

University of Southern Queensland
Faculty of Engineering and Surveying

A Study of Building Foundations in Malaysia

A dissertation submitted by

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Abstract

Malaysia is a developing country located in South-East Asia. The demands for buildings, for commercial or residential use, are high especially in Kuala Lumpur, the capital city of Malaysia. A building is designed to support the load applied to it safely. The ability of building to sustain the applied loads depends on the building's foundation system.

Foundation is an important part of every building, which interfaces the superstructures to the adjacent soil or rock below it. The superstructure loads will be transferred to the underlying soil or rock. Without a proper design and construction of foundation, problems such as cracking, settlement of building may occur and even to the extent, the whole building may collapse within its design life. Therefore, a proper foundation system is required to maintain the safeness of a building.

This research project can be separated into two main parts that are the project literature reviews and case studies. The literature reviews involve a comprehensive study on the soil available in Kuala Lumpur, the types of foundation system currently used, the factors governing the selection of a safe and economical foundation system, the design approaches adopted by the design engineers and lastly, the problems related to the foundation system.

The second part of this project involves the discussion of five case studies conducted in Kuala Lumpur area. These cases are about the development of medium and high rise buildings for commercial or residential purpose. The discussion comprises of the site conditions, selection of foundation system, the foundation system used and also some of the main issue in the cases. At later stage, comparisons between these case studies are made. Conclusions are drawn from the literature reviews and case studies conducted.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

Malaysia is a developing country situated in the South East Asia region. The demands for building for commercial and residential use are high in Malaysia, particularly in the states of Kuala Lumpur. The location map of Malaysia and Kuala Lumpur state can be seen in Figure 1.1 and Figure 1.2 respectively. A building is designed to support the load applied to it safely. Therefore, to ensure that the building is able to perform its function satisfactorily, the foundation system of that building must be safe to support the applied weight or loads.

Foundation is an important part of any building as it acts as a medium to transmit the loads to the soil or rock below it. Without a proper design, problems such as cracking, settlement of building and to the extent, the whole building may collapse within the design life.



(Source: The World Factbook, 2005)

Figure 1.1: Malaysia Location Map

Normally in Malaysia, the design life of foundation is taken as 50 years and basic service of 10 years. After a period of 10 years, an assessment of the building condition is necessary to ensure the continuing safe use of that building.

The selection of foundation is the most important part of the design process and most difficult to define because the selection is governed by many factors such as soil condition, past site usage, adjacent construction, size of development and also the cost. (Curtin et. al, 1994)

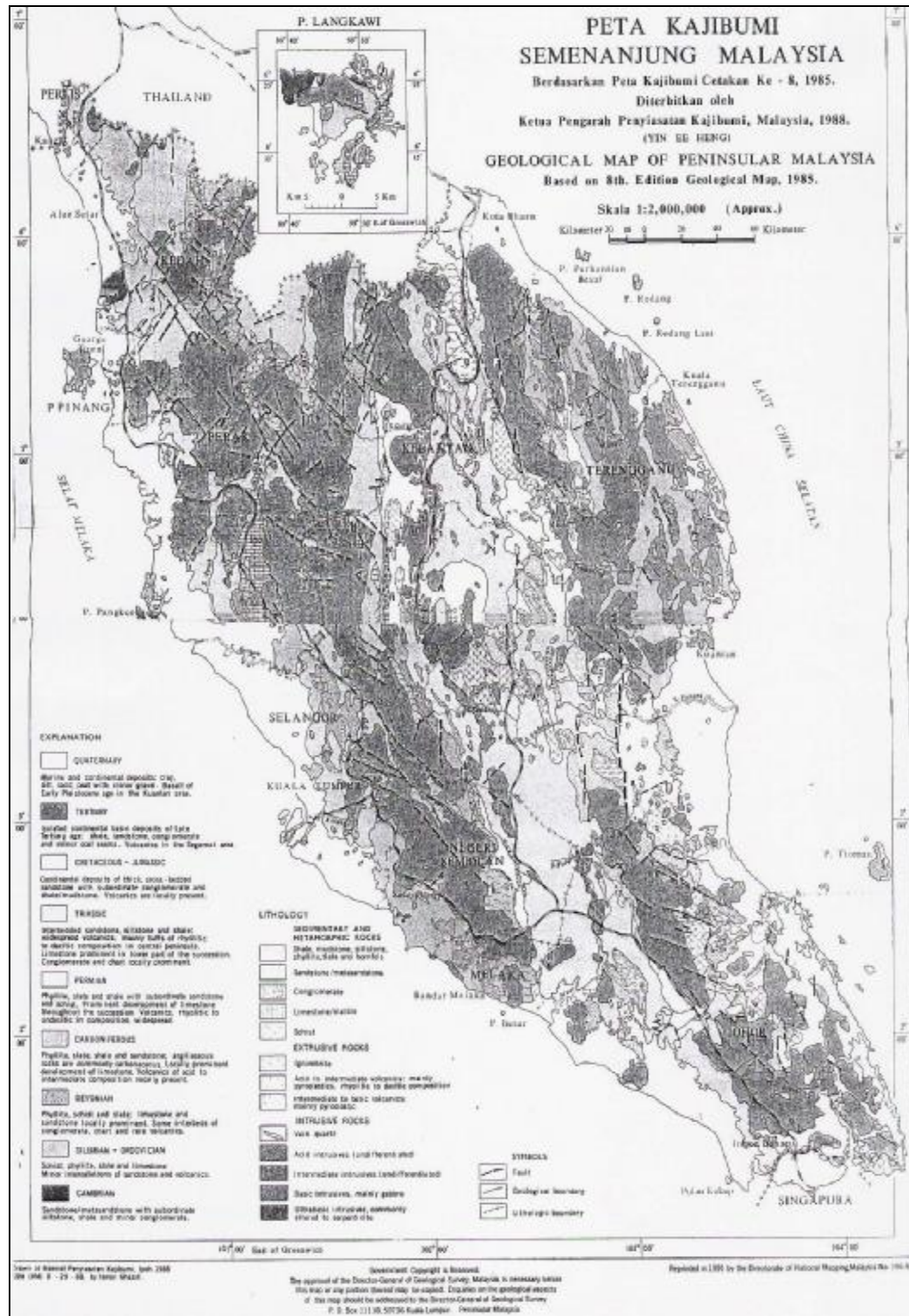


(Source: Castaneda Library, 2004)

Figure 1.2: The States in Malaysia

The ground or soil condition is one of the important factors in foundation design. Before starting to design a foundation, a civil engineer must first obtain the required soil information from soil investigation carried out at the proposed site. Therefore, he or she must have an adequate knowledge on the properties of soil and also the soil testing.

The distribution of soil varies with the type of terrain and climate at that certain place. Basically the types of soil found in Malaysia can be seen in Figure 1.3. The main types of soil in Kuala Lumpur are derived from the Kuala Lumpur Limestone and the Kenny Hill Formation. These soils have its own material properties and will affect the selection of foundation. After obtaining the soil data, an engineer will then proceed to designing a foundation to suit the soil condition.



(Source: Ghazali, 1988)

Figure 1.3: Distribution of Soil in Peninsular Malaysia

Since a foundation system is an essential part of building, a study on building foundations is made. This study focus in the most developed state, Kuala Lumpur, Malaysia. The scope of study for this project is to do a number of case studies on the foundation system for buildings particularly for commercial and residential use. Several case studies will be conducted to compare the foundation system for a given ground condition. The main focus of this study is on the type of foundation used and the factors governing the selection.

1.2 Objectives

The objectives of this study are:

1. Determine the types of soil in Kuala Lumpur
2. Study the properties of soil and local soil testing
3. Determine the different types of foundation used in Malaysia
4. Review the factors governing the selection of foundation system.
5. Compare the various type of foundation system used in the case studies.

1.3 Chapters Overview

Chapter 2

The first part of this chapter discuss about the type of soil encountered in Kuala Lumpur, Malaysia. The second and the third part discuss about the properties of soil and also the soil testing carried out to determine the soil engineering properties respectively.

Chapter 3

Discuss about the different types of foundation, which can be classified into deep and shallow foundation. This chapter also discussed about the factors governing the selection of foundation.

Chapter 4

Discuss about the design approach in deep and shallow foundation design. It also discuss about the procedures involve in designing a foundation and also the foundation relating problems.

Chapter 5

Contain 5 case studies on buildings' foundations in Kuala Lumpur, Malaysia. Each of the case studies has its own significant value.

Chapter 6

Contain the comparison of type of soil, type of foundation used, factors governing the foundation system and also some of the major issues involved in the case studies.

Chapter 7

This last chapter contains the project overall conclusions and further research works to be done in the future..

CHAPTER 2

SOIL PROPERTIES AND TESTING

An engineer must first determine the types of soil deposits at the site before designing a foundation that can safely support a building. A site investigation is a requirement for any project. An engineer must also understand that soil is not a homogenous material and the soil profiles vary with depth and location. Every foundation design requires the evaluation of the site conditions and also the soil parameters. The geo-technical properties of soil will influence the selection of a foundation system for a building and hence influencing the foundation design. Hence this chapter is created to provide an insight into the soil engineering properties and the soil testing available to assess the soil properties.

This chapter is divided into three parts. First part of this chapter will discuss about the soil condition in Kuala Lumpur, Malaysia. The second part is an overview of the engineering properties of soil and the third part will describes the soil testing available to access the soil properties.

2.1 Soil in Kuala Lumpur

Malaysia is a tropical country and consists of different types of soil. The types of soil vary with location and can be classified according to its age. The classification of the soil in Malaysia is shown in Table 2.1 below.

Type	Description
Quaternary	Marine and continental deposits. Clay, silt, sand and peat with minor gravel.
Tertiary	Isolated continental basin deposits of Late-Tertiary age. Shale, sandstone, conglomerate and minor coal seam.
Cretaceous – Jurassic	Continental deposits of thick, cross-bedded sandstone with subordinate conglomerate and shale/mudstone. Volcanics locally present.
Triassic	Interbedded sandstone, siltstone and shale. Widespread volcanic. Limestone prominent at lower part of succession. Conglomerate and cherts locally prominent.
Permian	Phyllite, slate and shale with subordinate sandstone and schist. Volcanic in composition. Prominent development of limestone throughout succession.
Carboniferous	Phyllite, slate and shale. Volcanic of acid to intermediate composition.
Devonian	Phyllite, schist, slate, limestone and sandstone locally prominent. Some interbeds conglomerate, chert and rare volcanics.
Silurian – Ordovician	Schist, phyllite, slate and limestone. Minor intercalations of sandstone and volcanics.
Cambrian	Sandstone/metasandstone with subordinate siltstone, shale and minor conglomerate.

