
Wt of cont. + dry sample	gm	**89.60**	**80.70**
Wt. of container	gm	**16.00**	**16.00**
Wt. of water	gm	**21.60**	**20.00**
Wt. of dry sample	gm	**73.60**	**64.70**
Moisture content	%	**29.35**	**30.91**
Av.Moisture content	%	**30.13**	
Proving ring constant		0.02649 (Div) kNf	

Penetration	Dial Gauge Top	Dial Gauge Bottom	Force Top	Force Bottom	Corrected Top	Corrected Bottom
0.00	0	0	0.00	0.00		
0.50	23	35	0.61	0.93		
1.00	38	43	1.01	1.14		
1.50	47	56	1.25	1.48		
2.00	55	67	1.46	1.77		
2.50	**63**	**78**	**1.67**	**2.07**		
3.00	70	88	1.85	2.33		
3.50	78	98	2.07	2.60		
4.00	85	107	2.25	2.83		
4.50	94	116	2.49	3.07		
5.00	**106**	**123**	**2.81**	**3.26**		
5.50						
6.00						
6.50						
7.00						
7.50						

Expansion & Absorption Determination		
Surcharge	kg	**4.5**
Gauge reading after soaking	Div	**3.50**
Gauge reading before soaking	Div	**0.0000**
Total expansion	mm	**3.5**
Wt. Of mould + sample after soaking	gm	**11350**
Wt. Of mould + sample before soaking	gm	**11310**
Wt. of water absortion	gm	**40.0**

Moisture Content (After Soaked)		**Top**	**Bottom**
Container No		**P8**	**P9**
Wt of cont. + wet sample	gm	**111.20**	**110.40**
Wt of cont. + dry sample	gm	**86.60**	**85.70**
Wt. of container	gm	**16.10**	**16.20**
Wt. of water	gm	**24.60**	**24.70**
Wt. of dry sample	gm	**70.50**	**69.50**
Moisture content	%	**34.89**	**35.54**

Period of soaking	4 days
Method of campaction	-

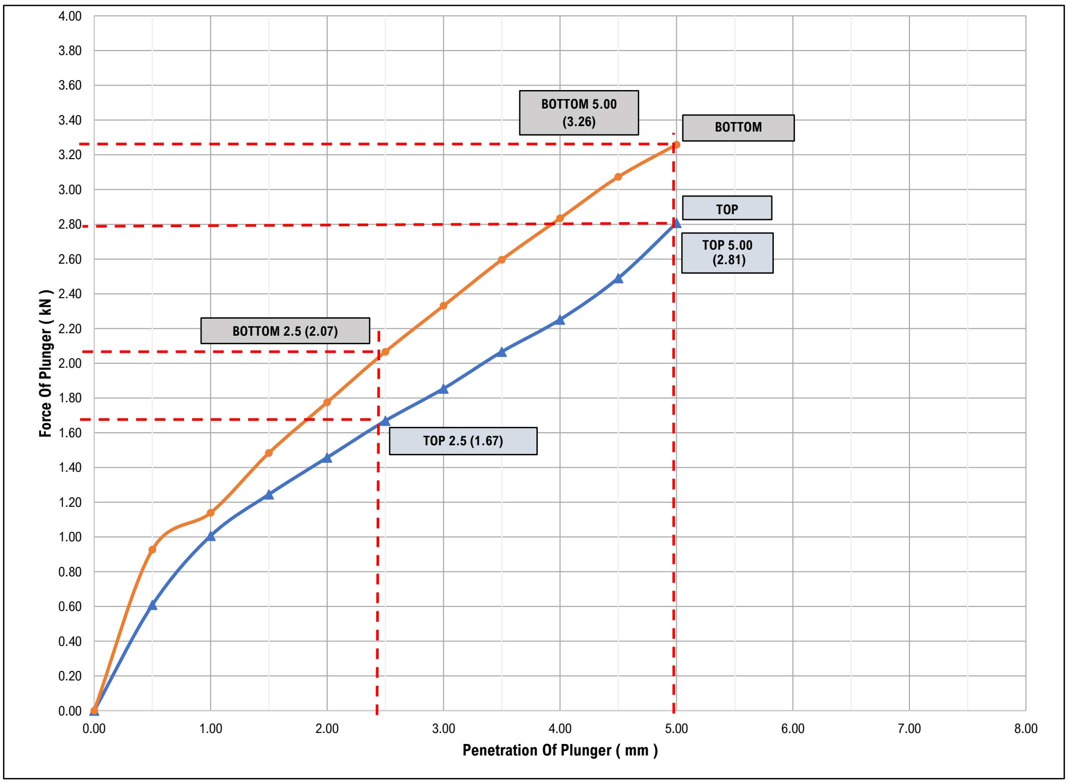
5 layers Method Of Compaction : Vabrating Hammer Time/Duratoin 1 min

Remarks :

Description of material :	**MARINE CLAY + 6% LIME**	Date test :	**20/05/2022**
SOURCES:	**PULAU BURUNG LANDFILL, PULAU PINANG**	Page :	**2 of 3**

SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4



Plunger Penetration (mm)	Top	
	C.B.R (%)	M/C (%)
2.50	**12.64**	**34.89**
5.00	**14.04**	

Plunger Penetration (mm)	Bottom	
	C.B.R (%)	M/C (%)
2.50	**15.65**	**35.54**
5.00	**16.29**	

Remarks : * Specification Limits from JKR's Design Specification is not less than 12%

Highest Value in this material 16.29%

Remarks :

APPENDIX 1 (H)

LABORATORY WORKSHEET

Description of material :	**MARINE CLAY + 6% LIME**	Date test :	**20/05/2022**
Source :	**PULAU BURUNG LANDFILL, PULAU PINANG**	Page :	**3 of 3**

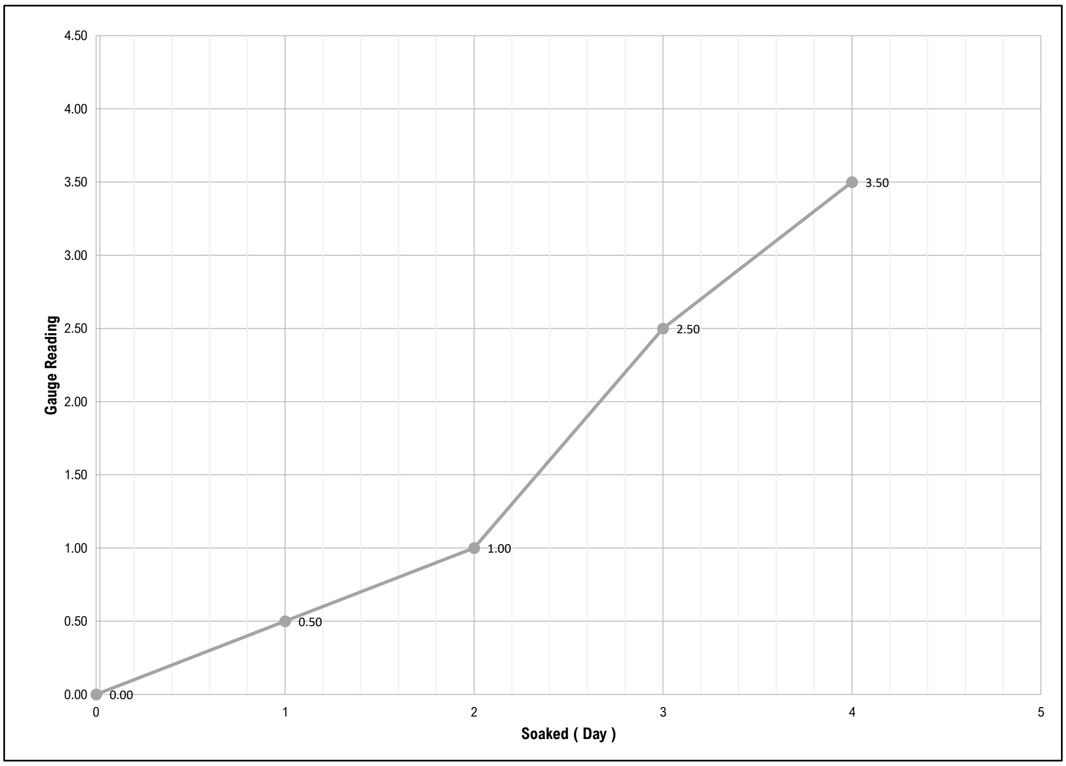
SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

DATE	DAY	TIME	PERIOD		Reading (mm)	REMARKS
20.05.2022	FRIDAY	4.40 PM	0		0.00	FULLY WATER
21.05.2022	SATURDAY	4.40 PM	1		0.50	-
22.05.2022	SUNDAY	4.40 PM	2		1.00	-
23.05.2022	MONDAY	4.40 PM	3		2.50	-
24.05.2026	TUESDAY	4.40 PM	4		3.50	-

Remarks:

C.B.R.(SWELL CHART)



LABORATORY WORKSHEET

APPENDIX 1 (I)

MATERIAL : MARINE CLAY

Date test : **31/05/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	28/05/2022	3	8024.3	15.0	15.0	15.0	3375.0	225.0	2.378		2.6	0.12		S
2	2	28/05/2022	3	8074.4	15.0	15.0	15.0	3375.0	225.0	2.392		3.0	0.13		S
3	3	28/05/2022	3	8026.7	15.0	15.0	15.0	3375.0	225.0	2.378	2.383	2.0	0.09	**0.11**	S
1	4	28/05/2022	7	7990.0	15.0	15.0	15.0	3375.0	225.0	2.367		3.0	0.13		S
2	5	28/05/2022	7	7995.0	15.0	15.0	15.0	3375.0	225.0	2.369		2.8	0.12		S
3	6	28/05/2022	7	8050.0	15.0	15.0	15.0	3375.0	225.0	2.385	2.374	3.2	0.14	**0.13**	S

Note :
1) delete as appropriate
2) determine by client
3) measured nearest to millimeter
4) measured nearest to $10kg / m^3$
5) measured nearest to $0.1 N / mm^2$
S = Satisfactory Failures / U = Unsatisfactory Failures

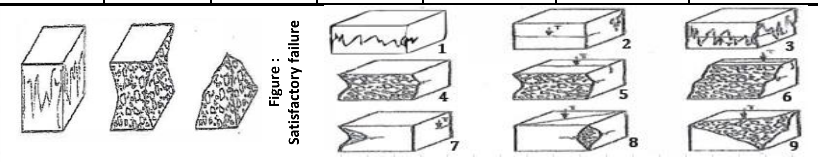


Figure : Satisfactory failure

Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (I)

MATERIAL : MARINE CLAY + 3 % LIME

Date test : **31/05/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	28/05/2022	3	7889.0	15.0	15.0	15.0	3375.0	225.0	2.337		13.7	0.61		S
2	2	28/05/2022	3	7910.0	15.0	15.0	15.0	3375.0	225.0	2.344		13.0	0.58		S
3	3	28/05/2022	3	7270.0	15.0	15.0	15.0	3375.0	225.0	2.154	2.278	13.8	0.61	**0.60**	S
1	4	28/05/2022	7	8010.0	15.0	15.0	15.0	3375.0	225.0	2.373		20.1	0.89		S
2	5	28/05/2022	7	8005.0	15.0	15.0	15.0	3375.0	225.0	2.372		21.9	0.97		S
3	6	28/05/2022	7	7994.0	15.0	15.0	15.0	3375.0	225.0	2.369	2.371	21.0	0.93	**0.93**	S

Note :
1) delete as appropriate
2) determine by client
3) measured nearest to millimeter
4) measured nearest to $10kg / m^3$
5) measured nearest to $0.1 N / mm^2$
S = Satisfactory Failures / U = Unsatisfactory Failures

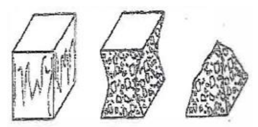
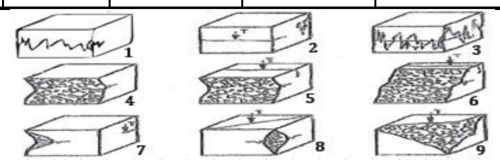


Figure : Satisfactory failure



Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (I)

MATERIAL : MARINE CLAY + 6 % LIME

Date test : **31/05/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	28/05/2022	3	8110.0	15.0	15.0	15.0	3375.0	225.0	2.403		25.7	1.14		S
2	2	28/05/2022	3	8104.0	15.0	15.0	15.0	3375.0	225.0	2.401		24.0	1.07		S
3	3	28/05/2022	3	8080.0	15.0	15.0	15.0	3375.0	225.0	2.394	2.399	26.5	1.18	1.13	S
1	4	28/05/2022	7	8090.0	15.0	15.0	15.0	3375.0	225.0	2.397		46.0	2.04		S
2	5	28/05/2022	7	8020.0	15.0	15.0	15.0	3375.0	225.0	2.376		45.6	2.03		S
3	6	28/05/2022	7	8190.0	15.0	15.0	15.0	3375.0	225.0	2.427	2.400	47.5	2.11	2.06	S

Note :
- [1] delete as appropriate
- [2] determine by client
- [3] measured nearest to millimeter
- [4] measured nearest to 10kg / m^3
- [5] measured nearest to 0.1 N / mm^2
- S = Satisfactory Failures / U = Unsatisfactory Failures

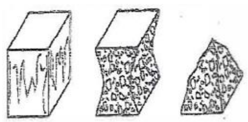
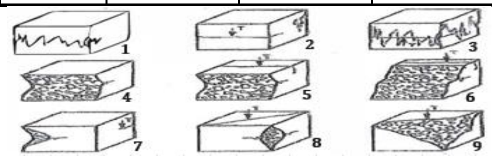


Figure : Satisfactory failure



Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (I)

MATERIAL : MARINE CLAY + 9 % LIME

Date test : **31/05/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	28/05/2022	3	8210.0	15.0	15.0	15.0	3375.0	225.0	2.433		45.5	2.02		S
2	2	28/05/2022	3	8200.0	15.0	15.0	15.0	3375.0	225.0	2.430		47.9	2.13		S
3	3	28/05/2022	3	8230.0	15.0	15.0	15.0	3375.0	225.0	2.439	2.434	51.9	2.31	**2.15**	S
1	4	28/05/2022	7	8290.0	15.0	15.0	15.0	3375.0	225.0	2.456		58.8	2.61		S
2	5	28/05/2022	7	8280.0	15.0	15.0	15.0	3375.0	225.0	2.453		64.7	2.88		S
3	6	28/05/2022	7	8170.0	15.0	15.0	15.0	3375.0	225.0	2.421	2.443	60.9	2.71	**2.73**	S

Note :

- [1] delete as appropriate
- [2] determine by client
- [3] measured nearest to millimeter
- [4] measured nearest to 10kg / m^3
- [5] measured nearest to 0.1 N / mm^2
- S = Satisfactory Failures / U = Unsatisfactory Failures

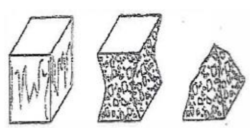
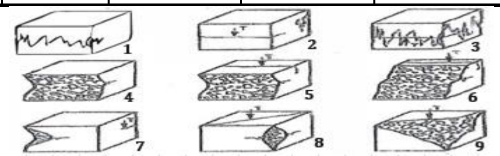


Figure : Satisfactory failure



Remarks :

LABORATORY WORKSHEET

APPENDIX 1 (I)