(Source: Ghazali, 1988)

Table 2.1: Type of Soil in Malaysia

Kuala Lumpur is the capital city of Malaysia and is located at the western part of Peninsular Malaysia. The two main soils in Kuala Lumpur are derived from the Kuala Lumpur Limestone and the Kenny Hill Formation. About one third of the area

is on limestone formation. The limestone in Peninsular Malaysia is of Ordovician to Triassic age. Almost all limestone are re-crystallised and technically referred as marble in Malaysia. This limestone has been subjected to high pressure and temperature, accompanied by regional metamorphism and granite intrusion. The limestone is white, pale grey or slightly yellowish, fine to coarse rock. (Gue, 1999)

Due to the humid condition in Malaysia, two of the dominant limestones are calcite and dolomite and the solution forms of limestone are pinnacles, sinkholes and cavities. The soil properties of limestone may leads to variety of geotechnical problems. Pinnacles which are columns of limestone can influence the setting of piles on top of it. Cavities are voids formed by the dissolution of limestone that may also affect the foundation system.

The sinkhole phenomenon is common in area covered by loose and non-cohesive sand over the limestone. The liquid from the dissolved of limestone by acidic solution will penetrate through the weak zones in limestone and develops channels and voids. Loose sand may also flow into the voids and cavities in limestone. The movement of sand will create empty spaces in the sand layer. A critical stage is reached when the roof is not able to support the overburden and this will result in sinkhole which is very dangerous as the sinking process is sudden and catastrophic.

The second type of soil in Kuala Lumpur is the Kenny Hill Formation. This formation consists of Carboniferous to Triassic meta-sediment with some quartzite and phyllite which are the products of metamorphic sandstones and silts. (Gue&Tan, 2003)

Only a small area of Kuala Lumpur is covered by granite formation. The granitic rock ranges from coarse to very coarse-grained and is pale to grey in colour.

2.2 Soil Properties

2.2.1 Particle Size

The size of a particle depends on how it was measured. The two common ways to determine the particle size are sieve analysis and hydrometer analysis. Sieve analysis is used for particles larger than 0.06mm. In sieve analysis, the soil sample is shaken on a sieve with specific sizes openings. The hydrometer analysis is used for smaller particles. The size of particle is the diameter of a sphere which settles in water at the same velocity as the particle. (Lambe and Whitman, 1979) Particle sizes vary from 1×10^{-6} mm up to large rocks of several meters. Table 2.2 below shows the types of particle according to its sizes.

Particle	Size
Boulder	> 300 mm
Cobble	150 – 300 mm
Gravel	2 – 150 mm
Sand	0.06 – 2 mm
Silt	0.002 – 0.06 mm
Clay	< 0.002 mm

Table 2.2: Types of Particle

2.2.2 Particle Shape

Sands are broken rocks formed by physical weathering process. They have a round shape. Clays which are the product of chemical weathering have a flaky shape. These different shape characteristics depend on the amount of wear during transportation by wind, water or ice. The shape of the grains will influence the permeability, compressibility, shrinking and swelling potential of the soil.

2.2.3 Degree of Roundness, Surface Texture and Colour

The degree of roundness refers to the sharpness of a particle. There are 5 degrees of roundness that are angular, sub-angular, sub-rounded, rounded and well-rounded. The texture of the particle will influence its ability to hold water. Fine textured soil can hold more water than the rough textured soil. The surface roughness of a particle can be described as dull, polished, smooth, rough, etched, pitted or frosted.

Colours can be used to describe a soil. Organic soil is dark brown or dark green in colour. Meanwhile the peat soil is dark brown or black in colour. The colour description must be used with caution as the colour of the soil mass can change with a change in moisture content or chemical composition. (Lambe and Whitman, 1979) The different colours used to describe a soil are illustrated in Figure 2.1.



Figure 2.1: Soil Colour Chart

2.2.4 Grading

The grading of soil is the distribution of sizes. A poorly-graded or uniform soil has a narrow range of particle sizes meanwhile a well-graded soil contains a wide range of sizes.

2.2.5 Structure

The arrangement or orientation of particles within a soil mass is termed as soil structure. The arrangements include the bedding orientation, voids, layer thickness, stratification, joint and fissures and the bonding between the particles. The types of structures available are platy, prismatic, columnar, blocky and granular.

The structure of natural soil will vary from one another. The structural features have influence in the properties of soil such as the permeability, strength, stiffness and stability of the soil.

2.2.6 Permeability and Compactness

Permeability refers to the ability of soil to transfer water through soil. Permeability depends on the structure, porosity and texture of that soil. Compactness or field strength is the degree of easiness a soil can be driven. The soil can be reported as loose, dense or slightly cemented. (Whitlow, 1995)

2.2.7 Dilatancy

Dilatancy is the appearance of film of water on the surface of a soil pat when it was tapped. Fine sand and inorganic silts exhibit dilatancy while clays and medium-to-coarse sand does not.

2.2.8 Cohesion, Plasticity and Consistency

Cohesion refers to the ability of soil particles to stick together. If the soil can be moulded easily without breaking, it possesses plasticity. Both of these properties depend on the moisture content of the soil. The consistency is the indicator of cohesive or plastic soil. The consistency varies with water content and it can range from dry (solid) to wet (liquid). The water content at which the consistency changes from one stage to the next stage is called the Atterberg Limits.

2.2.9 Atterberg Limits

The Atterberg limits are based on the concept that a soil can exist in 4 stages that are solid, semisolid, plastic and liquid state. The water contents at these boundaries of adjacent states are the shrinkage limit, plastic limit and liquid limit.

The shrinkage limit is determined as the water content of the necessary water added to fill all the voids of a soil dry pat. The plastic limit is determined by measuring the water content of the soil when threads of 3mm diameter soil begin to crumble. Meanwhile, the liquid limit is determined by measuring the water content and the number of blows needed to close a specific width channel for a specified length in a standard liquid limit device. (Lambe and Whitman, 1979)

These limits are very useful for soil identification and classification. They are also used to determine the use of soil for fill.

2.2.10 Dry Strength

Dry strength of the soil is determined by breaking a pat of oven-dried soil with fingers. Clay with high plasticity has a high dry strength meanwhile inorganic silt has lower dry strength and is powdery when rubbed. The presence of sand in silt will reduce the dry strength and feel gritty when rubbed. (Whitlow, 1995)

2.3 Soil Testing

2.3.1 Vane Shear Test

The vane shear test is used to determine the undrained shear strength of clay soil. The apparatus consists of 4 blades at the end of a rod. The height of a vane is twice the diameter. The vane which is rectangular or tapered is pushed into the soil at least 4 borehole diameter. The vane rod is enclosed in a sleeve to prevent soil adhesion during rotation. Soil is undisturbed by the pushing. Vanes will rotate at a standard rate of $0.1^\circ/\text{sec}$. The soil will fail in cylindrical shape surrounding the vanes. The maximum torque applied for rotation that cause failure is then measured. (Das, 2004)

The undrained shear strength value obtained is too high due to the increased of strength from the high rate of shear straining and soil anisotropy (Merifield, 2004) It needs to be corrected using the correction factors. It can also be correlated with the pre-consolidation pressure and the overconsolidation ratio of clay. (Das, 2004)

The vane shear test is a simple and economical test. It is rapid so that the excess pore pressure developed during the testing does not have the time to dissipate. (Merifield, 2004) It gives a good result for medium-stiff clay. However, this test is time consuming and is limited for soft to stiff clay. Errors in test might occur due to poor calibration of torque measurement, damaged vanes and improper control of vane rate of rotation.



(Source: Envi, 2004)

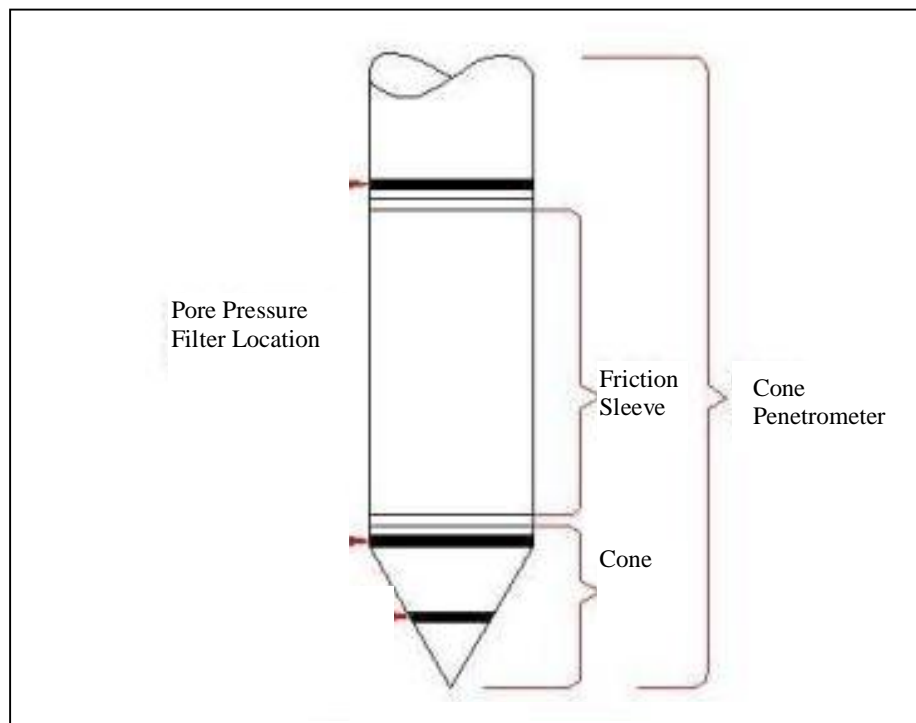
Figure 2.2: Shear Vane Apparatus

The result obtained from this test is used to estimate the down-drag force on the pile shaft. The re-moulded strength of the sensitive clay is relevant to the overall stability problem associated with pile driving in soft clay ground.

(Fleming, Weltman, Randolph and Elson, 1992)

2.3.2 Cone Penetration Test

Cone penetration test (CPT), also known as static penetration test is used to determine the materials in the soil profile and to evaluate the engineering properties of soil. (Das, 2004) The test is also used to access the bearing capacity and settlement of foundation. Beside that, the shear strength of clay can also be estimated.



(Source: Brouwer, 2002)

ne

Figure 2.3: Cone Penetrometer

The penetrometer as shown in Figure 2.3 consists of a cone attached to a rod. The cone has an apex angle 60° and 35.7mm end diameter (Whitlow, 1995). Both the cone and the rod are protected by an outer sleeve. Cone is pushed into the soil at a uniform rate of 20mm/sec.

The test measures the soil resistance to penetration (point resistance), the cone resistance and frictional resistance. The ratio of the force required to end area is the cone penetration resistance. (Whitlow, 1995) The friction ratio which is the ratio of frictional resistance to cone resistance obtained is used to determine the type of soil present at the testing area. The ranges of friction ratio for different soil types are given in the Table 2.3.

Several correlations have been developed between the cone resistance and other soil properties such as the relative density, undrained shear strength and overconsolidation ratio for clay and the friction angle for sand. (Das, 2004)

Soil Type	Friction Ratio, R
Clay	4 – 8
Silt	2 – 4
Sand	1 – 2
Coarse Sand and Gravel	0.67 – 1

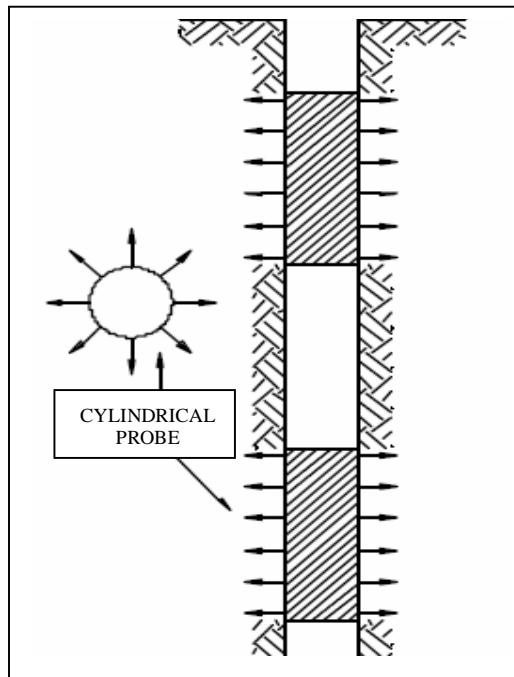
Table 2.3: Friction Ratio

The cone penetration test is able to determine the variation between hard and soft soil layers. It is a cheap and fast test. It provides a continuous record for the ground conditions. However, this test needs a high investment and a skilled operator to perform the test. The test is not suitable for gravel or boulder deposits. (Merifield, 2004)

2.3.3 Pressuremeter Test

Pressuremeter test measures the soil strength and the deformability for most of the soil and weak rock. The test also estimates the soil moduli such as the Young's and shears modulus. The analysis is based on undrained parameter for cohesive soil and drained parameter for granular soil.

The test is conducted in a borehole with diameter 1.03 to 1.2 the diameter of the probe. The apparatus consists of a cylinder with 3 cells. The top and bottom parts of the probe are the guard cells and the middle part is called the measuring cell. (Das, 2004) The probe, shown in Figure 2.4, will be lowered down the borehole. A membrane is inflated using air or fluid, to compress the sides of the borehole as the membrane expands.

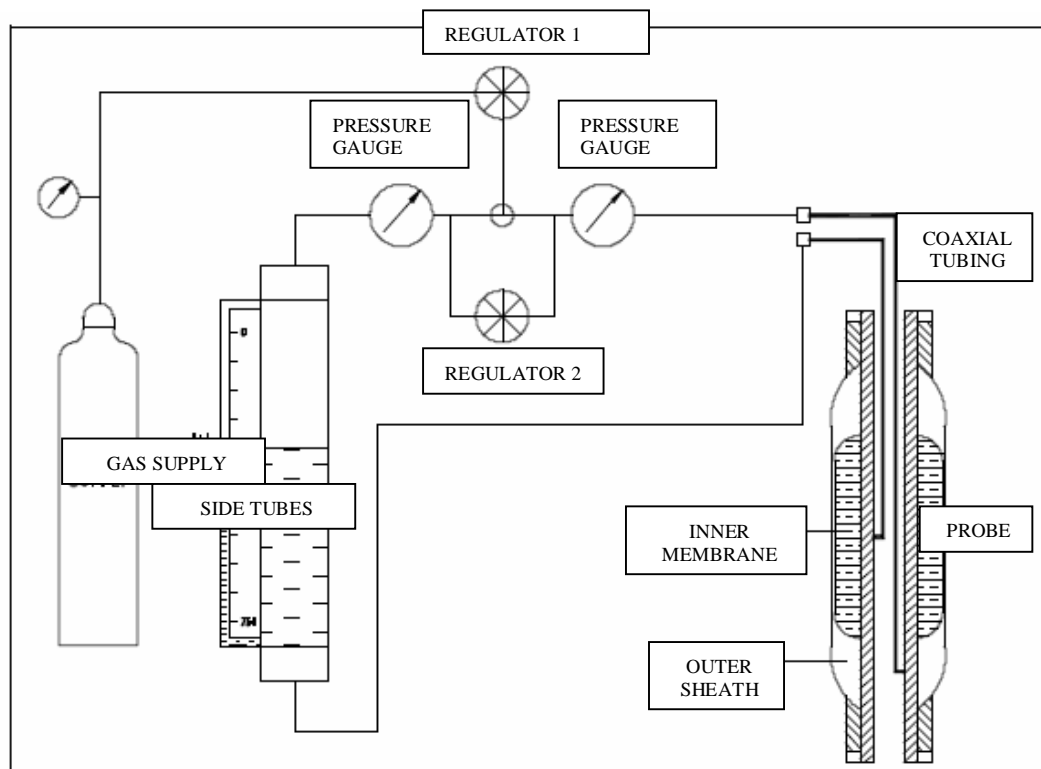


(Source: Roctest-Telemac)

Figure 2.4: Pressuremeter Test

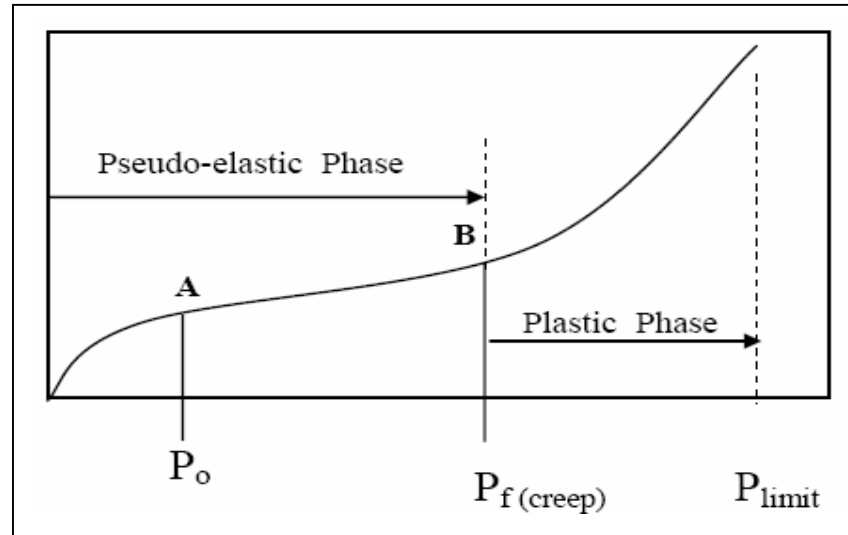
Pressuremeter is installed at the test location and the zero reading is taken with the sides just touching the soil. The pressure in the cell is increased gradually and the volume change is recorded. The volume changes are then plotted against pressure and the net result is a stress-strain curve as shown in Figure 2.6.

The Young's modulus of the soil can be found from the relationship between volumetric expansion of the membrane and the increments of pressure. This test is quick but expensive and is generally used in special or large project. High technical care is required during its use. (Merifield, 2004)



(Source: Roctest-Telemac, 2002)

Figure 2.5: MENARD - Type Pressuremeter



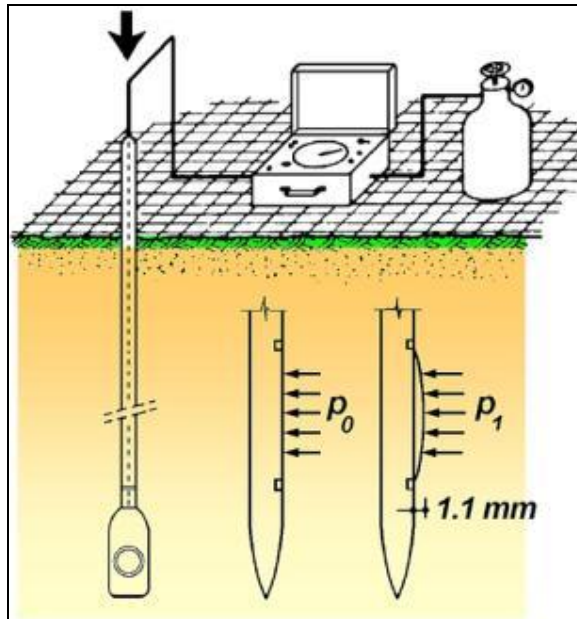
(Source: GME Consultant Inc, 2003)

Figure 2.6: Graph of Result

2.3.4 Dilatometer Test

Dilatometer is used to obtain the pressure at selected depth. The test also provides information on soil type from its material index, the vertical drained constrained modulus, undrained shear strength and the horizontal stress index which is similar to the overconsolidation ratio.

The test involves pushing a steel blade into the soil by using push or drill rigs or the SPT hammer. No borehole cuttings are produced by this test. The dilatometer system consists of a high strength steel blade, tubing, pressure gauge, readout unit and nitrogen gas tank. The tapered steel blade is 240mm long, 95mm wide, 15mm thick and an 18° wedge tip. (Merifield, 2004) A 60mm diameter membrane which exists on the blade face is inflated with nitrogen gas. The tubing contains a wire which is connected to an alarm that notifies the operator to take specific pressure reading. The layout of the testing is shown in Figure 2.7.



(Source: Totani, Marchetti, Monaco and Calabrese, 2001)

Figure 2.7: Layout of Dilatometer Test

2 pressure readings that are the pressure required to lift off the membrane and the pressure at which the membrane expands 1.1mm into the surrounding soil are taken. (Das, 2004) These 2 readings will be corrected to find the liftoff pressure and expansion pressure of the soil.

2.3.5 Plate Load Test

Plate load test is used to access the bearing capacity and settlement of the soil. It can also be used to determine the shear strength of soil needed in pile design. This test is suitable to be carried out in clay soil.

A trial pit is excavated to a required depth and the bottom part is leveled. The rigid steel plate, square or round shape, is placed firmly on the soil. Load is then increased at a constant rate and the deformation is measured. The load is applied until the steel plate yields. (Whitlow, 1995) The failure usually happens when the soil settlement exceeds 15% of the plate diameter.

This test is expansive and time consuming. Errors may occur due to the difference between the plate area and the actual foundation. Besides that, the plate which is not carefully placed will also affects the readings. Errors also happen when the deep layer of soil is stressed by the actual structure and not the plate test. The surface crushing which gives misleading result can be minimized by performing the test at foundation level.

(Fleming, Randolph and Elson, 1992)

When the ground condition is not uniform, a screw plate can be used. The screw plate is self-bored and the test can be carried out at various depths. (Merifield, 2004)

2.3.6 Triaxial Test

The triaxial test is used to determine the stress-strain properties of soil. It can be conducted on sand and clay. A cylindrical soil sample is placed in a plastic chamber and is subjected to an all-around confining pressure by the liquid in the chamber. The equipments used in triaxial test are shown in Figure 2.8.