MATERIAL : MARINE CLAY + 12 % LIME

Date test : **15/12/2023**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	12/12/2023	3	8290.0	15.0	15.0	15.0	3375.0	225.0	2.456		94.8	4.21		S
2	2	12/12/2023	3	8300.0	15.0	15.0	15.0	3375.0	225.0	2.459		96.0	4.27		S
3	3	12/12/2023	3	8270.0	15.0	15.0	15.0	3375.0	225.0	2.450	2.455	96.0	4.27	**4.25**	S
1	4	12/12/2023	7	8320.0	15.0	15.0	15.0	3375.0	225.0	2.465		114.0	5.07		S
2	5	12/12/2023	7	8330.0	15.0	15.0	15.0	3375.0	225.0	2.468		117.0	5.20		S
3	6	12/12/2023	7	8300.0	15.0	15.0	15.0	3375.0	225.0	2.459	2.464	115.8	5.15	**5.14**	S

Note :
1) delete as appropriate
2) determine by client
3) measured nearest to milimeter
4) measured nearest to 10kg / m^3
5) measured nearest to 0.1 N / mm^2
S = Satisfactory Failures / U = Unsatisfactory Failures

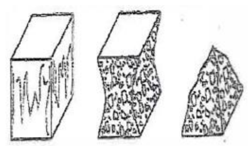
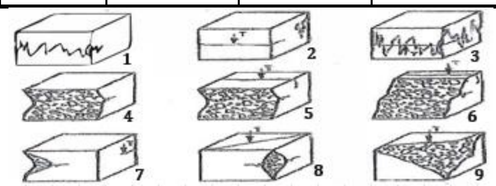


Figure : Satisfactory failure



Remarks :

DATA STRENGTH 3DAY

SOIL	0.11	0	0	0
3%	0.60	0	0	0
6%	1.13	0	0	0
9%	2.15	0	0	0
12%	4.25	0	0	0

DATA STRENGTH 7DAY

SOIL	0.13	0	0	0
3%	0.93	0	0	0
6%	2.06	0	0	0
9%	2.73	0	0	0
12%	5.14	0	0	0

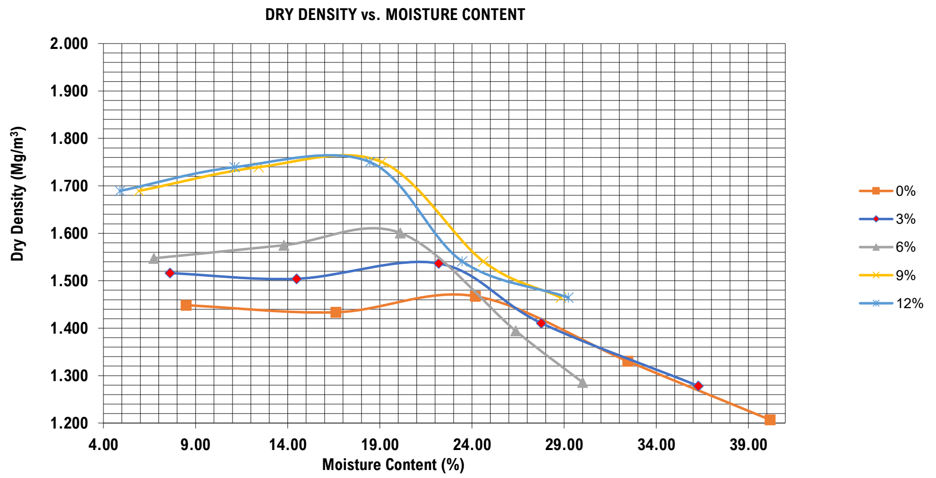
LABORATORY WORKSHEET

APPENDIX (I)

Description of material : **MARINE CLAY + LIME**
Source : **PULAU BURUNG LANDFILL, SEBERANG PRAI, PENANG**
Date sample :
Date test : **22.06.2021**

CHART DRY DENSITY VS MOISTURE CONTENT

LIME	%	0%	3%	6%	9%	12%
Dry Density 0% Lime	Mg/m^3	1.448	1.433	1.467	1.330	1.207
Dry Density 3% Lime	Mg/m^4	1.516	1.504	1.536	1.411	1.278
Dry Density 6% Lime	Mg/m^4	1.547	1.575	1.601	1.394	1.286
Dry Density 9% Lime	Mg/m^4	1.625	1.648	1.682	1.503	1.383
Dry Density 12% Lime	Mg/m^4	1.689	1.739	1.750	1.541	1.464
Moisture content 0% Lime	%	8.49	16.62	24.19	32.45	40.18
Moisture content 3% Lime	%	7.61	14.48	22.20	27.76	36.29
Moisture content 6% Lime	%	6.73	13.80	20.11	26.37	30.00
Moisture content 9% Lime	%	5.93	12.43	19.10	24.61	28.82
Moisture content 12% Lime	%	4.90	11.11	18.45	23.47	29.23



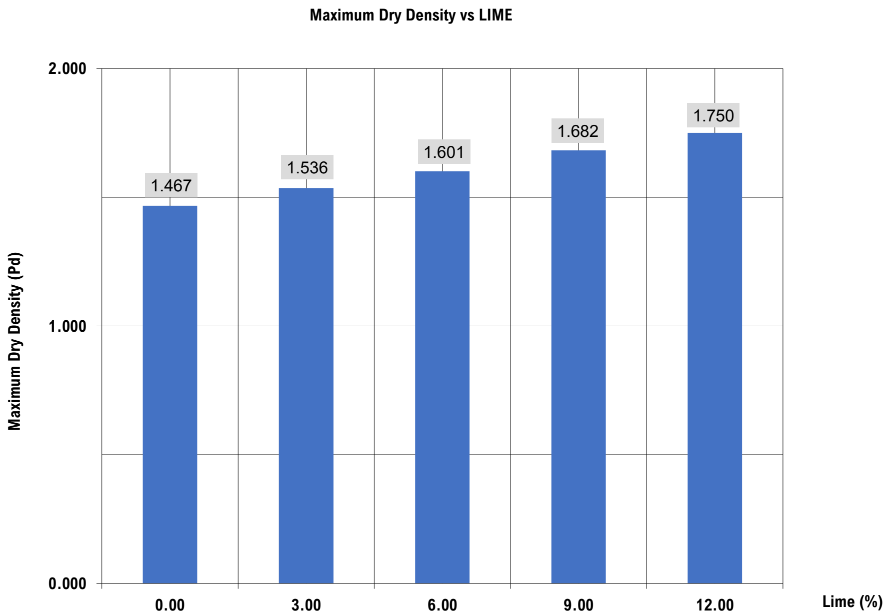
LABORATORY WORKSHEET

APPENDIX 1 (K)

Description of material : **MARINE CLAY + QUICKLIME**
Source : **PULAU BURUNG LANDFILL, SEBERANG PRAI, PENANG**
Date sample :
Date test : **22.06.2021**

BAR CHART Max Pd VS % Quicklime

Maximum Dry Density	**Pd**	**1.467**	**1.536**	**1.601**	**1.682**	**1.750**
Quicklime	**%**	**0**	**3**	**6**	**9**	**12**



LABORATORY WORKSHEET

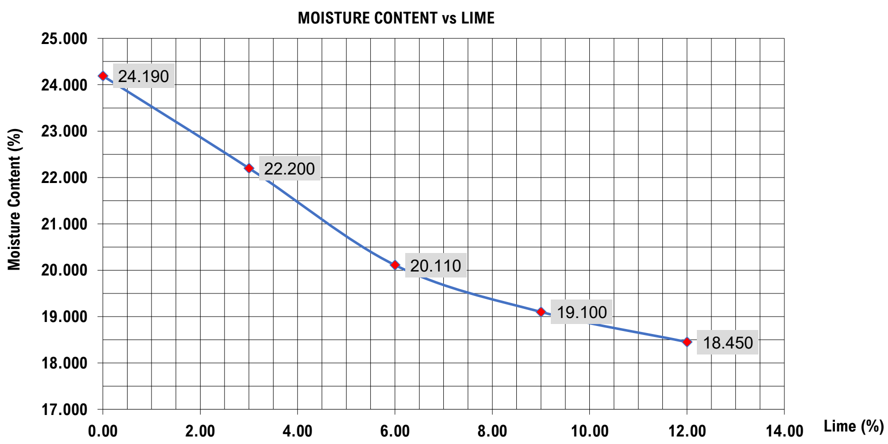
APPENDIX 1 (L)

Description of material : **MARINE CLAY + LIME**
Source : **PULAU BURUNG LANDFILL, SEBERANG PRAI, PENANG**

Date sample :
Date test : **22.06.2021**

CHART MOISTURE CONTENT VS LIME

Moisture content	**%**	**24.19**	**22.20**	**20.11**	**19.10**	**18.45**
Lime	**%**	**0**	**3**	**6**	**9**	**12**



LABORATORY WORKSHEET

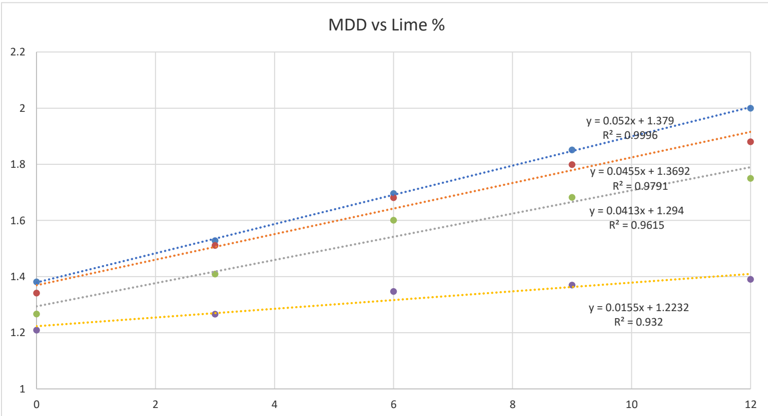
APPENDIX 1 (M)

Description of material : **MARINE CLAY + LIME**
Source :

Date sample :
Date test : **22.06.2021**

CHART MAXIMUM DRY DENSITY (MDD) VS PLASTICITY INDEX (PI)

			0%	3%	6%	9%	12%
Max Dry Density	Sample 1: > 300mm thick	%	1.467	1.536	1.601	1.682	1.750
Plasticity Index	at Pulau Burung Landfill	%	22.09	22.09	22.09	22.09	22.09
Max Dry Density	Sample 2: 300mm thick	%	1.209	1.266	1.347	1.270	1.129
Plasticity Index	at Pulau Burung Landfill	%	35.86	35.86	35.86	35.86	35.86
Max Dry Density	Sample 3 - MMTS	%	1.441	1.511	1.681	1.799	1.880
Plasticity Index	at Ijok Selangor	%	29.70	29.70	29.70	29.70	29.70
Max Dry Density	Sample 4 - MMTS	%	1.3810	1.5280	1.6560	1.8510	1.8970
Plasticity Index	at Ijok, Selangor	%	20.84	20.84	20.84	20.84	20.84



RESULT AND FINDING TO IDENTIFY CEMENT (Cp) / LIME CONTENT (Lc)

	CATEGORY (PI)	Criteria PI					
A	PLASTICITY INDEX (PI)		<	20%			USE
B	PLASTICITY INDEX (PI)	20%	<	PI	<	25%	USE
C	PLASTICITY INDEX (PI)	25%	<	PI	<	30%	USE
D	PLASTICITY INDEX (PI)		>	30%			USE

Equation						% Confident
MDD	=	0.0527	Lp	+	1.379	100.0%
MDD	=	0.0455	Lp	+	1.369	97.9%
MDD	=	0.0413	Lp	+	1.294	96.2%
MDD	=	0.0155	Lp	+	1.223	93.2%

CATEGORY A **MDD** = 1.75 mg/m3
PI = 20%

MDD = 0.0527 Lp + 1.379

MDD - 1.379 = 0.0527 Lp

$$Lp = \frac{MDD - 1.379}{0.0527}$$

$$Lp = \frac{1.750 - 1.379}{0.0527}$$

Lp = 7 %

CATEGORY B

MDD = 1.75 mg/m3
PI = 23%

MDD = 0.0455 Lp + 1.369

MDD - 1.369 = 0.0455 Lp

$$Lp = \frac{MDD - 1.369}{0.0455}$$

$$Lp = \frac{1.750 - 1.369}{0.0455}$$

Lp = 8 %

CATEGORY C

MDD = 1.75 mg/m3
PI = 28%

MDD = 0.0413 Lp + 1.294

MDD - 1.294 = 0.0413 Lp

$$Lp = \frac{MDD - 1.294}{0.0413}$$

$$Lp = \frac{1.750 - 1.294}{0.0413}$$

Lp = 11 %

CATEGORY D

MDD = 1.75 mg/m3
PI = 30%

MDD = 0.0155 Lp + 1.223

MDD - 1.223 = 0.0155 Lp

$$Lp = \frac{MDD - 1.223}{0.0155}$$

$$Lp = \frac{1.750 - 1.223}{0.0155}$$

Lp = 34 %

APPENDIX 2

A. MOISTURE CONTENT
B. PROCTOR TEST
SAMPLES: ORIGINAL Top < 300mm Depth, 3%, 6%, 9% AND 12% CEMENT)
SAMPLES: ORIGINAL Bottom >300mm Depth, 3%, 6%, 9% AND 12% CEMENT)
C. ATTERBERG LIMIT
D. HYDROMETER SEDIMETATION TEST
E. PARTICLE DENSITY TEST
F. SIEVE ANALYSIS
G. pH TEST (ORIGINAL, 3%, 6%, 9% and 12% CEMENT)
H. SOAKED CBR (ORIGINAL, 3% and 6% CEMENT)
I. UNCONFINED COMPRESSIVE STRENGTH (ORI., 3%, 6%, 9% AND 12%)
J. DRY DENSITY VS MOINTURE CONTENT
K. MAXIMUM DRY DENSITY VS % CEMENT (ORI., 3%, 6%, 9% AND 12%)
L. MOISTURE CONTENT VS CEMENT CONTENT (ORI., 3%, 6%, 9% AND 12%)
M. MAXIMUM DRY DENSITY VS PI (ORI., 3%, 6%, 9% AND 12%) – **NOVELTY**

APPENDIX 2 (A)

Soil Description : LATERITE SOIL Drying Method: HOT PLAT/ SAND BATCH						Date Sample: 03/07/2022 Date Test : 04/07/2022
NATURAL MOISTURE CONTENT						
MOISTURE CONTENT						
Container No.		1	2	3		
Wt. of wet Soil + Container (M2)	gm	53.48	44.32	46.88		
Wt. of dry Soil + Container (M3)	gm	48.72	39.66	42.25		
Wt. of Container (M1)	gm	23.48	14.31	16.88		
Wt. of Moisture (M2-M3)	gm	4.76	4.66	4.63		
Wt. of dry Soil (M3-M1)	gm	25.24	25.35	25.37		
Moisture Content W =100(M2-M3)/(M3-M1)%		18.86	18.38	18.25		
NATURAL MOISTURE CONTENT	%	**18.86**	**18.38**	**18.25**		
AVERAGE			**18.50**			