Axial stress is applied by a loading piston until the specimen fails. The axial stress and the confining stress are the major and minor principal stress respectively. The increment of axial stress is called the deviator stress. (Lambe and Whitman, 1979)

The major and minor principal effective stresses are plotted as Mohr's Circles. The shear strength parameters (cohesion and angle of friction) can now be determined from the Mohr's circle. A common tangent to the circles is known as the Mohr-Coulomb failure envelope. (Das, 2004)

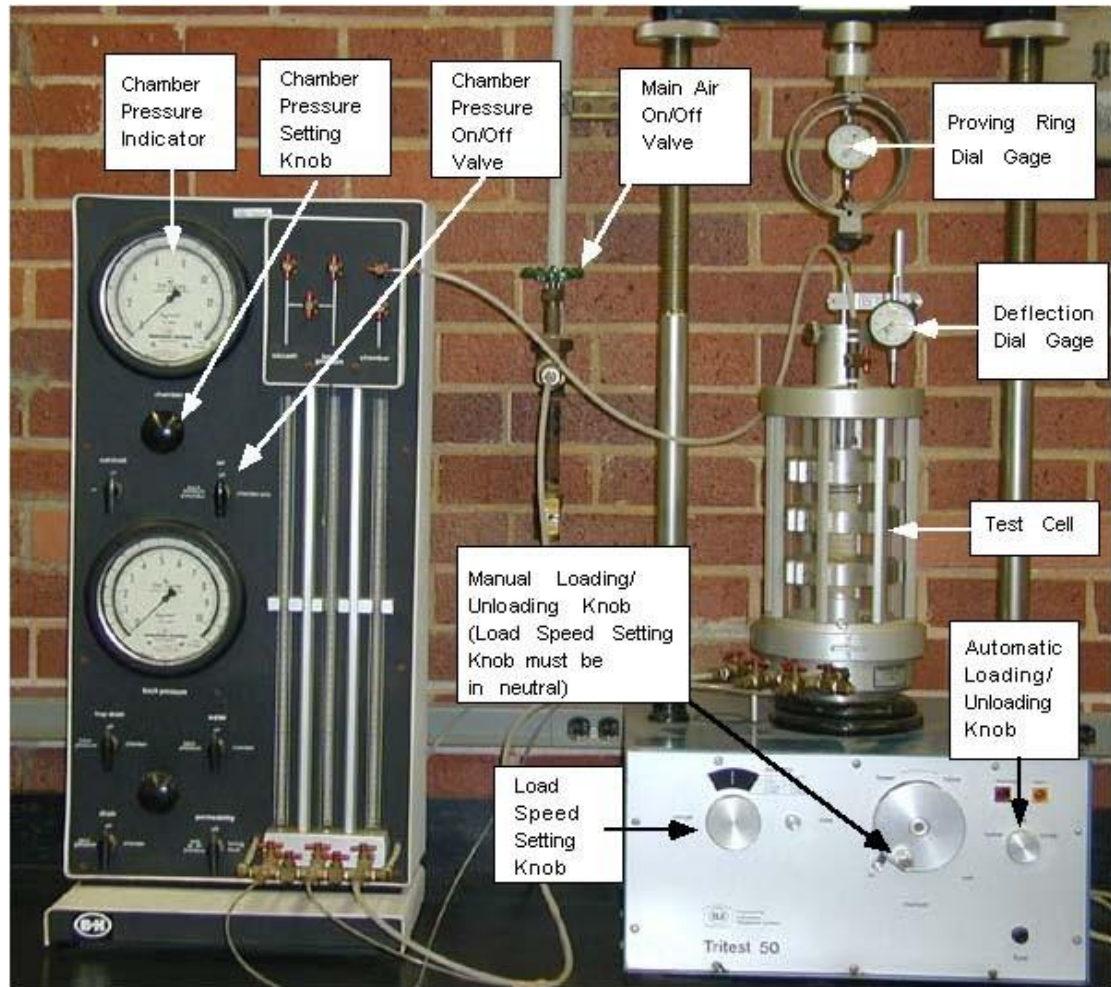


Figure 2.8: Triaxial Test Equipment

For clay, 3 main types of tests can be conducted with the triaxial equipment. (Das, 2004)

- a) Consolidated-drained test
- b) Consolidated-undrained test
- c) Unconsolidated-undrained test

The difference between these 3 tests is the drainage condition. Drainage is allowed in consolidated-drained test but not in the unconsolidated-undrained test. Meanwhile in the consolidated-undrained test, drainage is only allowed during the initial stage, when chamber pressure is applied.

2.3.7 Standard Penetration Test (SPT)

The SPT is an index test used for correlation of soil properties. It can be performed in a wide range of soils but not in gravel deposit and soft clay. SPT involves the driving of split-spoon sampler into the soil using hammer. The standard weight of a hammer is 622.72kN. The hammer will be dropped at a distance of 0.76m to achieve 3 penetrations at 150mm interval. The number of blows required for the last two intervals are added to give the N-value (Standard Penetration Number). The measured N-value will be corrected to the N₆₀-value. (Merifield, 2004)

The soil properties can then be determined from the N₆₀-value. The consistency of clay, undrained shear strength of clay, overconsolidation ratio (OCR) of natural clay, relative density of sand and soil friction angle can be correlated using the standard penetration number. The N-value is also used in pile load capacity and pile design.

However, according to Kulhawy and Mayne (1990) errors in SPT may still occur due to:-

- a) Inadequate cleaning of borehole
- b) Use of bent drill rod
- c) Careless count of blows
- d) Use non-standard sampler
- e) Inaccurate hammer weight
- f) Sampler is driven above the bottom of casing
- g) Failure to maintain adequate head of water in borehole
- h) Hammer strikes the drill rod collar eccentrically
- i) Existence of gravel or cobble in soil

2.3.8 Dynamic Cone Penetrometer Test

This test is similar to the SPT. The small hand-held equipment is pushed into the soil by hand. The number of blows required to advance the cone in 150mm of soil is

recorded. This blow counts can be used to identify the material types and relative density.

Soil sample is not obtained in this test, not as in SPT. The blow counts from the second increments are larger than the first in granular soil but tend to be the same in cohesive soil.

2.3.9 Direct Shear Test

The angle of friction for sand can be determined from the direct shear test. The sand specimen is placed on a shear box that split in the middle.

First, a normal load is applied by a loading press or dead weight. Then, a shear force is applied to cause relative displacement between the 2 parts of the box. (Lambe and Whitman, 1979)

The normal and shear stresses at failure are recorded. The angle of friction can be found from the graph of shear stress against effective normal stress.

CHAPTER 3

FOUNDATION TYPES AND SELECTION

“Every one who hears these words of mine and does them will be like a wise man who built his house upon the rock; and the rain fell and the flood came and the wind blew and beat upon that house but it did not fall.....” (Matthew 7.24-25)

Throughout ages, both the builders and laymen have recognised the necessity of good foundation, as did the wise man who saw that even the superstructure would resist the forces of nature better if it was founded upon the rock. (Sowers, 1962)

Foundation is a part of structure which interfaces the superstructure to the adjacent zone of soil or rock below it. The purpose of having a foundation is to transfer the superstructure loads to the underlying soil or rock without overstressing the soil or rock.

The three basic requirements of a satisfactory foundation as stated by Sowers, 1962 are:

- The foundation must be properly located with respect to any future influences which could affect its performances.
- The foundation, including the soil below it, must be stable and safe from failure.
- The foundation must not settle sufficiently to damage the structure.

The first part of this chapter describes the different types of shallow and deep foundations and the second part discusses about the selection of the different foundations in a certain condition.

3.1 Types of Foundation

The types of foundation used can be classified into 2 categories that are shallow and deep foundations. Shallow foundations are used when the soil formation has adequate strength for a safe bearing support. However, if the soil has lower shear strength or is highly compressible, shallow foundations may not be a suitable option. Under such circumstances, deep foundation may be used. The loads will be transmitted to a greater depth or to a stiffer stratum or to rock by deep foundation.

3.1.1 Shallow Foundation

Shallow foundations are used to transmit the loads of the superstructure to the adjacent soil below it. The types of shallow foundation available are pad foundation, strip foundation and raft foundation.

Pad Foundation

Pad foundation is used to support the point load of the column. There are a number of different types of pad foundations available which include the mass concrete for steel column, plain reinforced concrete, stepped reinforced concrete and the balanced

pad foundations. The types of pad foundations are shown in Figure 3.1. The detail and arrangement of pad foundation is shown in Figure 3.2.

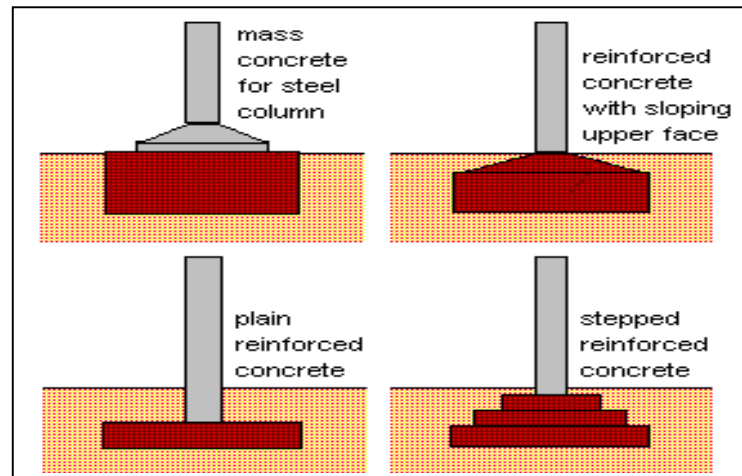


Figure 3.1: Types of Pad Foundations

Shallow mass concrete pad consists of a mass of concrete pad supporting the point loads from columns and piers. They are used for varying conditions of soil layers where the suitable load bearing soils exist at shallow depth. (Curtin, 1994) Meanwhile, a deep mass concrete pads which are cast at soil depth of 1.5 to 2m are used when the piling alternative is more expensive.

Shallow reinforced concrete pads are similar to the mass concrete pads but of a smaller thickness because of the usage of reinforcement on the tensile face of the pad which increases the pad resistance to bending moment. The deep reinforced concrete pads are constructed at a depth where the suitable soil layer is not available.

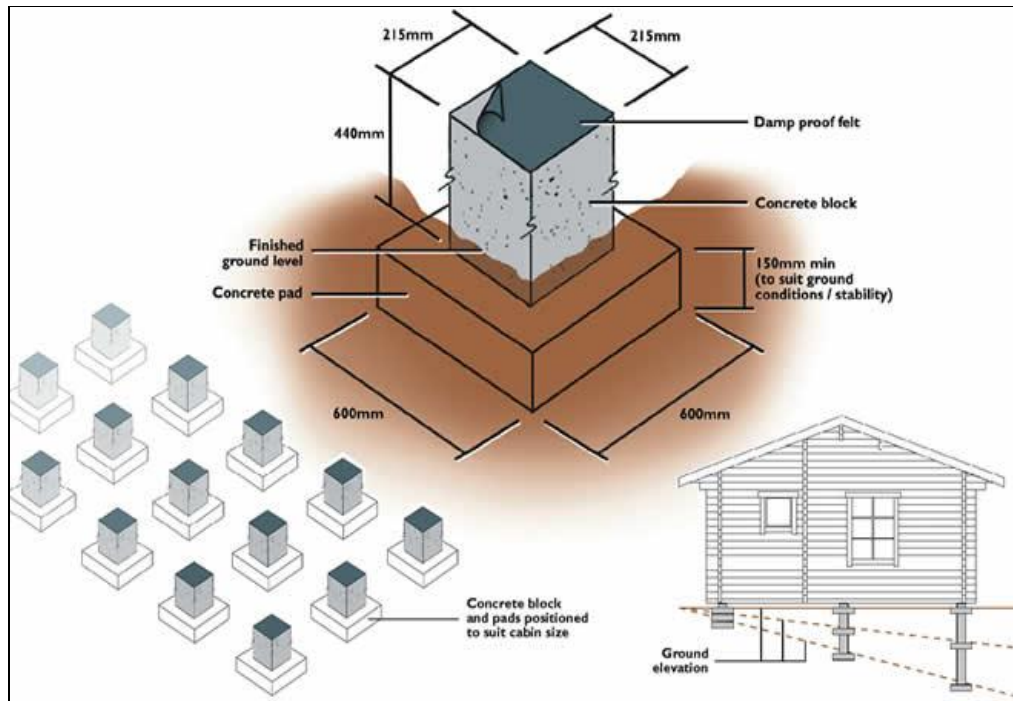


Figure 3.2: Pad Footing

Balanced pad foundations which consist of reinforced concrete pad bases are used when the amount of loads needed to be supported is high and where there is excessive differential settlement caused by the variation of soil pressure. The different types of balanced pad foundations include the rectangular, trapezoidal, holed and cantilever.

Strip Footing

Strip footings are used under relatively uniform point loads. The strip will distribute the concentration of the load sideways into an increased width of sub-strata to reduce the bearing stress and settlement to an allowable limit. (Curtin, 1994)

The structure will also distribute the loads in the longitudinal direction when the loading is not uniform and when the sub-strata resistance is variable. The width of the strip is according to the bearing stress limit and also the excavator bucket size. (Curtin, 1994) The cross section of a strip footing is shown in Figure 3.3.

Strip footings are also used when the rows of columns are spaced so closely that pad foundation nearly touched each other. There are different types of strip footings, which include the masonry strip, concrete (plain or reinforced) strips, trench fill (concrete or stone) and reinforced beam strip (rectangular or inverted T)

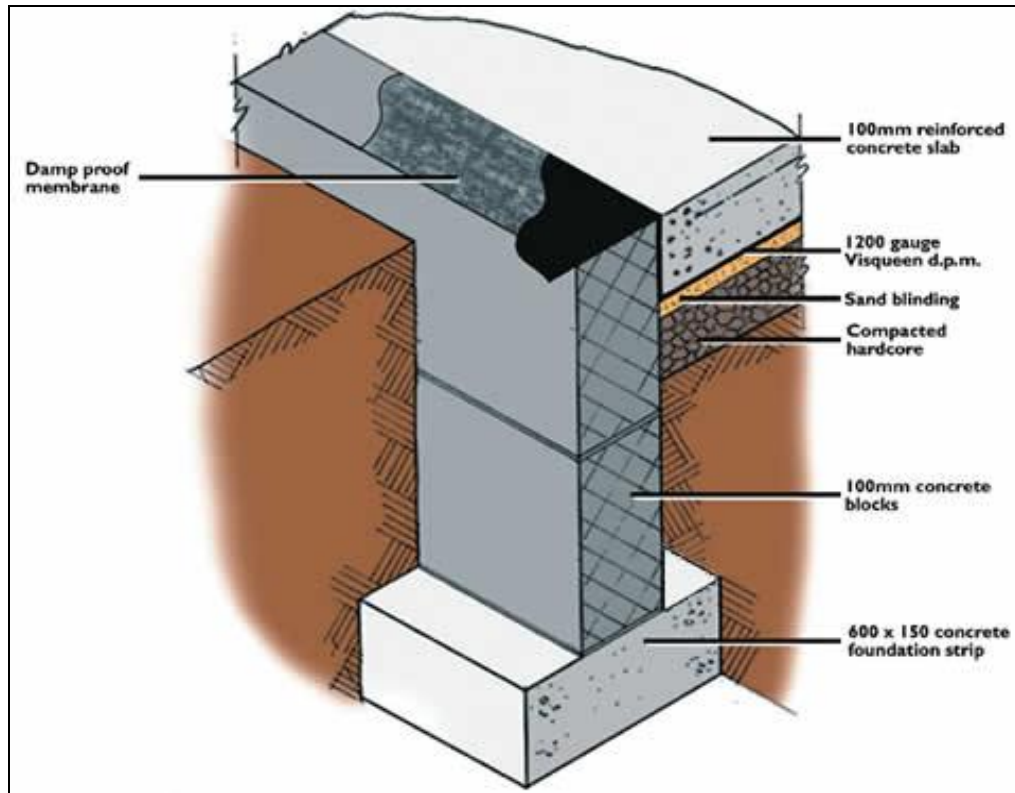


Figure 3.3: Strip Footing

Masonry strips are used when good quality of soil exist below the superstructure. (Curtin, 1994) When using masonry strip in clay soil, it is important to fill all the joints to prevent the strip to act as a drain, allowing the water to flow along the foundation level and through the opening of the joints in the strips.

In current practice, the masonry strips were replaced by concrete strips. The thickness of the concrete strip is determined by the requirement for the line of dispersion to pass through the side of the footing. (Curtin, 1994) The strip is often reinforced with a fabric or lattice reinforcement. The main bars that are the longitudinal bars are selected to cater for the cantilever action on the strip.

Concrete trench fill consist of a mass concrete strip cast into the open trench. (Curtin, 1994) It is used when the strip loads are required to be transferred to a shallow depth through soft material. Stone trench fills consist of layers of compacted stone deposited in an open trench. It is used in poor quality sands and sandy silt areas. It can be used down to a suitable layer at a shallow depth.

Rectangular beam strip consists of a rectangular ground beam. The width is designed to reduce the bearing pressures on the sub-strata. The beam is to resist the induced bending moment and shear forces in the longitudinal direction. The beam is reinforced with caged or ladder reinforcement.

The inverted T beam strip is of similar function as the rectangular beam but the cross section had been modified to an inverted T so that the flanges can reduce the contact pressure on the ground. (Curtin, 1994)

Wide strip footing is used when the bearing capacity of soil is low enough to necessitate a strip so wide that the transverse bending occurs in projecting portions of foundation beam. Adequate reinforcement is required to prevent cracking in wide strip footing.

Raft Foundation

Raft foundation is also known as mat foundation. It is a large spread footing that supports most of the structure loads. A raft foundation spreads the structural load over a large area to reduce the bearing pressure. It is more rigid and thus reduces the potential for excessive differential settlements. Raft has greater weight and is able to resist greater uplift loads. It distributes lateral loads into the soil more evenly and efficiently.

Since most of the structures require a ground floor slab, it is economic to incorporate it with the foundation into one element. The combination of floor ground slab and the foundation can be done by making the upper surface of the raft foundation coincides with the top surface of the floor slab.

The following figures show the construction of raft foundation in Malaysia.



Figure 3.5: Construction of Raft Foundation-1



Figure 3.6: Construction of Raft Foundation-2



Figure 3.5: Construction of Raft Foundation-3

3.1.2 Deep Foundation

Deep foundations are used to transfer the structural loads to a deeper soil strata and when the soils are subjected to scour. The different types of deep foundations available are piles, piers and caissons.

Pile Foundation

Piles are relatively slender column used to transmit the structural load to a lower, firmer soil or rock. (Cernica, 1995) Piles are used when the soil at normal foundation level cannot support the usual pad, strip or raft foundations. They are made of timber, concrete and steel. According to Cernica, 1995 the selection of type of pile depend on

- a) The corrosive properties of the soil
- b) The fluctuation in water table

- c) The ease of installation
- d) The length requirement
- e) The availability of material
- f) The installation equipments
- g) The restriction on driving noise
- h) Costs

The loads may be transfer through skin friction, from end-bearing or from the combination of both. A friction pile is a pile which resists the load by side or skin friction meanwhile an end bearing pile which rests on a firm stratum, transfer the load directly to the stratum via tip resistance.

The capacity of the pile is estimated based on soil data and can be verified by subjecting one or more pile to a load test. For a driven pile, load test is eliminated if careful observation of pile resistance to penetration is made during the driving process.

Usually, piles can be divided into two categories that are the displacement and non-displacement piles.

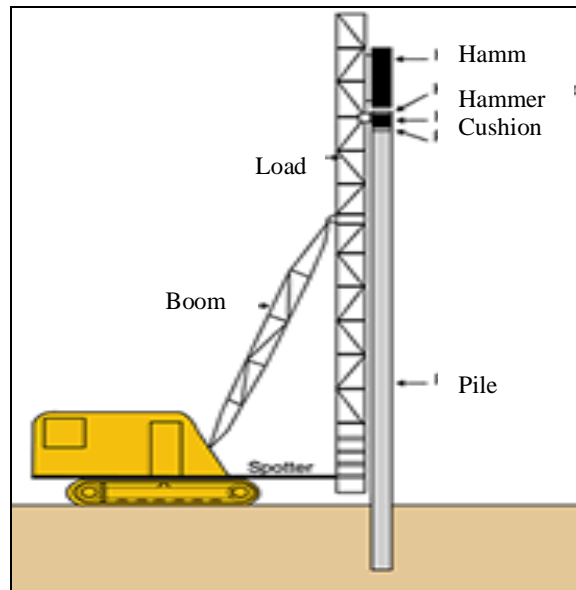
Displacement Piles

Displacement piles comprise of solid-section piles or hollow-section piles driven or jacked into the ground. The soil inside the ground is disturbed and displaced. All types of driven piles are displacement piles. Timber, steel, concrete, precast concrete, composite and micro piles are some of the displacement piles.

Basically a displacement pile can be installed into the soil by

- Inserting it into a bored hole.
- Pushing it into the soil by applying static load.
- Thumping it into place by a pile hammer.

The method of installation of piles can be seen in Figure 3.7



(Source: Hayward Baker, 2003)

Figure 3.7: Installation of Driven Pile

Timber Piles

Timber piles are made of tree trunks with the branches and bark removed. They are installed by driving it into the soil at selected depth. Timber pile has a natural taper with a top cross section of twice or more than that at the bottom end. The size of the pile is determined by the load capacity, type of soil encountered and the building code. Timber piles have longer life span if they are not subjected to alternate drying and wetting. Timber should be treated with preservatives such as creosote oil and the top end of the pile is protected by lead paint, zinc coating before concrete cap is poured.