LABORATORY WORKSHEET

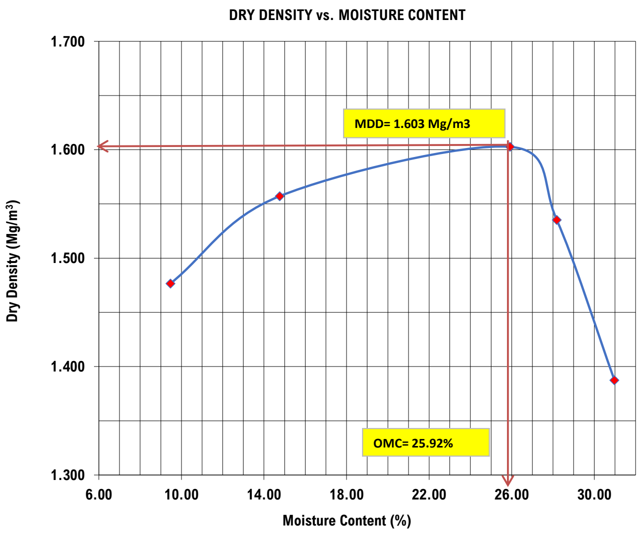
Description of material :	**LATERITE**	Date sample :	**03/07/2022**
Source :	**BORROW PIT**	Date test :	**05/07/2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number		1	2	3	4	5
Mass of mould + base + compacted specimen (M2)	gm	6590	6760	6990	6940	6790
Mass of mould + base (M1)	gm	4980	4980	4980	4980	4980
Mass of compacted specimen (M2-M1)	gm	1610	1780	2010	1960	1810
Bulk density = $\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.616**	**1.787**	**2.018**	**1.968**	**1.817**
Dry density = $\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.477**	**1.557**	**1.603**	**1.535**	**1.387**
Moisture content container No.						
Mass of container + bulk specimen (m1)	gm	43.82	46.42	47.56	46.07	46.71
Mass of container (m2)	gm	13.88	16.42	17.49	15.96	16.69
Mass of container + dried specimen (m3)	gm	41.23	42.56	41.37	39.45	39.61
Moisture content = $\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**9.47**	**14.77**	**25.92**	**28.18**	**30.98**

** Delete as appropriate*



Test Method : 2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.603**	Mg/m^3
Opt. Moisture Content	**25.92**	(%)

Remarks :

LABORATORY WORKSHEET

Description of material :	**LATERITE + 3 % CEMENT**	Date sample :	**03/07/2022**
Source :	**BORROW PIT**	Date test :	**05.07.2022**

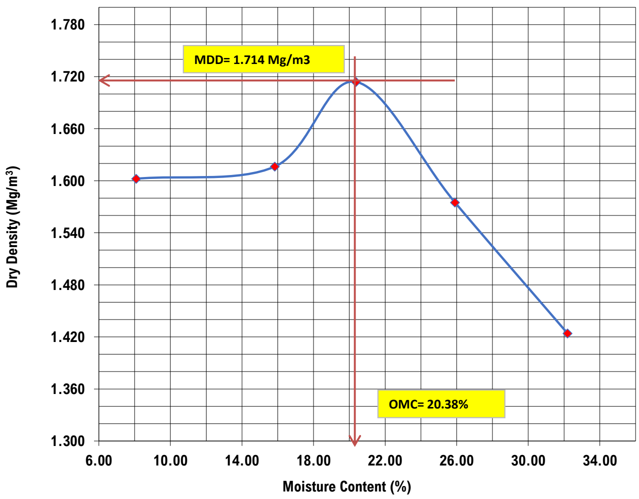
PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number			1	2	3	4	5
Mass of mould + base + compacted specimen (M2)		gm	5790	5930	6120	6040	5940
Mass of mould + base (M1)		gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)		gm	1725	1865	2055	1975	1875
Bulk density =	$\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.732**	**1.872**	**2.063**	**1.983**	**1.883**
Dry density =	$\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.602**	**1.616**	**1.714**	**1.575**	**1.424**
Moisture content container No.							
Mass of container + bulk specimen (m1)		gm	72.80	88.00	86.00	77.00	70.00
Mass of container (m2)		gm	14.00	17.80	15.70	14.80	15.40
Mass of container + dried specimen (m3)		gm	68.40	78.40	74.10	64.20	56.70
Moisture content =	$\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**8.09**	**15.84**	**20.38**	**25.91**	**32.20**

** Delete as appropriate*

DRY DENSITY vs. MOISTURE CONTENT



Test Method : 2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.714**	Mg/m^3
Opt. Moisture Content	**20.38**	(%)

Remarks :

LABORATORY WORKSHEET

Description of material : **LATERITE + 6% CEMENT**
Source : **STOCKE PILE**

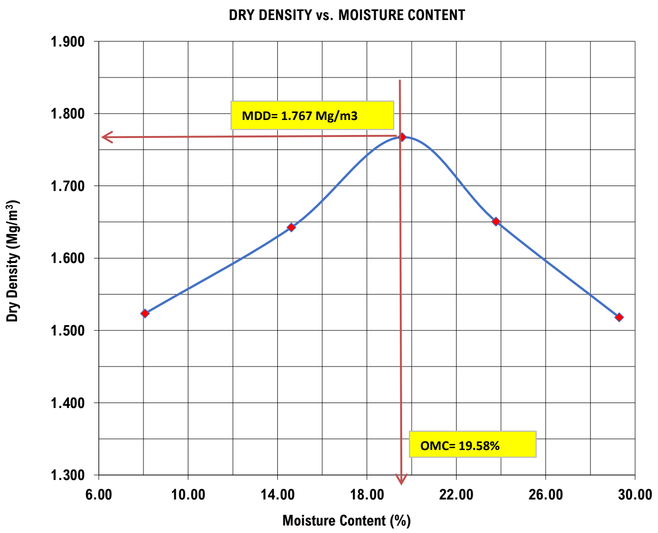
Date sample : **03/07/2022**
Date test : **05.07.2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number			1	2	3	4	5
Mass of mould + base + compacted specimen (M2)		gm	5705	5940	6170	6100	6020
Mass of mould + base (M1)		gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)		gm	1640	1875	2105	2035	1955
Bulk density =	$\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.647**	**1.883**	**2.113**	**2.043**	**1.963**
Dry density =	$\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.524**	**1.642**	**1.767**	**1.651**	**1.518**
Moisture content container No.							
Mass of container + bulk specimen (m1)		gm	77.00	83.00	71.00	75.00	78.00
Mass of container (m2)		gm	14.10	14.00	14.20	14.10	14.00
Mass of container + dried specimen (m3)		gm	72.30	74.20	61.70	63.30	63.50
Moisture content =	$\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**8.08**	**14.62**	**19.58**	**23.78**	**29.29**

** Delete as appropriate*



Test Method : 2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.767**	Mg/m^3
Opt. Moisture Content	**19.58**	(%)

Remarks :

LABORATORY WORKSHEET

Description of material : **LATERITE + 9 % CEMENT**
Source : **BORROW PIT**

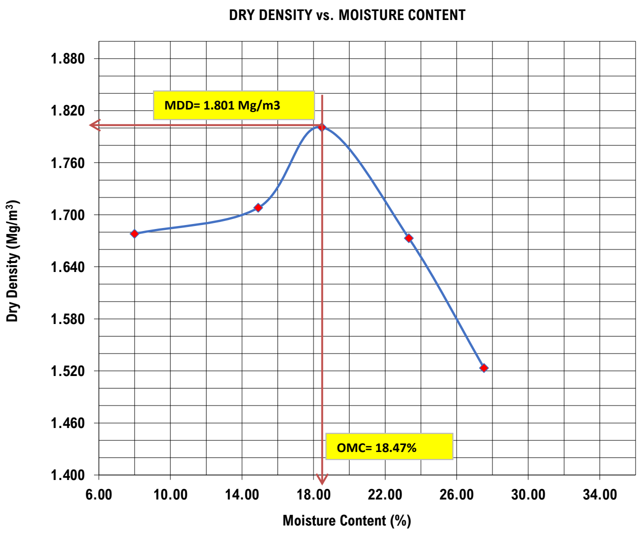
Date sample : **03/07/2022**
Date test : **05.07.2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number		1	2	3	4	5
Mass of mould + base + compacted specimen (M2)	gm	5870	6020	6190	6120	6000
Mass of mould + base (M1)	gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)	gm	1805	1955	2125	2055	1935
Bulk density = $\frac{M2 - M1}{(3.142 \times d^2 /4) \times h}$	Mg/m^3	**1.812**	**1.963**	**2.134**	**2.063**	**1.943**
Dry density = $\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.678**	**1.708**	**1.801**	**1.673**	**1.523**
Moisture content container No.						
Mass of container + bulk specimen (m1)	gm	70.80	87.60	88.40	75.00	77.00
Mass of container (m2)	gm	14.10	14.40	14.00	14.20	14.00
Mass of container + dried specimen (m3)	gm	66.60	78.10	76.80	63.50	63.40
Moisture content = $\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**8.00**	**14.91**	**18.47**	**23.33**	**27.53**

** Delete as appropriate*



Test Method : 2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.801**	Mg/m^3
Opt. Moisture Content	**18.47**	(%)

Remarks :

LABORATORY WORKSHEET

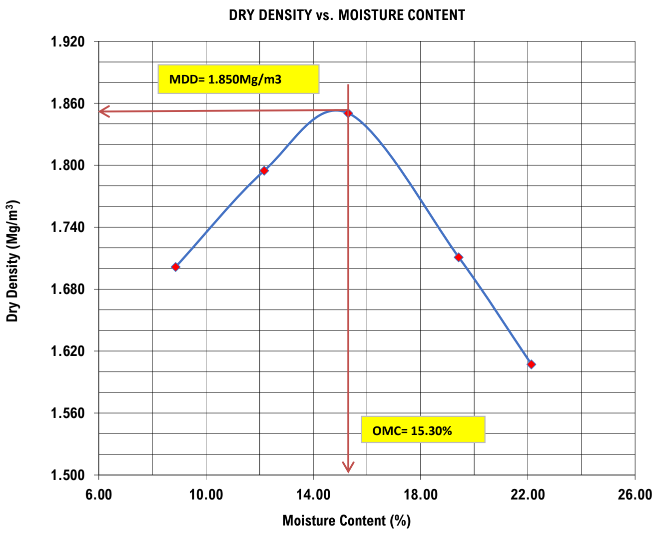
Description of material :	**LATERITE + 12 % CEMENT**	Date sample :	**03/07/2022**
Source :	**BORROW PIT**	Date test :	**05.07.2022**

PROCTOR TEST (COMPACTION TEST)

BS 1377 Part4 : 1990 : 3.3/3.4/3.5/3.6 * / JKR/SPJ/2008-S4

Test Number		1	2	3	4	5
Mass of mould + base + compacted specimen (M2)	gm	5910	6070	6190	6100	6020
Mass of mould + base (M1)	gm	4065	4065	4065	4065	4065
Mass of compacted specimen (M2-M1)	gm	1845	2005	2125	2035	1955
Bulk density = $\frac{M2 - M1}{(3.142 \times d^2/4) \times h}$	Mg/m^3	**1.852**	**2.013**	**2.134**	**2.043**	**1.963**
Dry density = $\frac{100 \times \text{bulk density}}{(100 + w)}$	Mg/m^3	**1.702**	**1.795**	**1.850**	**1.711**	**1.607**
Moisture content container No.						
Mass of container + bulk specimen (m1)	gm	88.90	85.90	90.40	92.90	92.90
Mass of container (m2)	gm	14.00	14.00	14.30	14.20	14.00
Mass of container + dried specimen (m3)	gm	82.80	78.10	80.30	80.10	78.60
Moisture content = $\frac{(m1 - m3) \times 100}{m3 - m2}$	%	**8.87**	**12.17**	**15.30**	**19.42**	**22.14**

** Delete as appropriate*



Test Method : 2.5/4.5 kg * hand/mechanical rammer		
Mould Diameter. (d)	0.104	m
Sample height (h)	0.1162	m
Vol. of Mould	996	cm^3
Blow/layer	27	cm^3
No. Of Layer	5	Layers

Results :

Max. Dry Density	**1.850**	Mg/m^3
Opt. Moisture Content	**15.30**	(%)

Remarks :

LABORATORY WORKSHEET

Description of material : **LATERITE** Date sample : **03/07/2022**
Source : **Borrow Pit** Date test : **06.07.2022**

ATTERBERG LIMIT

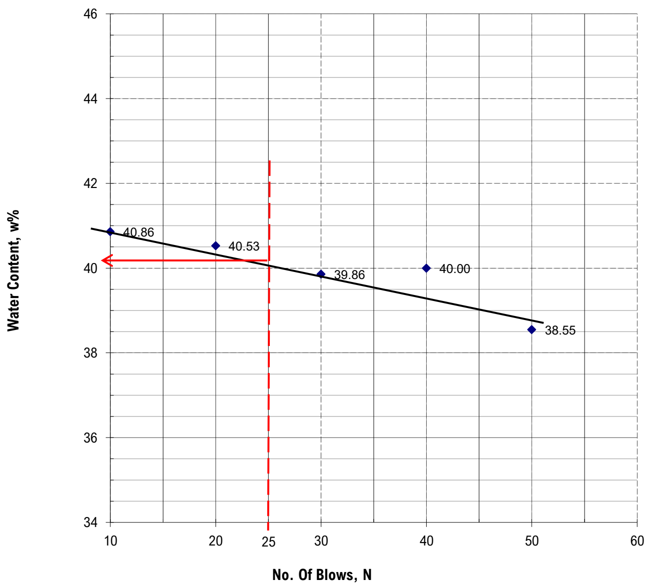
(Casagrande Method) / BS 1377: Part 2: 1990: 4.5/4.6* / JKR/SPJ/2008-S4

PLASTIC LIMIT

Test no.		1	2	3	4	5	Average Moisture Content
Container no.		N1	N2				
Mass of wet soil + container	gm	18.59	18.89				
Mass of dry soil + container	gm	18.19	18.41				
Mass of container	gm	16.67	16.87				**28.74**
Mass of moisture	gm	0.40	0.48				
Mass of dry soil	gm	1.52	1.54				
Moisture content	%	**26.32**	**31.17**				

LIQUID LIMIT

Test no.		1	2	3	4	5	Average Moisture Content
Container no.							
Number of bumps / Blows/Peneration		50	40	30	20	10	
Mass of wet soil + container	gm	33.39	27.31	27.01	26.60	28.61	
Mass of dry soil + container	gm	30.63	23.59	24.22	23.69	25.19	
Mass of container	gm	23.47	14.29	17.22	16.51	16.82	**39.96**
Mass of moisture	gm	2.76	3.72	2.79	2.91	3.42	
Mass of dry soil	gm	7.16	9.30	7.00	7.18	8.37	
Moisture content	%	**38.55**	**40.00**	**39.86**	**40.53**	**40.86**	



Sample Preparation*		
As received washed on 425μm sieve		
*Air Dried	-	°C
*Oven Dried	-	°C
Proportion retained on 425μm sieve	-	°C
(*Delete as appropriate)		

RESULTS	**(%)**
LIQUID LIMIT	39.96
PLASTIC LIMIT	28.74
PLASTICITY INDEX	**11.22**

Remarks :

PARTICLE DENSITY TEST

APPENDIX 2 (D)

Location/ Source	BORROW PIT / LATERITE	Sample no :	S1
		Date sample :	03.07.2022
		Date test :	08.07.2022

Sample no.	1	2	3
Mass of bottle, m_1 (g)	42.28	48.06	38.7
Mass of bottle + soil, m_2 (g)	62.59	67.9	58.63
Mass of bottle + soil + water, m_3 (g)	152.76	161.15	149.5
Mass of bottle full of water, m_4 (g)	141.07	150.64	138.25
Particle density, ρ_s (Mg/m^3)	2.356	2.126	2.296
Average value ρ_s (Mg/m^3)	**2.260**		