Timber piles are driven by top hammer with the hammer weight at least 1.5 times the weight of the pile. The head of the piles is protected by a cap. Care should be taken to prevent damage to the pile. Overdriving of timber pile will result in splitting, crushing and shearing of the pile which can be detected by a sudden reduction in

penetration resistance. Because of these, the driving operation of timber piles needs a close observation on the pile behaviour.

In Malaysia, timber piles are not commonly used because of the decreasing supply of timber and also its shorter life span than concrete.

Steel Piles

Rolled-H, fabricated shapes and pipe piles are some of the steel piles available. Steel piles, as can be seen in Figure 3.8, can withstand high driving pressure and are a reliable end bearing member. The bottom end of the pile is usually capped with a flat or a cone shaped point, welded to the pile. Steel piles have higher strength, easy to splice and economy. The steel piles are driven by a vibrator. Soil plug at the bottom of the piles are removed to achieve the required depth of penetration.



(Source: Turner-Fairbank Highway Research Center, 2005)

Figure 3.8: Steel Piles

Steel piles are subjected to corrosion agents such as salt, acid, moisture and oxygen. It will corrode more rapidly in disturbed soil because of higher oxygen content. The steel piles can be protected from corrosion by coating the pile with paint, encasing them in a concrete shell, increase the thickness of steel section or use cathodic protection.

The usage of steel piles in Malaysia is less preferable than concrete piles because of the increasing steel price in the market.

Concrete Piles

Concrete piles are used to transfer loads through a bearing material to a deeper load-bearing strata. They are more immune to corrosive elements. The types of concrete piles available are the precast concrete piles and cast-in-situ concrete piles.

Concrete piles are lifted at the quarter and should not be rested on the ground on their head. The lifting of concrete piles is shown in Figure 3.9. The hammer weight used must be sufficient to ensure a final penetration of 5mm per blow. The drop of hammer should be reduced if cracking occurs.



Figure 3.9: Lifting of Concrete Piles

Precast Concrete Piles

Precast concrete piles are used in area of soft and undisturbed soil. The piles are formed, cast to specified length and shape and cured before they are driven into the

ground. Piles are cast with pointed tip, square or hexagon cross section and are tapered. For marine installation, part of pile serves as column above the ground.

The pile head and reinforcement are designed to take the impact loads caused during pile driving. Cutoff and splices may be required to adjust the pile length. The disadvantages of precast concrete pile are:

- a) Pile can be damaged during driving.
- b) Pile can be displaced when it meet an obstruction such as boulders in the ground.
- c) Short pile is difficult to extend and long pile can prove to be expansive and wasteful.
- d) It requires a large rig for driving.

Cast-in-situ Piles

The installation of cast-in-situ piles consists of driving a steel or precast concrete tubing or casing into the ground and then fill it with concrete. The piles can be cast accurately to the required length. The piles may have a smooth or irregular side surface depend on the method of driving. Large rigs are required to drive the piles. Cast-in-situ pile is economic for sandy, gravels, soft silts and clay, particularly when large numbers of piles are required.

Composite Piles

Composite piles are made of combination of various materials that can be used to overcome the problems resulting from a particular site condition. The joints between the different elements must be constructed rigidly to withstand bending and tensile stresses.

Micropiles

Micropiles are piles having a diameter less than 300 mm. They can be installed into the ground by driving small diameter steel tubes, followed by the injection of grout with or without the withdrawal of the tubes.

Non-displacement Piles

Non-displacement piles are formed by first removing the soil by boring method. Concrete is then used to fill the hole. Bored and augered piles belong to this category of pile.

Bored Piles

Bored piles are formed using a simple cable percussion rig. (Curtin, 1994) The soil is removed by shell and auger. Then the hole is filled with in situ reinforced concrete. It is economic for filled sites or soft clay overlying stiff clay or rock. Bored piles are not used in granular soil as the removal and disturbance of surrounding ground can cause excessive removal of soil and cause settlement. During operation, the hole can be lined with a casing which can be driven ahead of the bore to overcome difficulties caused by groundwater and soft sub-soil.

Augered Piles

Augered pile is a shell-less type of pile formed using a continuous auger with hollow stem. The auger diameter size range from 25 to 40cm and the shape of the auger can be seen in Figure 3.10. The tip of auger has a small opening which is plugged up during the downward augering. The hole is filled with concrete or a cement grout is injected under pressure down the hole during withdrawal of auger.

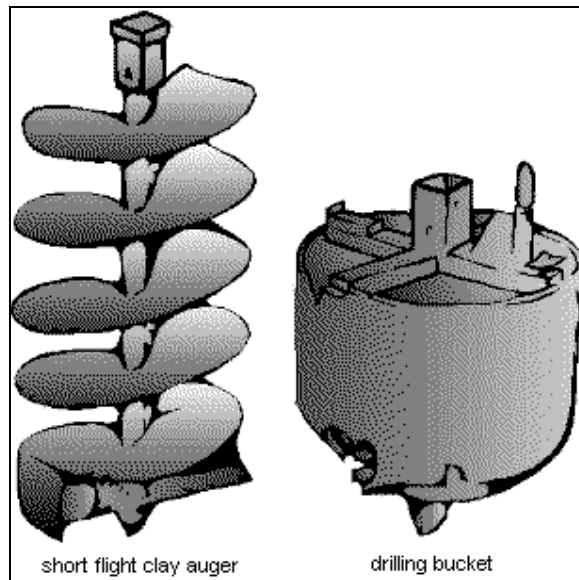


Figure 3.10: Auger Drilling Bit

Augered piles are used in sand and gravel strata. They provide an excellent resistance against friction and are vibration free. They require no splicing and have a fixed dimension length.

The difference between displacement and non-displacement piles are discussed in the Table 3.1 below.

Type of Piles	Advantages	Disadvantages
Displacement	<ol style="list-style-type: none"> 1. Piles can be checked for quality and soundness before driving. 2. Construction is not affected by ground water. 3. Piles can be driven in long length. 4. Piles can withstand high bending and tensile stresses. 	<ol style="list-style-type: none"> 1. Piles may break during driving. 2. Piles may suffer from unseen damages. 3. Displacement of soil during driving may damage the adjacent structures or piles. 4. Noise and vibration impacts.

Non-displacement	<ol style="list-style-type: none"> 1. Length can be varied. 2. Materials forming the piles are not governed by handling and driving stresses. 3. Insitu loading test and penetration tests can be made. 4. Piles can be installed without noise and vibration impacts. 5. Soil removed during boring can be inspected. 6. No ground heaves. 	<ol style="list-style-type: none"> 1. Concrete cannot be inspected after installation. 2. Low end bearing resistance in cohesionless soil due to loosening by drilling operation. 3. Concrete is liable to necking in soft soil. 4. Drilling piles in group can cause settlement of adjacent structures.
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Table 3.1: Advantages and Disadvantages of Displacement and Non-Displacement Piles

Piers and Caissons

Piers and caissons are larger piles. For drilled piers, a shaft is drilled into the soil and filled with concrete. The shaft may be cased with metal casing which will be later left in place as part of the pier or may be gradually removed. Caissons are used for the construction of bridge piers and abutment in river and lakes. It may be an open end or box type.

Piers and caissons are used instead of piles when:

- 1) Pile capacities are not sufficient to carry the imposed loads.
- 2) When larger end-bearing area is needed.
- 3) When driving noises and vibrations are not allowed at the site.

3.2 Foundation Selection

The selection of suitable foundation is the most important part of design process but the most difficult to define. The selection process involves the consideration of the type, nature and availability of the information required, its collection and its validity. The selection is governed by many factors which include sub-soil conditions, past site usage, adjacent construction, size and scale of development proposal, timescale and cost limitation.

The selection process will be based on the site information obtained in the following list, which is not presented in any significant order. (Curtin et. al, 1994)

- a) History of the site
- b) Soil qualities
- c) The effect of water table
- d) Chemical properties of soil
- e) Access to site
- f) Condition of existing structures
- g) Site contours and vegetation
- h) Acceptable settlements and movement
- i) Availability of materials
- j) Proposed superstructures requirement
- k) Foundation of adjoining buildings
- l) Nature of proposed works
- m) Future development and extension
- n) Amount of loads supported
- o) The variation of pressure and loading with time
- p) The effect of removal of overburden
- q) The possibility of errors in information received
- r) The costs of materials and labour

The selection of foundation based on its usage is summarized in the following table:

Types of Foundation	Usage and Considerations
Footing	<ol style="list-style-type: none"> 1. Are virtually unlimited in use. 2. Things to be considered include soil profile, location of water table, potential of scour, size and shape of footing, adjacent structures.
Driven Piles	<ol style="list-style-type: none"> 1. Are used when the foundation material cannot support a footing foundation. 2. Considerations include soil profile, driving problem, pile length, noise restriction and adjacent structures.
Non-Driven Piles	<ol style="list-style-type: none"> 1. Are used when driven piles are not practical. 2. Considerations include location of water table, soil profile and adjacent structures.

(Source: Caltrans, 1997)

Table 3.2: Foundation Usages and Considerations

CHAPTER 4

FOUNDATION DESIGN AND PROBLEMS

4.1 Foundation Design

Foundation design is one of the most challenging aspects of engineering. No two foundation conditions are similar. It requires a good sound basic approach to achieve the required result. Foundation design is carried out using a careful blend of geological, soil mechanics, theory of structures, design of materials, experiences, engineering judgments and logic. (Curtin, 1994)

The design is a process using all the available tools and information to produce a good solution. Part of the design process is to understand the practical difficulties, seek indications of unreliability in the results, assess the implication and magnitude of such errors, make suitable allowance in design and to understand the theories used. The design of a foundation consists of determining the elevation, size, shape and structural details of the foundation which need to meet the three basic requirements that are sufficient depth, safety against failure and safety from objectionable deflection.

The procedures for foundation design follow the currently accepted methods which are usually conservatives. The procedures are subjected to improvement for greater precision as new techniques are developed to better determine the soil properties. The foundation profession is still considered as a “state of art” profession. Although scientific methods were used, exact answers are not always expected. Final decisions regarding the foundation type, design criteria and methods for construction are greatly influenced by experiences and intuition. (McCarthy, 1998)

A safe foundation design provides a suitable factor of safety against the shear failure of the soil and excessive settlement. The limiting shear resistance is known as the ultimate bearing capacity of soil. For design, the allowable bearing capacity which is obtained by dividing the ultimate bearing capacity by a safety factor is used.