No.	Date	Time	Elapsed Time, t (min)	Temp (degree Celsius)	Reading, Rh'	Rh' + Cm =Rh	Effective Depth, Hr (mm)	Particle Diameter, D (mm)	Rh'-R0' =Rd
1	8/07/2022	10.25 am	0	28	25			0.063	
2	8/07/2022	10.25 am	0.5	28	24	27	110	0.057	22
3	8/07/2022	10.26 am	1	28	23	26	113.8	0.041	21
4	8/07/2022	10.27 am	2	28	22	25	117.6	0.029	20
5	8/07/2022	10.31 am	4	28	21	24	121.4	0.021	19
6	8/07/2022	10.39 am	8	28	21	24	121.4	0.015	19
7	8/07/2022	11.09 am	30	28	21	24	121.4	0.008	19
8	8/07/2022	1.39 pm	120	28	21	24	121.4	0.004	19
9	8/07/2022	9.40 am	1080	28	17	20	136.6	0.001	15

Hydrometer Data:	
Meniscus Correction, Cm =	3
Reading In Dispersant, Ro' =	2
Dry Mass of Soil, m (g) =	50
Viscosity of Water At 28 Degree, B =	0.798
Particle Density of Soil, A=	2.65

Calibration Reading	
L (mm) =	310
Vh (Volume of Bulb = Mass of Bulb)	72.15
h (cm) =	13
N (cm) =	0.6
d (cm) =	1.2
H (cm) =	4.2

Percentage Finer than D (K%)	Cumulative Percentage Passing (%)
	99.82
70.664	69.42
67.452	46.82
64.24	30.87
61.028	18.39
61.028	11.59
61.028	5.49
61.028	3.39
48.18	0.39

LABORATORY WORKSHEET

Description of material :	**LATERITE**	Date sample :	**03/07/2022**
Source :	**BORROW PIT**	Date test :	**08.07.2022**
Sample preparation :	air dried **/oven dried/** others *	Page :	**1 of 2**

SIEVE ANALYSIS

JKR

/SPJ/2008 - S4

B.S. Sieve (mm)	Weight Retained (g)	Retained (%)	Passing (%)	MS 30: PART 4: 1995 BS 1377 Part 2 :1990 : 9.2 / 9.3 / 9.4 / 9.6 / 9.7*					
75.00	0.00	0	**100**						
4.75	0.20	0.02	**99.98**						
3.35	0.40	0.04	**99.94**						
2.00	0.20	0.02	**99.92**						
1.18	0.10	0.01	**99.91**						
0.60	0.20	0.02	**99.89**						
0.425	0.20	0.02	**99.87**						
0.300	0.10	0.01	**99.86**						
0.212	0.20	0.02	**99.84**						
0.150	0.10	0.01	**99.83**						
(µm)									
0.063	0.10	0.01	**99.82**						
HYDROMETER									
0.057	304.00	30.40	**69.42**						
0.041	226.00	22.60	**46.82**						
0.029	159.50	15.95	**30.87**						
0.021	124.80	12.48	**18.39**						
0.051	68.00	6.80	**11.59**						
0.008	61.00	6.10	**5.49**						
0.004	21.00	2.10	**3.39**						
0.001	30.00	3.00	**0.39**						
Total (g)	**995.0**								

Weight of sample before oven	(g)	**1000.0**
Weight of sample after oven	(g)	**995.0**

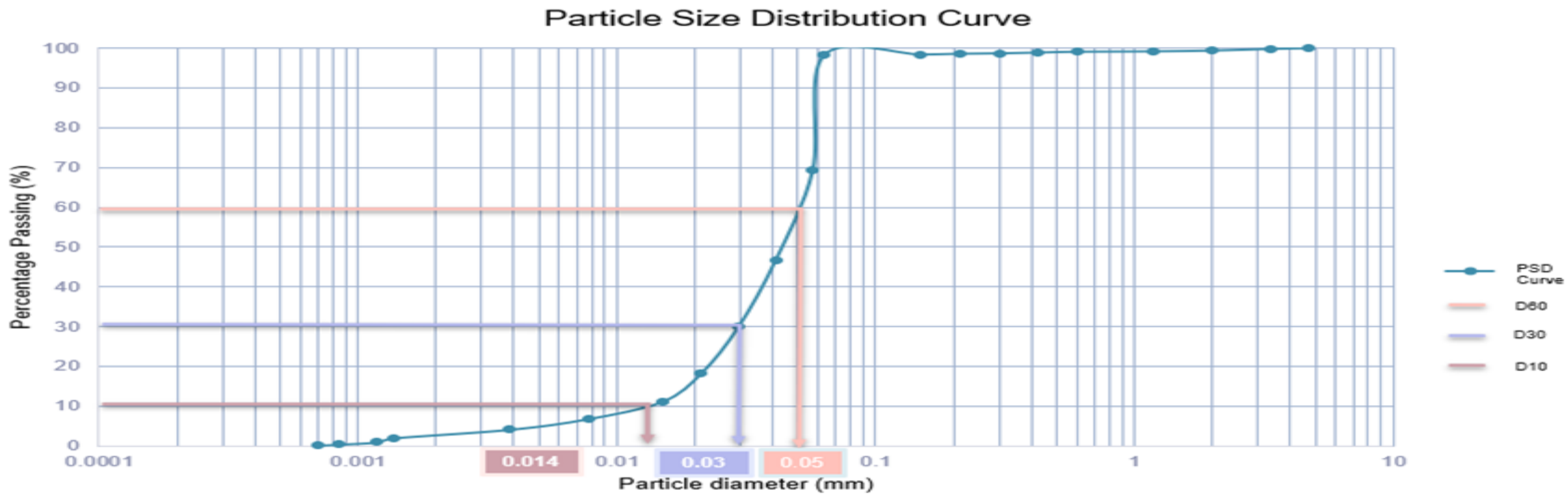
Remarks :

LABORATORY WORKSHEET

Source :	BORROW PIT / LATERITE	Date sample :	**03/07/2022**
Sample preparation :	air dried /**oven dried**/ others *	Date test :	**08.07.2022**
		Page :	**2 of 2**

SIEVE ANALYSIS - DRY SIEVING (CHART)

JKR/SPJ/2008-S4



Remarks :

Gravelly	**0.60**	%	**Clay**	**2.91**	%	**D60 (mm)**	0.05	Cu	3.57
Sandy	**1.40**	%				**D30 (mm)**	0.03	Cg	1.29
Silty	**94.90**	%				**D10 (mm)**	0.014		

PH TEST

DISCRIPTION	NO 1	NO 2	NO 3	NO 4	NO 6
Weight Of Speciment (gm)	20.00	20.00	20.00	20.00	20.00
Weight Of Container (gm)	37.00	37.50	37.80	37.20	37.00
Weight Of Speciment + Container (gm)	57.00	57.50	57.80	57.20	57.00
Water 100mℓ (gm)	100.0	100.0	100.0	100.0	100.0
	3%	**6%**	**9%**	**12%**	0%
Weight Of Cement (%)	4	6	8	10	SOIL
Reading Of pH 1hr	**12.10**	**12.35**	**12.5**	**12.8**	**4.3**
Temperature 0 °c	32	32	32.5	32.8	32

moisture content : 18.50%

No.	pH value	Average pH
1	4.27	
2	4.32	4.35
3	4.47	

APPENDIX 2 (G)

Description of material :	**LATERITE**	Date sample :	**03.07.2022**
		Date test :	**14.07.2022**

CALIFORNIA BEARING RATIO (CBR) SOAKED 95% MDD

BS 1377 PART 4 /

JKR/SPJ/2008 - S4

A. Preparation Procedure :

	Compresion with tamping
	Compresion in layers
	Rammer compaction to a spec. density
X	Vibrating compaction to a spec. density
	Rammer compaction with specified effort

B. Compaction Data :

1.Max Dry Density (MDD)	1.6030	Mg/m^3
2.Optimum Moisture Content	25.92	%
3.Specified Density at 95%	1.5229	Mg/m^3
4.Volume of Mould	2305	cm^3
5.Specified Effort	5	layers
6.Blow per layer By Vibrating Hammer	1	minit
7.Calculated mass of soil required (m) = $\frac{V}{100} * (100 + W)$ pd	4420	g
8.Soaking Time	4	days
9.Final Swelling	2.6	mm
10.Water to be added	1146	gm

Note: V = Vol Of Mould
W = Moisture Content Of Sample
pd = Selected Density Of 95%

Description of material : **LATERITE** Date test : **14.07.2022**
Sources: **BORROW PIT** Page : **1 of 3**

SOAKED CBR TEST

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

Density Before Soaking		
Mould no		1
Wt Of mould + wet sample	gm	11270
Wt of mould,	gm	7290
Wt of sample	gm	3980
Volume of mould	cm^3	2305
Wet density	mg/m^3	1.727
Average moisture content	%	25.46
Dry density	mg/m^3	1.6030

Moisture Content (Before Soaked)			
Container No			
Wt of cont. + wet sample	gm	**100.30**	**98.90**
Wt of cont. + dry sample	gm	**82.80**	**82.10**
Wt. of container	gm	**15.20**	**15.00**
Wt. of water	gm	**17.50**	**16.80**
Wt. of dry sample	gm	**67.60**	**67.10**
Moisture content	%	**25.89**	**25.04**
Av.Moisture content	%	**25.46**	

Proving ring constant	0.02649 (Div) kNf

Penetration	Dial Gauge		Force		Corrected	
	Top	**Bottom**	**Top**	**Bottom**	Top	Bottom
0.00	0	0	0.00	0.00		
0.50	4	7	0.11	0.19		
1.00	10	14	0.26	0.37		
1.50	15	20	0.40	0.53		
2.00	21	28	0.56	0.74		
2.50	**27**	**32**	**0.72**	**0.85**		
3.00	34	40	0.90	1.06		
3.50	40	45	1.06	1.19		
4.00	44	53	1.17	1.40		
4.50	50	60	1.32	1.59		
5.00	**53**	**66**	**1.40**	**1.75**		
5.50						
6.00						
6.50						
7.00						
7.50						

Expansion & Absorption Determination		
Surcharge	kg	**4.5**
Gauge reading after soaking	Div	**2.60**
Gauge reading before soaking	Div	**0.0000**
Total expansion	mm	**2.6**
Wt. Of mould + sample after soaking	gm	**11630**
Wt. Of mould + sample before soaking	gm	**11205**
Wt. of water absortion	gm	**425.0**

Moisture Content (After Soaked)		**Top**	**Bottom**
Container No			
Wt of cont. + wet sample	gm	**110.30**	**103.80**
Wt of cont. + dry sample	gm	**88.90**	**83.70**
Wt. of container	gm	**15.00**	**15.00**
Wt. of water	gm	**21.40**	**20.10**
Wt. of dry sample	gm	**73.90**	**68.70**
Moisture content	%	**28.96**	**29.26**

Period of soaking	4 days
Method of campaction	-

5 layers Method Of Compaction : Vabrating Hammer Time/Duratoin 1 min

Remarks :

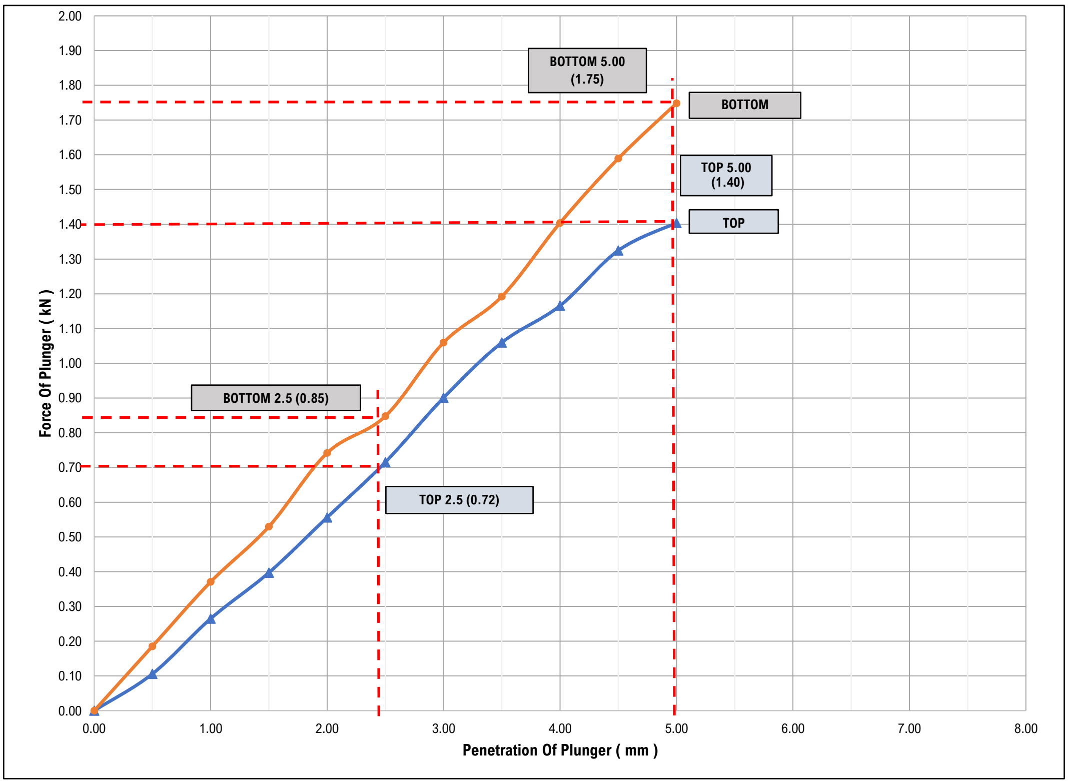
LABORATORY WORKSHEET

Description of material : **LATERITE**
BORROW PIT

Date test : **14.07.2022**

SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4



Plunger Penetration (mm)	Top	
	C.B.R (%)	M/C (%)
2.50	**5.42**	**28.96**
5.00	**7.02**	

Plunger Penetration (mm)	Bottom	
	C.B.R (%)	M/C (%)
2.50	**6.42**	**29.26**
5.00	**8.74**	

Remarks : * Specification Limits from JKR's Design Specification is not less than 12%

Highest Value in this material 8.74%

Remarks :

Description of material : **LATERITE**
Source : **BORROW PIT**

Date test : **14.07.2022**

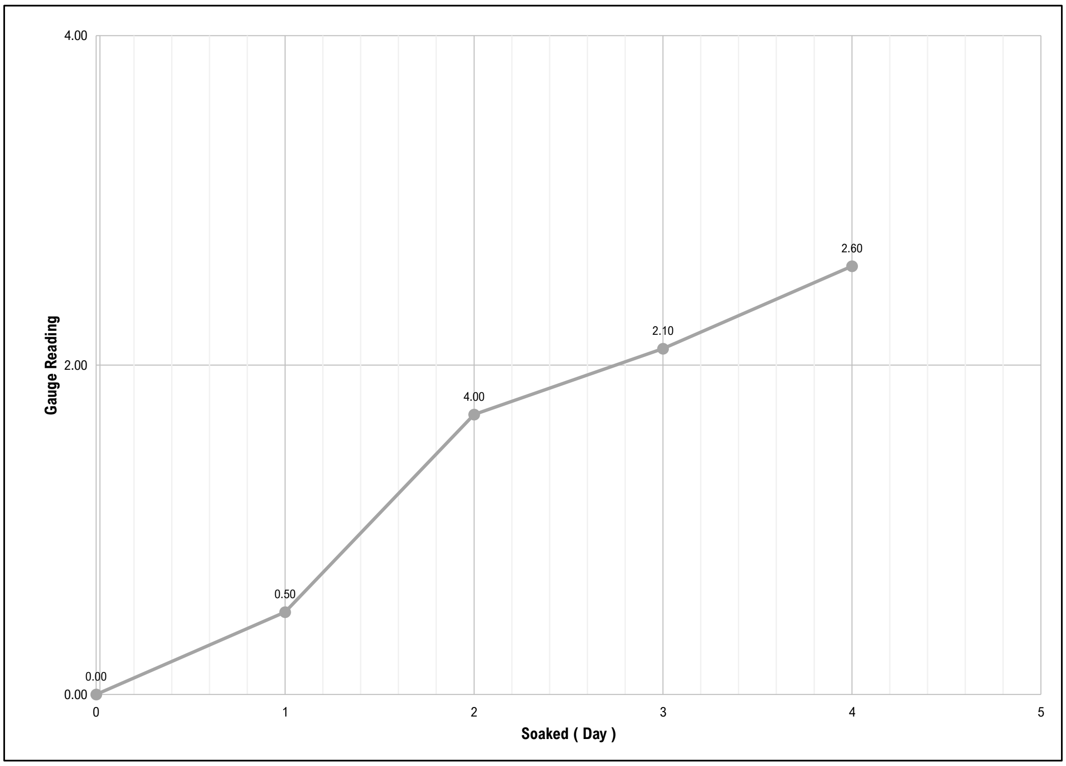
SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

DATE	DAY	TIME	PERIOD		Reading (mm)	REMARKS
14.07.2022	THURSDAY	1.40 PM	0		0.00	FULLY WATER
15.07.2022	FRIDAY	1.40 PM	1		0.50	-
16.07.2022	SATURDAY	1.40 PM	2		1.70	-
17.07.2022	SUNDAY	1.40 PM	3		2.10	-
15/07/1940	MONDAY	1.40 PM	4		2.60	-

Remarks:

C.B.R.(SWELL CHART)



Description of material :	**LATERITE**	Date sample :	**03.07.2022**
		Date test :	**14.07.2022**

CALIFORNIA BEARING RATIO (CBR) SOAKED 95% MDD

BS 1377 PART 4 /

JKR/SPJ/2008 - S4

A. Preparation Procedure :

	Compresion with tamping
	Compresion in layers
	Rammer compaction to a spec. density
X	Vibrating compaction to a spec. density
	Rammer compaction with specified effort

B. Compaction Data :

1.Max Dry Density (MDD)	1.7140	Mg/m^3
2.Optimum Moisture Content	20.38	%
3.Specified Density at 95%	1.6283	Mg/m^3
4.Volume of Mould	2305	cm^3
5.Specified Effort	5	layers
6.Blow per layer By Vibrating Hammer	1	minit
7.Calculated mass of soil required (m) = $\frac{V}{100}$ * (100 + W) pd	4518	g
8.Soaking Time	4	days
9.Final Swelling	2.3	mm
10.Water to be added	921	gm

Note: V = Vol Of Mould
W = Moisture Content Of Sample
pd = Selected Density Of 95%

Description of material : **LATERITE + 3% CEMENT**

Date test : **14.07.2022**

SOAKED CBR TEST

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

Density Before Soaking		
Mould no		2
Wt Of mould + wet sample	gm	12260
Wt of mould,	gm	7290
Wt of sample	gm	4970
Volume of mould	cm^3	2305
Wet density	mg/m^3	2.156
Average moisture content	%	20.23
Dry density	mg/m^3	1.7140

Moisture Content (Before Soaked)			
Container No			
Wt of cont. + wet sample	gm	**101.30**	**96.20**
Wt of cont. + dry sample	gm	**86.50**	**82.80**
Wt. of container	gm	**15.00**	**15.00**
Wt. of water	gm	**14.80**	**13.40**
Wt. of dry sample	gm	**71.50**	**67.80**
Moisture content	%	**20.70**	**19.76**
Av.Moisture content	%	**20.23**	
Proving ring constant		0.02649 (Div) kNf	

Penetration	Dial Gauge		Force		Corrected	
	Top	**Bottom**	**Top**	**Bottom**	Top	Bottom
0.00	0	0	0.00	0.00		
0.50	11	17	0.29	0.45		
1.00	18	24	0.48	0.64		
1.50	27	33	0.72	0.87		
2.00	35	48	0.93	1.27		
2.50	**47**	**52**	**1.25**	**1.38**		
3.00	54	60	1.43	1.59		
3.50	60	75	1.59	1.99		
4.00	74	83	1.96	2.20		
4.50	80	90	2.12	2.38		
5.00	**93**	**106**	**2.46**	**2.81**		
5.50						
6.00						
6.50						
7.00						
7.50						

Expansion & Absorption Determination		
Surcharge	kg	**4.5**
Gauge reading after soaking	Div	**2.30**
Gauge reading before soaking	Div	**0.0000**
Total expansion	mm	**2.3**
Wt. Of mould + sample after soaking	gm	**12305**
Wt. Of mould + sample before soaking	gm	**12260**
Wt. of water absortion	gm	**45.0**

Moisture Content (After Soaked)		**Top**	**Bottom**
Container No			
Wt of cont. + wet sample	gm	**98.50**	**101.80**
Wt of cont. + dry sample	gm	**83.30**	**85.30**
Wt. of container	gm	**15.00**	**15.00**
Wt. of water	gm	**15.20**	**16.50**
Wt. of dry sample	gm	**68.30**	**70.30**
Moisture content	%	**22.25**	**23.47**

Period of soaking	4 days
Method of campaction	-

5 layers Method Of Compaction : Vabrating Hammer Time/Duratoin 1 min

Remarks :

LABORATORY WORKSHEET

APPENDIX 2 (G)

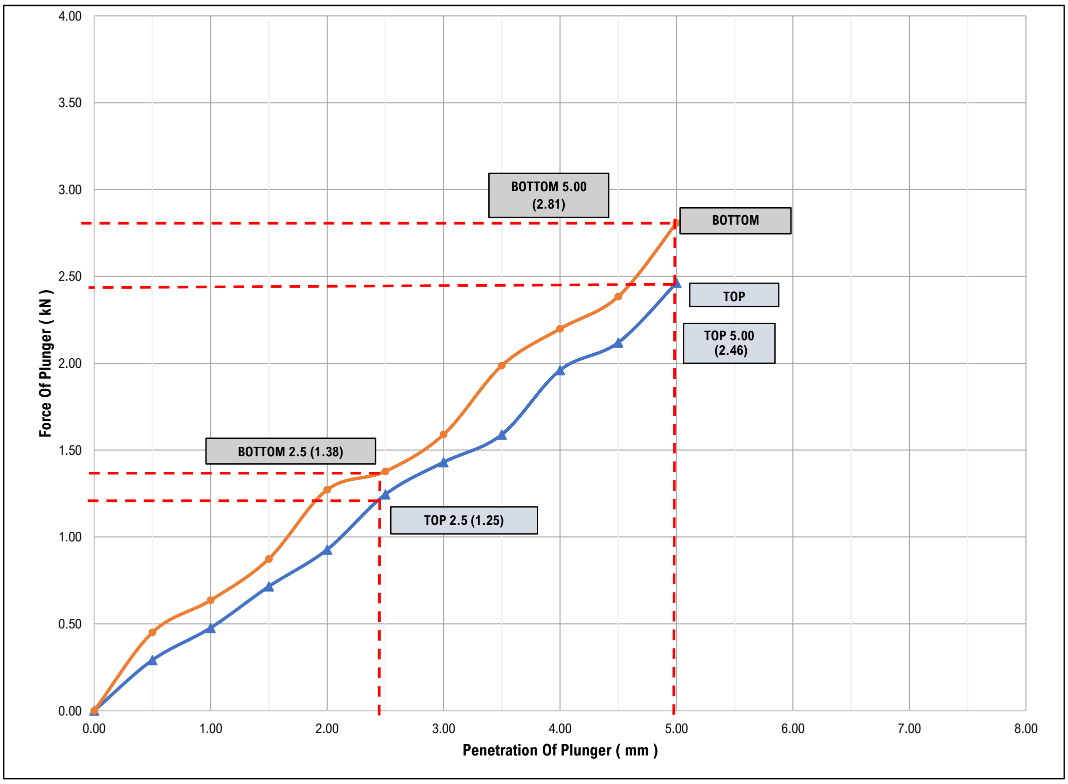
Description of material : **LATERITE + 3% CEMENT**

Date test : **14.07.2022**

Page : **2 of 3**

SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4



Plunger Penetration (mm)	Top C.B.R (%)	Top M/C (%)
2.50	**9.43**	**22.25**
5.00	**12.32**	

Plunger Penetration (mm)	Bottom C.B.R (%)	Bottom M/C (%)
2.50	**10.44**	**23.47**
5.00	**14.04**	

Remarks : * Specification Limits from JKR's Design Specification is not less than 12%

Highest Value in this material 14.04%

Remarks :

APPENDIX 2 (G)

LABORATORY WORKSHEET

Description of material : **LATERITE + 3% CEMENT**

Date test : **14.07.2022**
Page : **3 of 3**

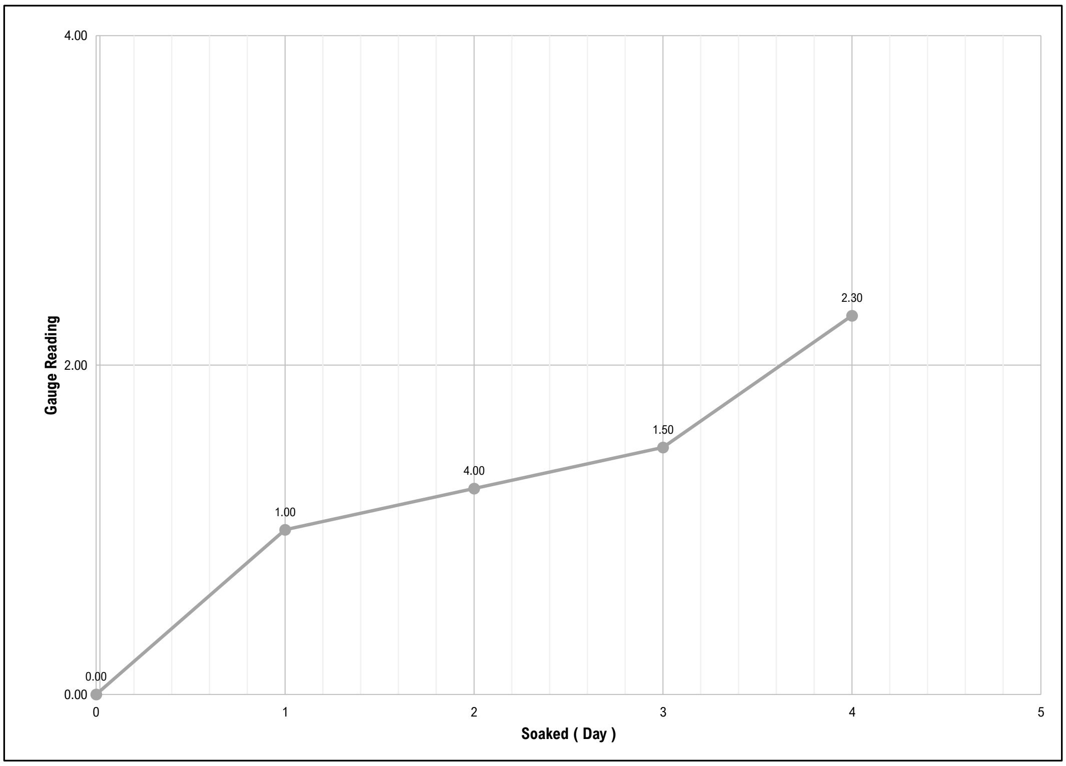
SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

DATE	DAY	TIME	PERIOD		Reading (mm)	REMARKS
14.07.2022	THURSDAY	2.40 PM	0		0.00	FULLY WATER
15.07.2022	FRIDAY	2.40 PM	1		1.00	-
16/01/1900	SATURDAY	2.40 PM	2		1.25	-
17.07.2022	SUNDAY	2.40 PM	3		1.50	-
18.07.2022	MONDAY	2.40 PM	4		2.30	-

Remarks:

C.B.R.(SWELL CHART)



APPENDIX 2 (G)

Description of material : **LATERITE + 6% CEMENT**

Date sample : **03.07.2022**

Date test : **14.07.2022**

CALIFORNIA BEARING RATIO (CBR) SOAKED 95% MDD

BS 1377 PART 4 /

JKR/SPJ/2008 - S4

A. Preparation Procedure :

	Compresion with tamping
	Compresion in layers
	Rammer compaction to a spec. density
X	Vibrating compaction to a spec. density
	Rammer compaction with specified effort

B. Compaction Data :

1.Max Dry Density (MDD)	1.7670	Mg/m^3
2.Optimum Moisture Content	19.58	%
3.Specified Density at 95%	1.6787	Mg/m^3
4.Volume of Mould	2305	cm^3
5.Specified Effort	5	layers
6.Blow per layer By Vibrating Hammer	1	minit
7.Calculated mass of soil required (m) = $\frac{V}{100}$ * (100 + W) pd	4627	g
8.Soaking Time	4	days
9.Final Swelling	1.8	mm
10.Water to be added	906	gm

Note:
V = Vol Of Mould
W = Moisture Content Of Sample
pd = Selected Density Of 95%

Description of material : **LATERITE + 6% CEMENT** Date test : **14.07.2022**
Sources: **BORROW PIT** Page : **1 of 3**

SOAKED CBR TEST

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

Density Before Soaking		
Mould no		9
Wt Of mould + wet sample	gm	12320
Wt of mould,	gm	7290
Wt of sample	gm	5030
Volume of mould	cm^3	2305
Wet density	mg/m^3	2.182
Average moisture content	%	19.00
Dry density	mg/m^3	1.7670

Moisture Content (Before Soaked)			
Container No			
Wt of cont. + wet sample	gm	**163.40**	**163.40**
Wt of cont. + dry sample	gm	**142.00**	**142.10**
Wt. of container	gm	**29.70**	**29.70**
Wt. of water	gm	**21.40**	**21.30**
Wt. of dry sample	gm	**112.30**	**112.40**
Moisture content	%	**19.06**	**18.95**
Av.Moisture content	%	**19.00**	
Proving ring constant		0.02649 (Div) kNf	

Penetration	Dial Gauge		Force		Corrected	
	Top	Bottom	Top	Bottom	Top	Bottom
0.00	0	0	0.00	0.00		
0.50	20	35	0.53	0.93		
1.00	35	50	0.93	1.32		
1.50	55	75	1.46	1.99		
2.00	73	93	1.93	2.46		
2.50	**90**	**125**	**2.38**	**3.31**		
3.00	124	140	3.28	3.71		
3.50	138	160	3.66	4.24		
4.00	157	174	4.16	4.61		
4.50	166	186	4.40	4.93		
5.00	**182**	**206**	**4.82**	**5.46**		
5.50						
6.00						
6.50						
7.00						
7.50						

Expansion & Absorption Determination		
Surcharge	kg	**4.5**
Gauge reading after soaking	Div	**1.80**
Gauge reading before soaking	Div	**0.0000**
Total expansion	mm	**1.8**
Wt. Of mould + sample after soaking	gm	**12355**
Wt. Of mould + sample before soaking	gm	**12320**
Wt. of water absortion	gm	**35.0**