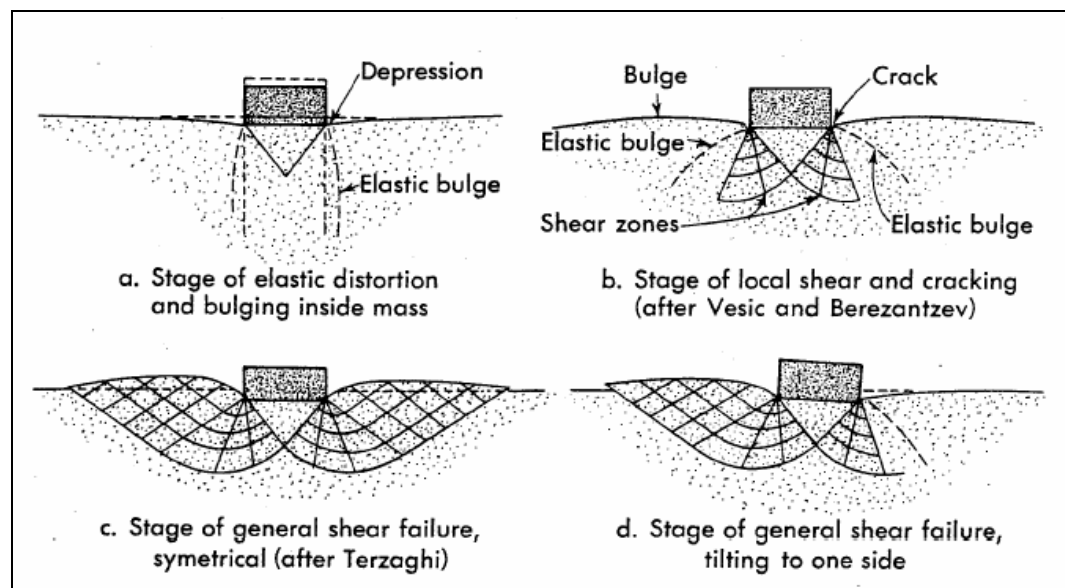
The design criteria as stated by Curtin, et al, 1994 are as follows:

- Foundations should be kept as shallow as possible because excavation can be expensive and timely.
- Avoid expensive and complex details of foundation.
- Pay attention to the buildability of foundation.
- The effect of new foundation loading on the existing adjoining structures.
- The reliability of soil investigation.
- Cost and speed of construction.
- Effect of construction on existing ground.
- The effect of varying length, shape and rigidity of foundation.
- After-effect on completed foundation such as sulphate attack, ground movement.
- Change in local environment such as new construction, re-routing of road.
- Follow the currently reviewed construction techniques.

4.1.1 Shallow Foundation

Foundations need to be capable to carry the structural loads without undergoes movement that can causes structural damage. The soil supporting the foundation should not be stressed beyond its strength limit. Besides that, the deformation caused by this compressed soil cannot be excessive. The pressure that the soil can take without overstressing (shear failure) is the soil bearing capacity. (McCarthy, 1998)

The ultimate soil bearing capacity for the foundation is related to the properties of soil and the characteristics of the foundation itself. The three principal modes of soil failures are defined as general shear failure, local shear failure and punching shear failure. The development of shear failure can be seen in Figure 4.1



(Source: Coduto, 1994)

Figure 4.1: Development of Shear Failure Beneath a Foundation

The general shear failure is expected for soil possessing a brittle type stress-strain relationship. A well-defined wedge exists beneath the foundation and the slip surfaces extending diagonally from the side edges of the footing downward through the soil and then upward to ground surface. The ground surface will then bulge upward and the soil displacement can be noticed by the tilting of the foundation. The

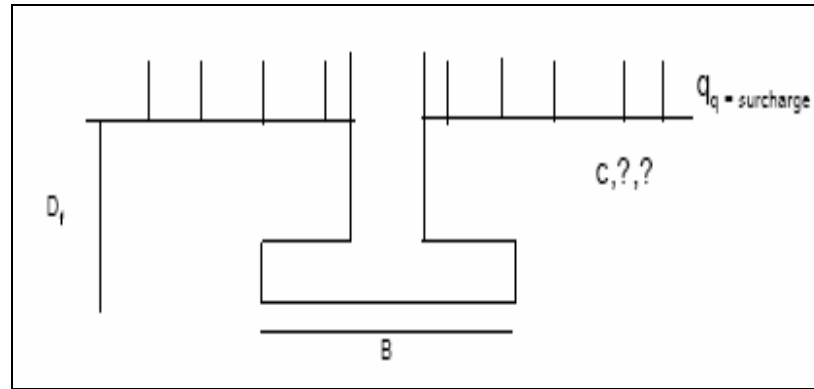
punching shear failure involves the characteristic of plastic material. Significant compression happens beneath the foundation and vertical shear occurs beneath the edges of the foundation. The soil zones beyond the edges are little affected and no significant surface bulging occurs. Meanwhile, local shear failure involves the characteristic of both general shear and punching shear failure modes. A well-defined wedge and slip surfaces are formed beneath the foundation. Slight bulging occurs at the ground surfaces. The local shear condition represents a transitional mode between the general and punching shear failure. (McCarthy, 1998)

In shallow foundation design, it is common to consider that general shear failure happens in dense granular soil and in firmer saturated cohesive soils subjected to undrained loading. The punching shear case is applied for compressible soil such as sands having a low to medium relative density and for cohesive soils subjected to slow loading. (McCarthy, 1998)

Bearing Capacity Equation

Classical theories of elasticity and plasticity are applied in investigating the soil behaviour to develop the equations for foundation design. The original concept using the theory of elasticity was introduced by Prandtl and Reissner. Prandtl studied the effect of a long metal tool bearing against a smooth surface of a metal mass that possess cohesion and internal friction. Meanwhile, Reissner included the condition of the bearing area located below the surface of structure and the surcharge weight acting on the plane in his studies. (McCarthy, 1998)

The studies done by these two men were combined by Terzaghi, whom considered the rough surface of the foundation in design. The theory assumed that no deformation occurs beyond the point of shear failure when loads are applied to the foundation. It also assumed that plastic deformation occurred is small. Terzaghi developed an equation which combined the effect of soil cohesion and internal friction, soil weight, surcharge and foundation size in design. The ultimate bearing capacity equation is shown as:



$$q_u = cN_c + \frac{1}{2}Bg_1N_g + g_2D_fN_q$$

- Where
- q_u = ultimate bearing capacity of soil
 - c = cohesion of soil below the foundation
 - D_f = depth of footing
 - g_1 = effective unit weight of soil below the foundation
 - g_2 = effective unit weight of soil above the foundation
 - N_c, N_g, N_q = soil bearing capacity factors where
- $$N_q = \tan^2\left(45 + \frac{f}{2}\right)e^{p \tan f} ; e = 2.71828$$
- f = angle of internal friction
- $$N_c = (N_q - 1) \cot f ; (f > 0)$$
- $$= 5.14 ; (f = 0)$$
- $$N_g = 2(N_q + 1) \tan f$$

Terzaghi had simplified the above equation to suit different shapes of footing:

For square footing $q_u = 1.3cN_c + qN_q + 0.4gBN_g$

For circular footing $q_u = 1.3cN_c + qN_q + 0.3gBN_g$

Meyerhof proposed an equation similar to Terzaghi but had introduced the shape factors and depth factors. The expressions are presented as the following equations:

For vertical load
$$q_u = cN_c s_c d_c + qN_q s_q d_q + 0.5gBN_g s_g d_g$$

For inclined load
$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + 0.5gBN_g s_g d_g i_g$$

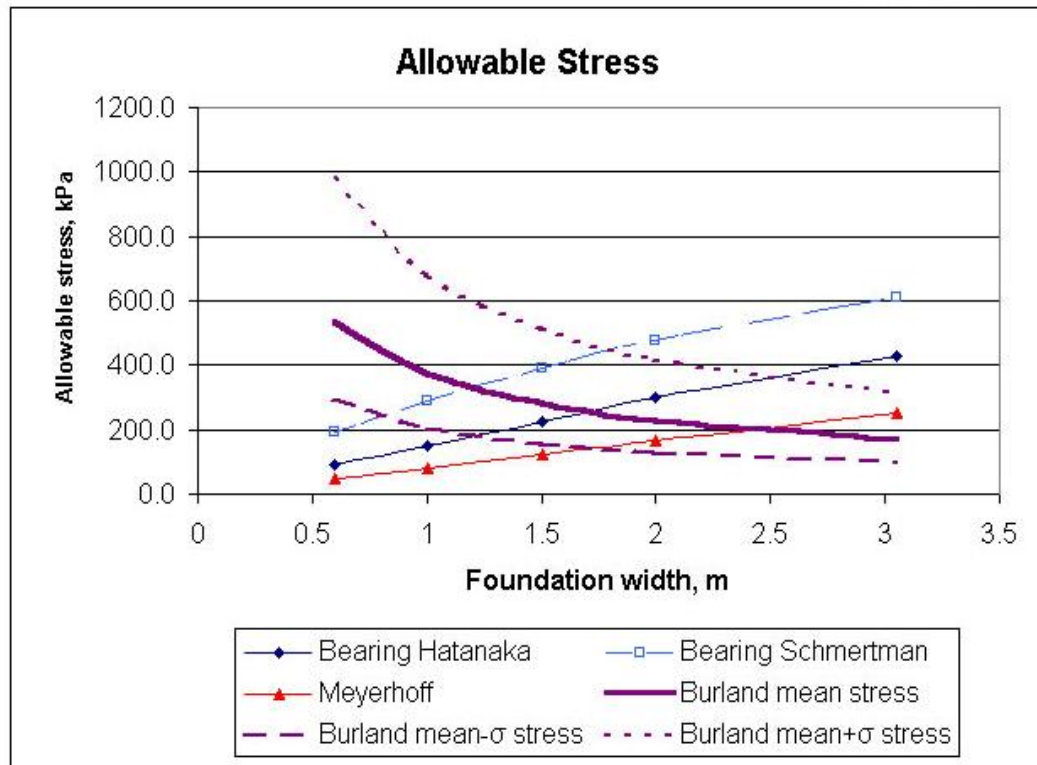
The general bearing-capacity equation below was proposed by Brinch Hansen, an extension of Meyerhof's equation. Besides the shape, depth and inclination factors, Hansen had added ground factor and base factor for footing on a slope.

$$q_u = cN_c s_c d_c i_c b_c g_c + qN_q s_q d_q i_q b_q g_q + 0.5gBN_g s_g d_g i_g b_g g_g$$

Comparison Between Terzaghi's, Meyerhof's and Hansen's Equations

Terzaghi's equations were widely used because they are simpler than Meyerhof's and Hansen's equations. They are used for concentrically loaded horizontal footing and are used in cohesive soil where the ratio of $D/B \leq 1$. Hansen's equation is used when the base of the foundation is tilted or when the footing is on a slope and when $D/B > 1$. (Cernica, 1995)

Terzaghi's equations are acceptable as the ultimate bearing capacity is reduced to an allowable bearing capacity by factor of safety. All the above mentioned equations have their own limitation and usage, therefore judgement is still needed when choosing the suitable equation for design. A sample of the result analysis is shown in Figure 4.2



(Source: Zekkos, 2004)

Figure 4.2: Result Interpretation for Shallow Foundation Design

4.1.2 Deep Foundation

Typically the capacity of deep foundation such as piles is evaluated using analytical methods, field load tests or through pile-driving formulae. The analytical methods require the application of principles of soil mechanics and knowledge on soil conditions and properties, pile type, dimension and installation method. Field load tests involve the application of loads onto the pile and the settlement values for each load increment are obtained. This test is reliable in providing information on the capacity of an installed pile. (McCarthy, 1998)

Pile driving formulae were derived from a concept that relates dynamic energy imparted by pile-driving hammer to the capacity of pile. These formulae are not suitable for determining the actual capacity of pile. Another method used to predict the pile capacity is through the pile-driving data.