Moisture Content (After Soaked)		Top	Bottom
Container No			
Wt of cont. + wet sample	gm	**39.90**	**39.90**
Wt of cont. + dry sample	gm	**35.30**	**35.20**
Wt. of container	gm	**15.20**	**15.20**
Wt. of water	gm	**4.60**	**4.70**
Wt. of dry sample	gm	**20.10**	**20.00**
Moisture content	%	**22.89**	**23.50**

Period of soaking	4 days
Method of campaction	-

5 layers Method Of Compaction : Vabrating Hammer Time/Duratoin 1 min

Remarks :

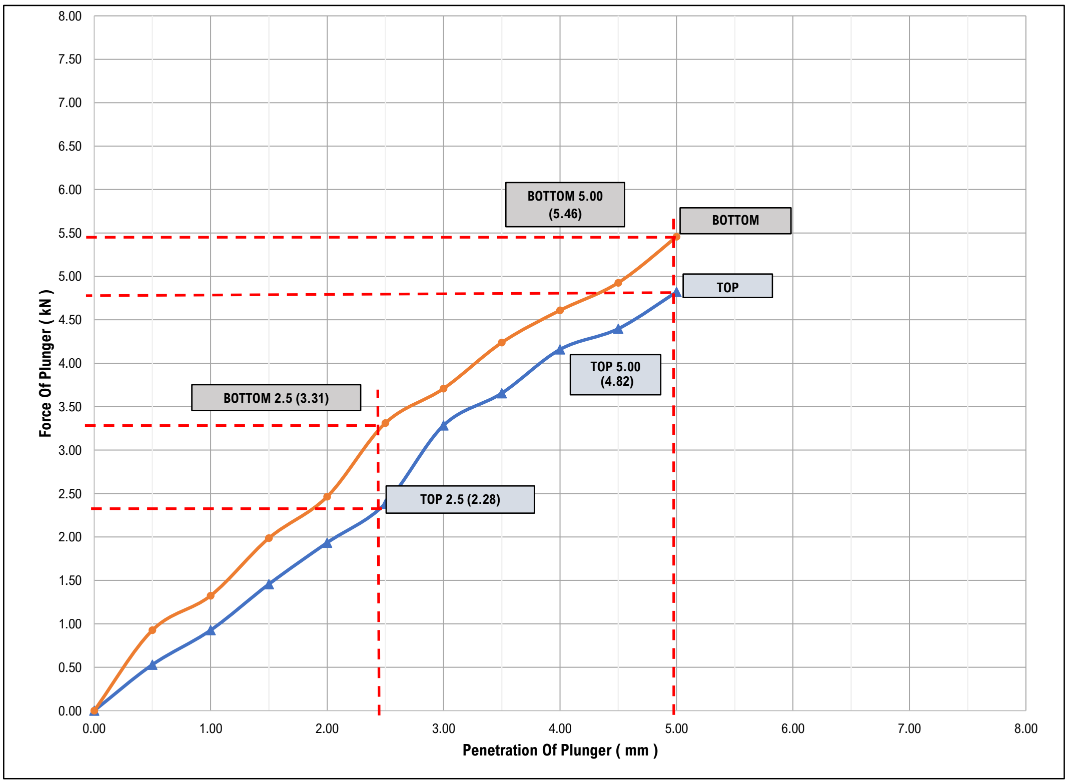
LABORATORY WORKSHEET

APPENDIX 2 (G)

Description of material :	**LATERITE + 6% CEMENT**	Date test :	**14.07.2022**
Sources:	**BORROW PIT**	Page :	**2 of 3**

SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4



Plunger Penetration (mm)	Top	
	C.B.R (%)	M/C (%)
2.50	**18.06**	**22.89**
5.00	**24.11**	

Plunger Penetration (mm)	Bottom	
	C.B.R (%)	M/C (%)
2.50	**25.09**	**23.50**
5.00	**27.28**	

Remarks : * Specification Limits from JKR's Design Specification is not less than 12%

Highest Value in this material 27.28%

Remarks :

APPENDIX 2 (G)

LABORATORY WORKSHEET

Description of material : **LATERITE + 6% CEMENT**
Source : **BORROW PIT**

Date test : **14.07.2022**
Page : **3 of 3**

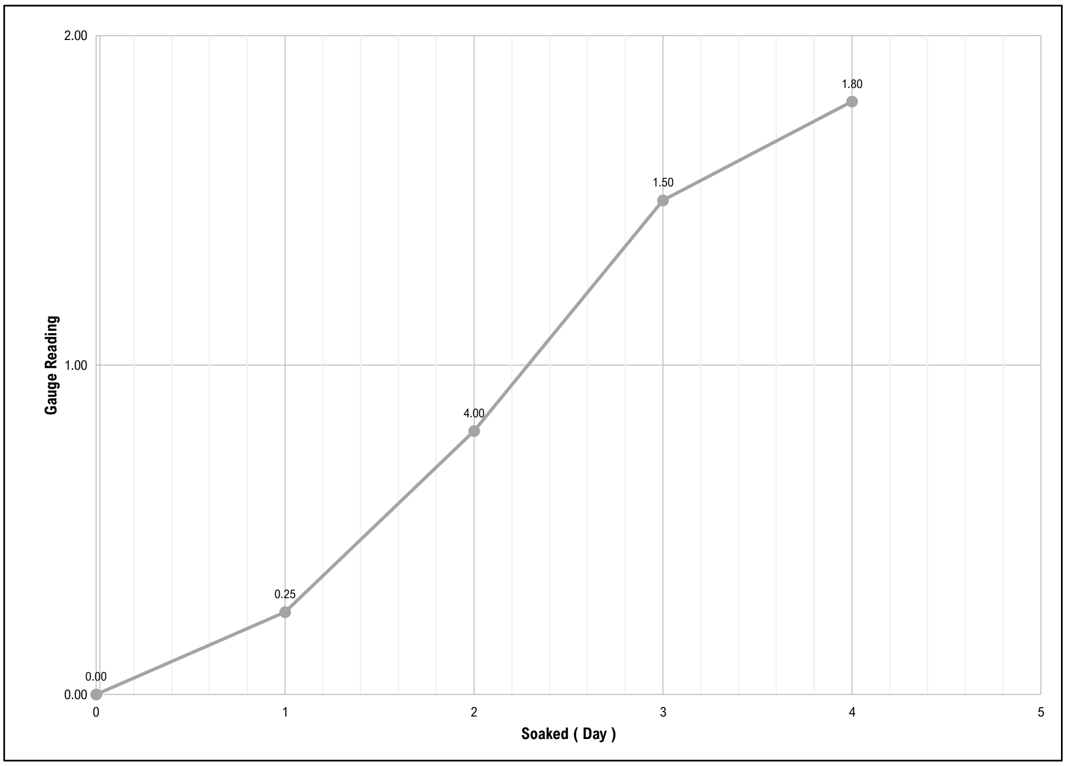
SOAKED CBR TEST GRAPH

BS 1377: Part 4: 1990 / JKR/SPJ/2008 - S4

DATE	DAY	TIME	PERIOD		Reading (mm)	REMARKS
14.07.2022	THURSDAY	4.00 PM	0		0.00	FULLY WATER
15.07.2022	FRIDAY	4.00 PM	1		0.25	-
16.07.2022	SATURDAY	4.00 PM	2		0.80	-
17.07.2022	SUNDAY	4.00 PM	3		1.50	-
18.07.2022	MONDAY	4.00 PM	4		1.80	-

Remarks:

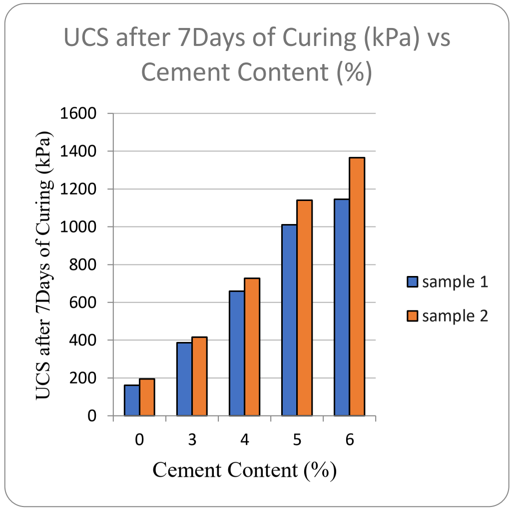
C.B.R.(SWELL CHART)



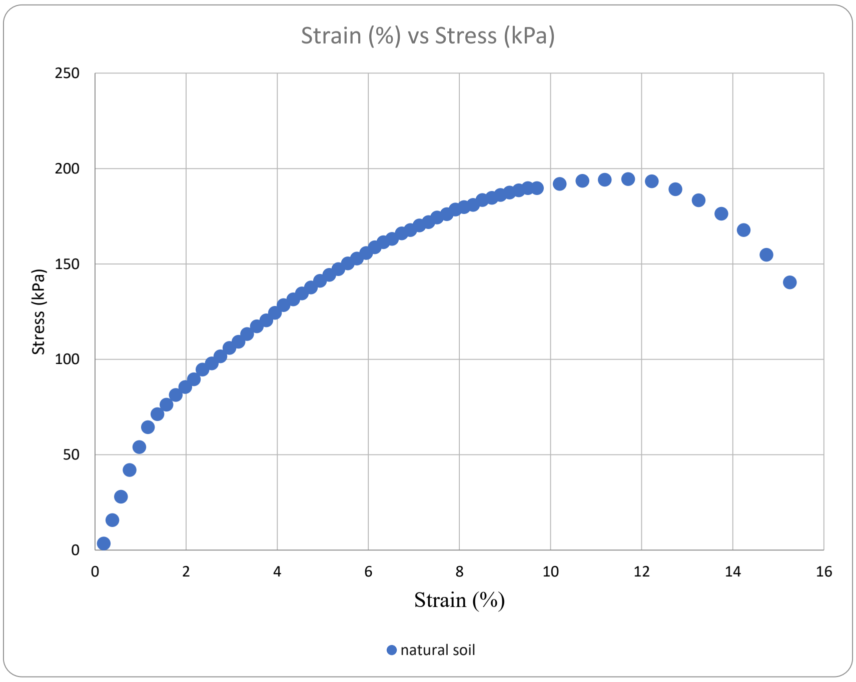
	strength after curing 7 days, kPa			Average, qu (kF	axial strain (%)			Average axial strain (%)	dry density (Mg/m3)			Average dry density (Mg/m3)
	sample 1	sample 2	sample 3		sample 1	sample 2	sample 3		sample 1	sample 2	sample 3	
0	161	195	186	180.67	11.26	11.7	9.89	10.95	2.3	1.94	1.95	2.06
4	659	728	716	701.00	1.8	3.01	2.36	2.39	2.02	1.96	2.04	2.01
5	1010	1140	1192	1114.00	2.17	1.95	3.54	2.55	1.9	1.92	2.05	1.96
6	1365	1384	1145	1298.00	2.79	3.56	2.57	2.97	1.96	1.94	1.94	1.95

Cement Content (%)	Sample 2	Dry Density (Mg/m3)	Axial Strain (%)
0	195	1.94	11.7
3	416	1.93	2.93
4	728	1.96	3.01
5	1140	1.92	1.95
6	1365	1.96	2.79

Cement Content (%)	Compresive strength, qu after curing 7 days, kPa	
	sample 1	sample 2
0	161	195
3	386	416
4	659	728
5	1010	1140
6	1145	1365



Unconfined Compressive Strength Result of Soil Sample									
Control Sample		3% Cement		4% Cement		5% Cement		6% Cement	
Strain (%)	Stress (kPa)	Strain (%)	Stress (kPa)	Strain (%)	Stress (kPa)	Strain (%)	Stress (kPa)	Strain (%)	Stress (kPa)
0	-5.3	0	0	0	0	0	0	0	0
0.19	3.5	0.2	10.6	0.21	6.2	0.19	58	0.19	7
0.38	15.8	0.39	54.1	0.42	54.4	0.38	238.8	0.4	121.1
0.57	28	0.58	90.8	0.63	138.3	0.57	450.3	0.61	279
0.76	42	0.77	129.4	0.82	222.5	0.77	627.8	0.82	464.9
0.97	54.1	0.96	158.8	1.03	310.5	0.97	773.6	1.01	631.5
1.16	64.4	1.16	191.6	1.24	398.6	1.16	897	1.2	784.9
1.37	71.3	1.36	228.2	1.45	482.8	1.35	991.4	1.41	917.3
1.57	76.3	1.55	263.7	1.64	549.5	1.54	1065	1.6	1030
1.77	81.4	1.74	301.3	1.85	582.9	1.75	1115.5	1.79	1126.7
1.98	85.5	1.94	337	2.04	635.3	1.95	1140.5	2	1208.1
2.17	89.6	2.14	362.2	2.23	673.7	2.15	1139.8	2.19	1273.8
2.36	94.6	2.34	385.5	2.44	691.7	2.34	1117.4	2.39	1324.5
2.56	97.9	2.54	401.9	2.63	700.1	2.54	1075.1	2.59	1357
2.75	101.6	2.74	412.2	2.82	719.3	2.74	1007.4	2.79	1365.3
2.95	106	2.93	415.7	3.01	728.1	2.95	885.9	2.99	1333.5
3.15	109.2	3.14	406.3	3.2	718.1			3.2	1276
3.34	113.3	3.33	397.5	3.39	689.5			3.39	1253
3.55	117.3	3.54	392.7	3.59	635.1			3.6	1226.5
3.76	120.4	3.73	389.9					3.8	1147.7
3.95	124.4	3.92	386.1					4	1055.6
4.14	128.4							4.19	995.3
4.35	131.5							4.38	945.3
4.54	134.6								
4.74	137.7								
4.94	141.2								
5.14	144.3								
5.34	147.3								
5.55	150.3								
5.75	152.8								
5.95	155.8								
6.14	158.8								
6.33	161.4								
6.52	163.1								
6.73	166								
6.92	167.8								
7.12	170.2								
7.32	172								
7.51	174.4								
7.72	176.1								
7.91	178.5								
8.1	179.8								
8.3	181								
8.5	183.5								
8.71	184.7								
8.9	186.2								
9.1	187.4								
9.3	188.6								
9.5	189.8								
9.7	189.8								
10.2	192								
10.7	193.6								
11.19	194.1								
11.7	194.5								
12.22	193.4								
12.74	189.2								
13.25	183.4								
13.75	176.3								
14.24	167.8								
14.74	154.8								
15.25	140.4								



LABORATORY WORKSHEET

APPENDIX 2 (H)

MATERIAL :LATERITE

Date test : **25/07/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	22/07/2022	3	8020.0	15.0	15.0	15.0	3375.0	225.0	2.376		36.7	1.63		S
2	2	22/07/2022	3	7995.0	15.0	15.0	15.0	3375.0	225.0	2.369		40.5	1.80		S
3	3	22/07/2022	3	7990.0	15.0	15.0	15.0	3375.0	225.0	2.367	2.371	38.5	1.71	**1.71**	S
1	4	22/07/2022	7	7885.0	15.0	15.0	15.0	3375.0	225.0	2.336		44.9	2.00		S
2	5	22/07/2022	7	7900.0	15.0	15.0	15.0	3375.0	225.0	2.341		46.5	2.07		S
3	6	22/07/2022	7	8010.0	15.0	15.0	15.0	3375.0	225.0	2.373	2.350	46.0	2.04	**2.04**	S

Note :
- 1) delete as appropriate
- 2) determine by client
- 3) measured nearest to milimeter
- 4) measured nearest to 10kg / m^3
- 5) measured nearest to 0.1 N / mm^2
- S = Satisfactory Failures / U = Unsatisfactory Failures

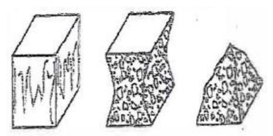
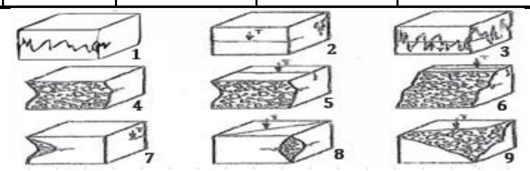


Figure : Satisfactory failure



Remarks :

LABORATORY WORKSHEET

APPENDIX 2 (H)

MATERIAL : LATERITE + 3 % CEMENT

Date test : **25/07/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	22/07/2022	3	7720.0	15.0	15.0	15.0	3375.0	225.0	2.287		75.6	3.36		S
2	2	22/07/2022	3	7860.0	15.0	15.0	15.0	3375.0	225.0	2.329		80.5	3.58		S
3	3	22/07/2022	3	7880.0	15.0	15.0	15.0	3375.0	225.0	2.335	2.317	81.4	3.62	**3.52**	S
1	4	22/07/2022	7	8050.0	15.0	15.0	15.0	3375.0	225.0	2.385		95.8	4.26		S
2	5	22/07/2022	7	8030.0	15.0	15.0	15.0	3375.0	225.0	2.379		99.5	4.42		S
3	6	22/07/2022	7	8011.0	15.0	15.0	15.0	3375.0	225.0	2.374	2.379	94.5	4.20	**4.29**	S

Note :

1) delete as appropriate
2) determine by client
3) measured nearest to milimeter
4) measured nearest to 10kg / m^3
5) measured nearest to 0.1 N / mm^2

S = Satisfactory Failures / U = Unsatisfactory Failures

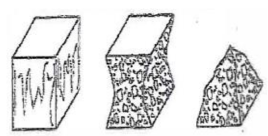
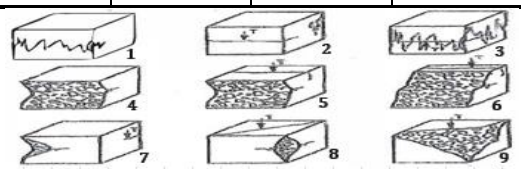


Figure : Satisfactory failure



Remarks :

LABORATORY WORKSHEET

APPENDIX 2 (H)

MATERIAL : LATERITE + 6 % CEMENT

Date test : **25/07/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	22/07/2022	3	8070.0	15.0	15.0	15.0	3375.0	225.0	2.391		95.8	4.26		S
2	2	22/07/2022	3	8060.0	15.0	15.0	15.0	3375.0	225.0	2.388		101.5	4.51		S
3	3	22/07/2022	3	8000.0	15.0	15.0	15.0	3375.0	225.0	2.370	2.383	100.0	4.44	**4.40**	S
1	4	22/07/2022	7	8110.0	15.0	15.0	15.0	3375.0	225.0	2.403		125.0	5.56		S
2	5	22/07/2022	7	8150.0	15.0	15.0	15.0	3375.0	225.0	2.415		128.0	5.69		S
3	6	22/07/2022	7	8170.0	15.0	15.0	15.0	3375.0	225.0	2.421	2.413	127.5	5.67	**5.64**	S

Note :	
	[1] delete as appropriate
	[2] determine by client
	[3] measured nearest to milimeter
	[4] measured nearest to 10kg / m^3
	[5] measured nearest to 0.1 N / mm^2
	S = Satisfactory Failures / U = Unsatisfactory Failures

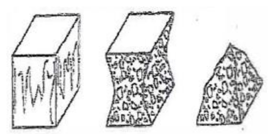
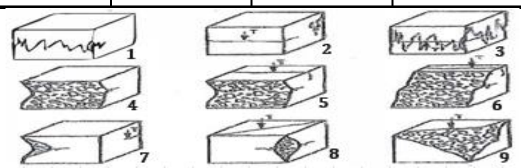


Figure : Satisfactory failure



Remarks :

LABORATORY WORKSHEET

APPENDIX 2 (H)

MATERIAL : LATERITE + 9 % CEMENT

Date test : **25/07/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample					Dimension (cm)			Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	22/07/2022	3	8150.0	15.0	15.0	15.0	3375.0	225.0	2.415		145.0	6.44		S
2	2	22/07/2022	3	8180.0	15.0	15.0	15.0	3375.0	225.0	2.424		148.0	6.58		S
3	3	22/07/2022	3	8192.0	15.0	15.0	15.0	3375.0	225.0	2.427	2.422	151.0	6.71	6.58	S
1	4	22/07/2022	7	8230.0	15.0	15.0	15.0	3375.0	225.0	2.439		168.0	7.47		S
2	5	22/07/2022	7	8210.0	15.0	15.0	15.0	3375.0	225.0	2.433		166.0	7.38		S
3	6	22/07/2022	7	8200.0	15.0	15.0	15.0	3375.0	225.0	2.430	2.434	165.0	7.33	7.39	S

Note :
- 1) delete as appropriate
- 2) determine by client
- 3) measured nearest to milimeter
- 4) measured nearest to 10kg / m^3
- 5) measured nearest to 0.1 N / mm^2
- S = Satisfactory Failures / U = Unsatisfactory Failures

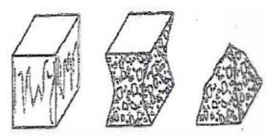
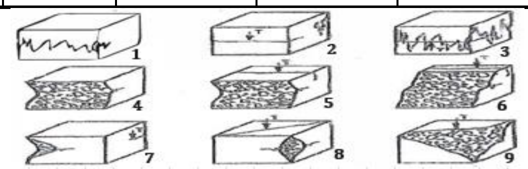


Figure : Satisfactory failure



Remarks :

LABORATORY WORKSHEET

APPENDIX 2 (H)

MATERIAL : LATERITE + 12 % CEMENT

Date test : **25/07/2022**

UNCONFINED COMPRESSIVE STRENGTH

BS EN 12390-3:2001 / JKR/SP J/2008-S4

Preparation sample				Dimension (cm)				Sample Parameters				Testing of sample			
No	Sample Ref.	Date Cast	Age (days)	Weight of Sample (g)	Length	Width	Height	Volume	Area (cm^2)	Density (g/cm^3)	Mean Density (g/cm^3)	Load (kN)	Strength (N/mm^2)	Mean Strength (N/mm^2)	Type of failure (S / U)
				(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)
									$b \times c \times d$	$\frac{a}{d}$			$i \times \frac{10}{f}$		
1	1	22/07/2022	3	8290.0	15.0	15.0	15.0	3375.0	225.0	2.456		188.0	8.36		S
2	2	22/07/2022	3	8250.0	15.0	15.0	15.0	3375.0	225.0	2.444		182.0	8.09		S
3	3	22/07/2022	3	8240.0	15.0	15.0	15.0	3375.0	225.0	2.441	2.447	180.0	8.00	**8.15**	S
1	4	22/07/2022	7	8310.0	15.0	15.0	15.0	3375.0	225.0	2.462		215.0	9.56		S
2	5	22/07/2022	7	8360.0	15.0	15.0	15.0	3375.0	225.0	2.477		220.0	9.78		S
3	6	22/07/2022	7	8310.0	15.0	15.0	15.0	3375.0	225.0	2.462	2.467	260.0	11.56	**10.30**	S

Note :
- [1] delete as appropriate
- [2] determine by client
- [3] measured nearest to milimeter
- [4] measured nearest to 10kg / m^3
- [5] measured nearest to 0.1 N / mm^2
- S = Satisfactory Failures / U = Unsatisfactory Failures

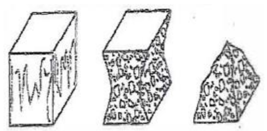
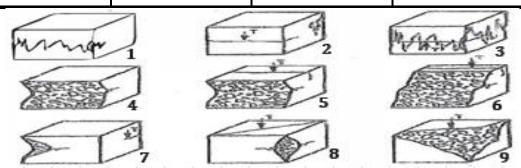


Figure : Satisfactory failure



Remarks :

DATA STRENGTH 3DAY

SOIL	1.71
3%	3.52
6%	4.40
9%	6.58
12%	8.15

DATA STRENGTH 7DAY

SOIL	2.04
3%	4.29
6%	5.64
9%	7.39
12%	10.30

LABORATORY WORKSHEET

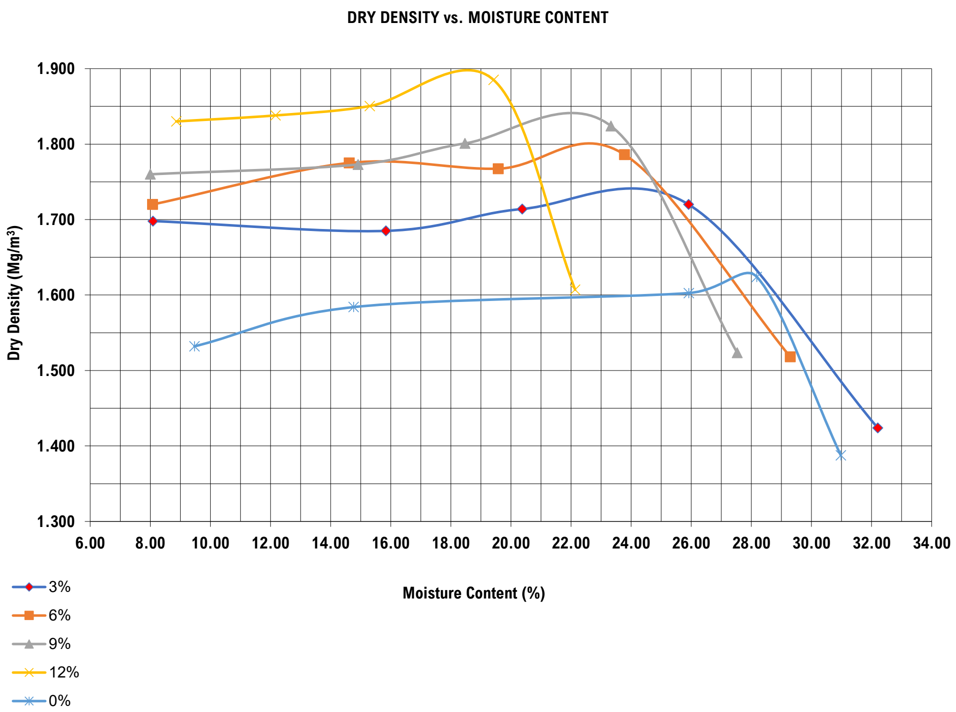
APPENDIX 2 (I)

Description of material : **LATERITE + CEMENT**
Source : **SUNGAI BULOH**

Date sample :
Date test : **22.06.2021**

CHART DRY DENSITY VS MOISTURE CONTENT

CEMENT		1	2	3	4	5
Dry density 0% cement	Mg/m^3	1.532	1.584	1.603	1.624	1.387
Dry density 3% cement	Mg/m^3	1.698	1.685	1.714	1.720	1.424
Dry density 6% cement	Mg/m^4	1.720	1.775	1.767	1.786	1.518
Dry density 9% cement	Mg/m^5	1.760	1.773	1.801	1.824	1.523
Dry density 12% cement	Mg/m^6	1.830	1.838	1.850	1.885	1.607
Moisture content 0%	%	9.470	14.767	25.921	28.182	30.977
Moisture content 3%	%	8.088	15.842	20.377	25.911	32.203
Moisture content 6%	%	8.08	14.62	19.58	23.78	29.29
Moisture content 9%	%	8.00	14.91	18.47	23.33	27.53
Moisture content 12%	%	8.87	12.17	15.30	19.42	22.14



LABORATORY WORKSHEET

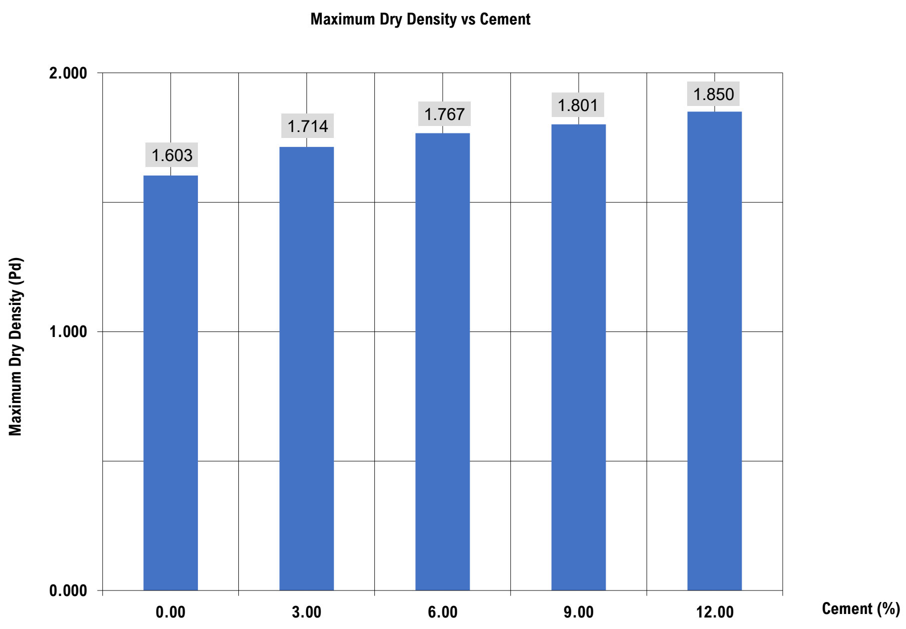
APPENDIX 2 (J)

Description of material : **LATERITE + CEMENT**
Source : **SUNGAI BULOH**

Date sample :
Date test : **22.06.2021**

BAR CHART Max Pd VS % Cement

Maximum Dry Density	**Pd**	**1.603**	**1.714**	**1.767**	**1.801**	**1.850**
Cement	**%**	**0**	**3**	**6**	**9**	**12**



LABORATORY WORKSHEET

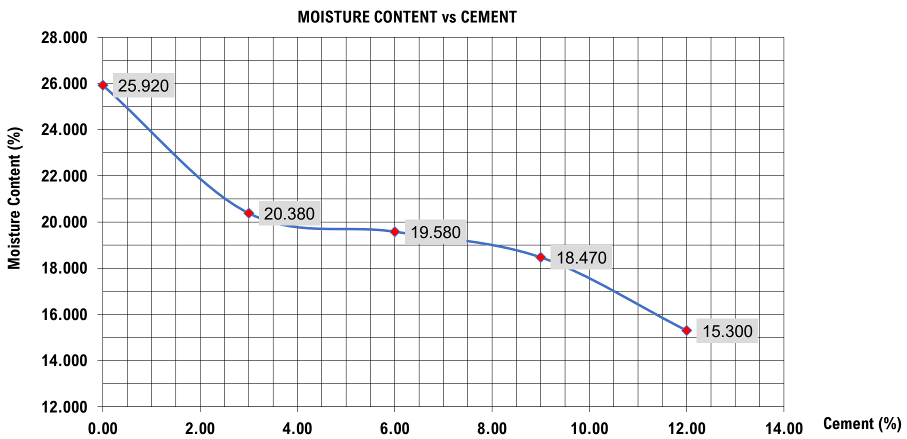
APPENDIX 2 (K)

Description of material : **LATERITE + CEMENT**
Source : **SUNGAI BULOH**

Date sample :
Date test : **22.06.2021**

CHART MOISTURE CONTENT VS CEMENT

Moisture content	**%**	**25.92**	**20.38**	**19.58**	**18.47**	**15.30**
Cement	**%**	**0**	**3**	**6**	**9**	**12**



LABORATORY WORKSHEET

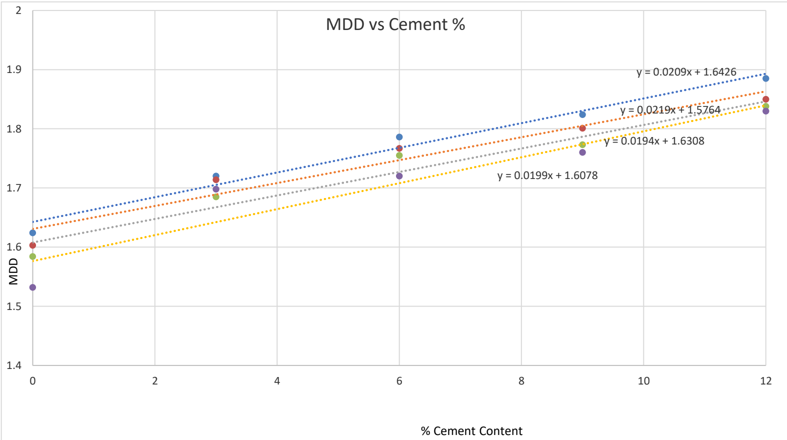
APPENDIX 2 (L)

Description of material : **LATERITE + CEMENT**
Source :

Date sample :
Date test : **22.06.2021**

CHART MAXIMUM DRY DENSITY (MDD) VS PLASTICITY INDEX (PI)

			0%	3%	6%	9%	12%
Max Dry Density	Sample 2: 300mm thick	%	1.624	1.720	1.786	1.824	1.885
Plasticity Index	at Borrow Pit, Sg. Buloh	%	9.590	9.590	9.590	9.590	9.590
Max Dry Density	Sample 1: > 300mm thick	%	1.603	1.714	1.767	1.801	1.850
Plasticity Index	at Borrow Pit, Sg. Buloh	%	12.010	12.010	12.010	12.010	12.010
Max Dry Density	Sample 3 - MMTS	%	1.584	1.685	1.755	1.773	1.838
Plasticity Index	at Borrow Pit, Sg. Buloh	%	16.000	16.000	16.000	16.000	16.000
Max Dry Density	Sample 4 - MMTS	%	1.532	1.698	1.720	1.760	1.830
Plasticity Index	at Borrow Pit, Sg. Buloh	%	18.00	18.00	18.00	18.00	18.00



RESULT AND FINDING TO IDENTIFY CEMENT (Cp) / LIME CONTENT (Lc)

	CATEGORY (PI)	Criteria PI							Equation						% Confident
A	PLASTICITY INDEX (PI)		<	10%			USE	MDD	=	0.0209	Cp	+	1.643		97.5%
B	PLASTICITY INDEX (PI)	10%	<	PI	<	16%	USE	MDD	=	0.0194	Cp	+	1.631		94.4%
C	PLASTICITY INDEX (PI)	16%	<	PI	<	18%	USE	MDD	=	0.0219	Cp	+	1.576		88.7%
D	PLASTICITY INDEX (PI)		>	18%			USE	MDD	=	0.0147	Cp	+	1.623		77.5%

CATEGORY A : **MDD** = **1.75** mg/m3
PI = **10%**

MDD = 0.0209 Cp + 1.643

MDD - 1.643 = 0.0209 Cp

Cp = $\frac{\text{MDD} - 1.643}{0.0209}$

Cp = $\frac{1.750 - 1.643}{0.0209}$

Cp = **5** %

CATEGORY B : **MDD** = **1.75** mg/m3
PI = **13%**

MDD = 0.0194 Cp + 1.631

MDD **-** 1.631 = 0.0194 Cp

$$Cp = \frac{MDD - 1.631}{0.0194}$$

$$Cp = \frac{1.750 - 1.631}{0.0194}$$

Cp = **6** %

CATEGORY C : **MDD** = **1.75** mg/m3
PI = **16%**

MDD = 0.0219 Cp + 1.576

MDD **-** 1.576 = 0.0219 Cp

$$Cp = \frac{MDD - 1.576}{0.0219}$$

$$Cp = \frac{1.750 - 1.576}{0.0219}$$

Cp = **8** %

CATEGORY D : **MDD** = **1.75** mg/m3
PI = **18%**

MDD = 0.0147 Cp + 1.623

MDD **-** 1.623 = 0.0147 Cp

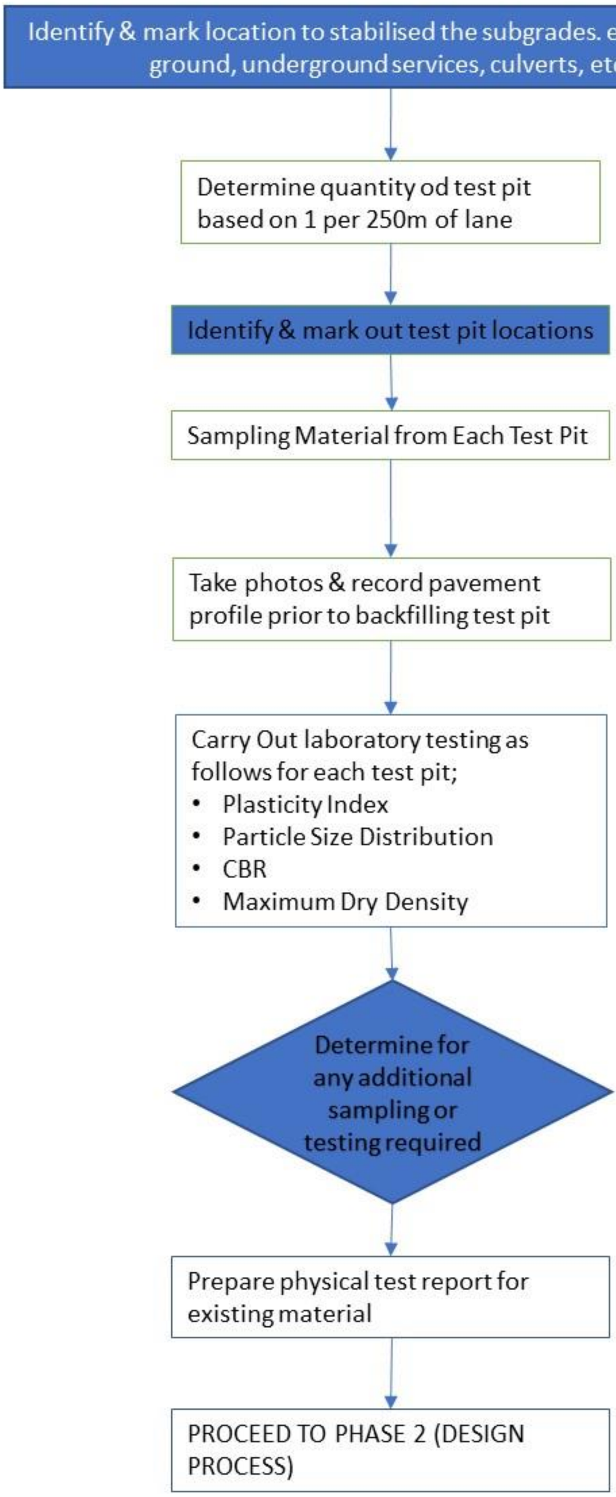
$$Cp = \frac{MDD - 1.623}{0.0147}$$

$$Cp = \frac{1.750 - 1.623}{0.0147}$$

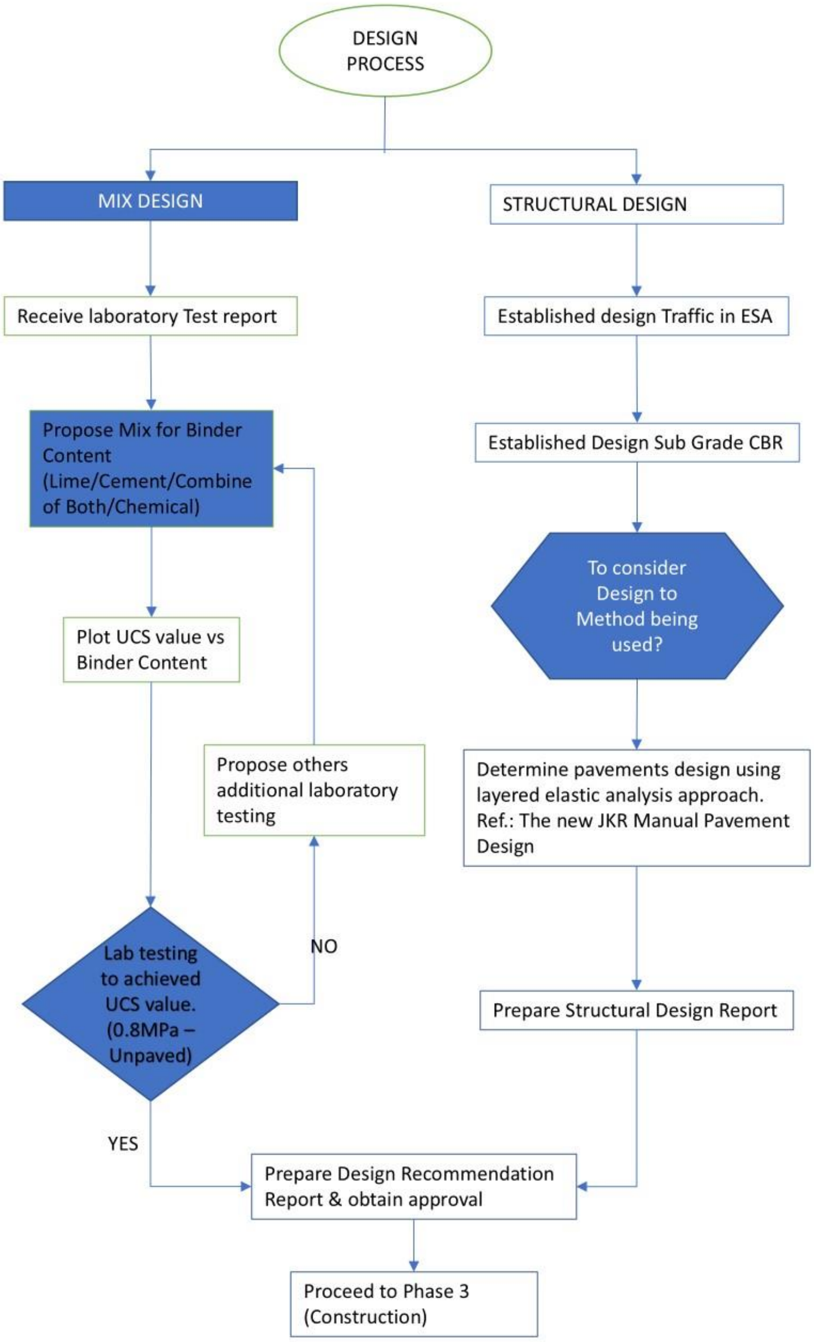
Cp = **9** %

APPENDIX 3
DESIGN AND PERFORMANCE PROCESS FLOW

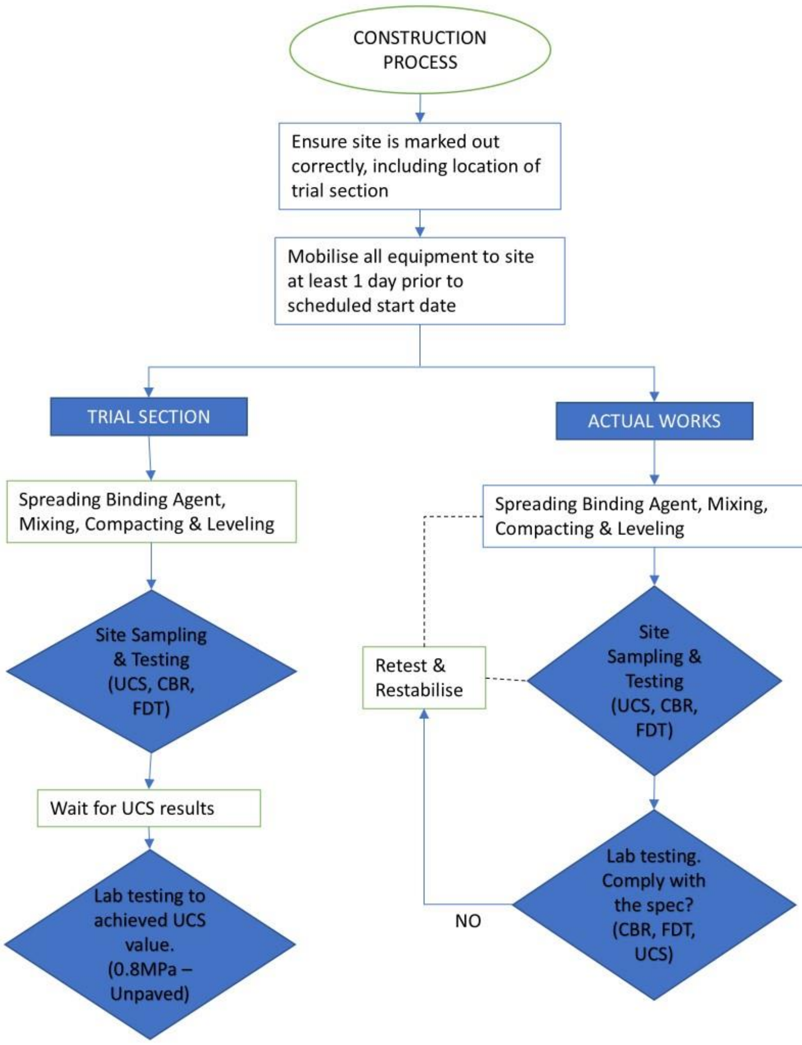
PHASE 1: SITE INVESTIGATION



PHASE 2: DESIGN



PHASE 3: CONSTRUCTION



AUTHOR'S PROFILE

Ghazali Ismail, is Director for SPM Group and General Manager, Operation. He is registered member of Road Engineering Association of Malaysia (REAM), Road Engineering Association of Asia & Australasia (REAAA), Professional Engineer of Association Building Engineer, UK and a member of the Royal Institute Surveyor Malaysia. He holds a MSc. In Facilities management from University Technology Mara, Malaysia and currently pursuing PhD in Civil Engineering, Major in Road Construction at University Technology Mara. Furthermore, he also a registered member of Malaysia Board of Technology (MBoT). He has more than 20 years of professional experience in full spectrum of contact management for road & highway, building, civil & infrastructure and bridges. He is emphasized and high spirit to bring up the innovation technologies to be adopted at our country for a better quality, performance, cost & time saving.

Experienced in infrastructure work locally and internationally, he is passionate in ability to provide the most effective and efficient solutions into road construction/rehabilitation industries. He strongly believed the innovation technologies especially in stabilisation method is a sunshine technology. Actively involved with CIDB, JKR, REAM, CREaTE, RISM and SIRIM Malaysia especially for knowledge sharing on technical and operation referring to Standard and Specification. Background about recent company with Stabilised Pavements (Malaysia), which a global and international construction company from Australia, specialist in road and pavement recycling, soil stabilisation, related engineering services, construction and maintenance with RM100 million in annual revenue and more than 500 employees.