Analytical Methods

Pile is supported in two ways namely end-bearing and side friction. The ultimate capacity of a pile is due to the soil resistance developed by friction between the soil and pile and due to end bearing at the tip of the pile. The total pile load can be expressed as

$$Q_u = Q_p + Q_s \quad \text{where } Q_u = \text{total pile load}$$

$$Q_p = \text{tip resistance}$$

$$Q_s = \text{side friction resistance}$$

The tip or point resistance can be found using Terzaghi's equation that is

$$Q_u = A_p \left(cN_c + \frac{1}{2} BN_g + gLN_q \right) \quad \text{where } A_p = \text{area of pile tip}$$

The frictional resistance of a pile can be estimated as the product of surface area and the friction resistance developed between the soil and pile that is

$$Q_s = \sum p \Delta L f \quad \text{where } p = \text{perimeter of pile section}$$

$$\Delta L = \text{increment of pile length}$$

$$f = \text{unit friction resistance}$$

The capacity of pile in a cohesionless soil can be expressed as

$$Q_u = A_p s' N_q + \sum p \Delta L K_s s' \tan d$$

Where K_s = average coefficient of earth pressure on pile shaft (Table 4.1)

s' = effective pressure along the pile

d = angle of skin friction

(Tomlinson, 1995)

Pile Type	Range of Ks
Concrete	1.5 ± 10%
Pipe	1.1 ± 10%
H-section	1.6 ± 10%

Table 4.1: Different Range of Ks

The capacity of pile in clay soil is given by

$$Q_u = A_p(cN_c + gLN_q) + A_s f_s$$

Where A_s = surface area that develop friction = $P \times L$

f_s = unit skin friction resistance

Pile Load Test

Load tests are performed to check the estimated capacities of the piles and also as a check on piles installed by the contractor. In pile load test, load is applied to the pile by hydraulic jack. Sufficient time is needed to allow for settlement to occur after each load is applied. The settlement is measured by dial gauges. The amount of loads applied depends on local building codes. The pile settlement can be calculated as

$$\text{Net Settlement, } S_{net} = S_t + S_e \quad \text{where } S_t = \text{total settlement}$$

$$S_e = \text{elastic settlement of pile}$$

The values of applied loads, Q can be plotted against settlement in a graph. The load corresponding to the point where the load-settlement curve becomes vertical is the ultimate load, Q_u for the pile. (Das, 2004)

Disturbance due to pile installation will cause remoulding and loss of strength in clay. Therefore testing in clay should be done after several weeks lapse. Meanwhile in

sand, temporary extra resistance is developed when pile is driven. Shortly after the installation, the extra resistance is lost. To obtain a good result, test should only be done at least several days lapse. (McCarthy, 1998)

The test described above is the load-controlled test. Another procedure is the constant-rate-of-penetration. The load-settlement curve is similar to load-controlled test but it is faster to perform. The third method, which is the cyclic loading involves the application and removal of loading repeatedly. (McCarthy, 1998)

The factor of safety (FOS) used should be designed to the extent of soil information and the number of tests performed. If the test results are uniform, then lower FOS can be used. Meanwhile, a higher FOS must be used for greater variation in test results to protect against unexpected poorer soil condition and lesser pile capacity at untested areas.

Pile-Driving Formulae

The formulae involve the effect of hammer blows on the pile and supporting soil. When a hammer strikes on top of a pile, a stress wave caused by the blow is transmitted through the pile. Some of the wave will be absorbed by the soil surrounding it and some by the soil at the tip of pile. This theory is known as the wave equation theory. (McCarthy, 1998)

The wave equation requires the length and weight of pile, cross-section, and pile hammer characteristics. An analysis is done by the computer to determine the effect of hammer blow at a particular time. The penetrations of pile due to the hammer blows are recorded. The pile capacity is then expressed in term of penetration per hammer blow.

Pile capacity can also be found from the work-energy theory. This theory assumed that pile hammer delivers kinetic energy and this energy is the work done on the pile. Energy is loss due to compression of soil, pile and heat generation occurs during pile driving. Therefore it is necessary to include a correction for the energy lost. One of

the commonly used pile driving formula, derived from this theory is the Engineering News formula. (McCarthy, 1998)

The formula stated that:

Energy imparted by hammer per blow = (Pile resistance) x (Penetration per hammer blow)

The pile resistance that is the ultimate load, Q_u can be expressed as

$$Q_u = \frac{W_R h}{s + c}$$

Where W_R = weight of ram
 h = height of fall of ram
 s = penetration per hammer blow
 c = constant; for drop hammer, $c = 25.4\text{mm}$
 for steam hammer, $c = 2.53\text{mm}$

A factor of safety of 6 is required to determine the allowable pile capacity. (Das, 2004)

4.2 Design Procedures

A good foundation design must not only be safe but also minimize the construction costs, time and materials. Curtin, et al, 1994 provides the following procedures to design a good foundation.

- 1) Mark the position of columns and load bearing walls and any other induced loading and bending moments on the building plans. Loads are classified as dead, imposed and wind loads. Adopt a suitable factor of safety for these loads.

- 2) Determine the strength of soil at various depths below the foundation from the site investigation to find the safe bearing capacity. These values will be used to estimate the allowable bearing pressure.
- 3) Determine the invert level of the foundation by either using the minimum depth below the ground level which is unaffected by temperature, moisture content or by the depth of basement.
- 4) Determine the foundation area from the characteristics working loads and allowable pressure. This will determine the type or combination of types of foundation. This selection is based on economic consideration, speed and buildability.
- 5) Determine the various depth of vertical stress and check for possible overstressing of any underlying weak layer.
- 6) Carry out settlement calculations to check whether the total and differential settlements are acceptable. If the settlements are unacceptable, a revised allowable bearing pressure will be determined and foundation size is increase or is taken to a deeper and stronger layer.
- 7) Preliminary costing of alternative design is made before finalizing the choice of foundation type.
- 8) Alternative safe designs are checked for economy, speed and ease of construction.
- 9) Design office should be prepared to amend the design if excavation shows variation in ground condition from those predicted from soil investigation.

4.3 Foundation Failures

A whole building may, to the extent, collapse due to the failures in its foundation system. Therefore, it is important to identify the factors that can cause a foundation system to fail and also the ways to minimize these problems. The major cause for foundation failures especially in expansive soil is due to water. The variation of water level results in settlement and upheaval problem. Besides that, failures may also due to non-moisture factors. (Brown, 1999)



Figure 4.3: Building Settlement

4.3.1 Soil Moisture

Soil moisture is loss through 3 processes that are evaporation, transpiration and combination of both. Evaporation losses can be noticed by the appearance of cracks on the walls. The cracks appearances can be seen in Figure 4.4. The foundation will move up and down during each cycle and the length of foundation will be shorter during wet cycle. The transpiration loss is limited on top shallow soil. The estimation of transpiration loss is difficult because transpiration depends on the type, size and density of plant and also the ambient wind and temperature. (Brown, 1999)



(Source: Hewson, 2004)

Figure 4.4: Cracks on the Wall

The major problem the plants pose to a foundation is the existence of surface roots beneath a shallow foundation. As the root grows, the shallow beam will be jacked upward. Besides that, removing old trees near to a foundation may also causes problem as the decaying roots can results in upheaval and voids which will later resulted in settlement. On the other hand, tree roots can enhance the stability of foundation by increasing the soil resistance to shear. (Brown, 1999) Transevaporation is the combined loss due to evaporation and transpiration, which can cause soil shrinkage and foundation settlement

4.3.2 Other Causes

Lateral movement can occur during soil erosion, sliding or sloughing. The soil movement can cause water to sip into the soil, reducing the cohesion and structural strength of the foundation. Settlement is another problem deals with foundation. Settlement occurs as a result of consolidation or compaction of fill, base or sub-base materials. The settlement crack can be seen in Figure 4.5



Figure 4.5: Settlement Crack

A settlement crack is more likely to be wider at top than its bottom as the foundation bends over a single point to allow for differential settlement. The cracks need to be separated into initial settlement due to construction or site factors and ongoing settlement due to site factors. (Friedman, 2001)

The structural defects due to foundation settlements, as stated by Friedman, 2001, can be observed as follows:

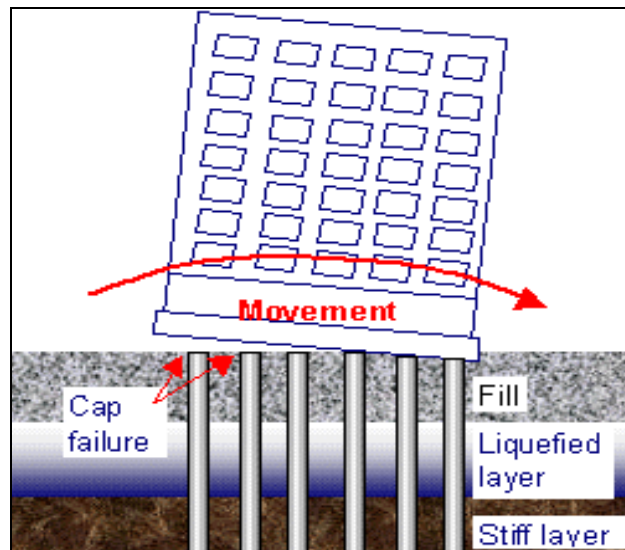
- Vertical crack patterns – straight or wandering with even width and in wall the cracks are wider at bottom than the top
- Diagonal and Step crack patterns - from corner towards adjacent opening
 - wider at top than bottom
 - cracks appear in short time if the building settles over a sinkhole

Besides that, construction practices can also cause problems to the building and foundation. Some of the practices and its effect are shown in Table 4.2.

Construction Practices	Effects
Pouring the slab foundation off-grade	Interior surfaces swell and appearance of cracks in sheetrock.
Improper reinforcement	Poor quality of concrete with substantial loss in strength and encourage foundation problem.
Non-consistent watering	Promote foundation problem
Vibrate concrete in pile shaft	Causing segregation

Table 4.2: Foundation Problems Causes and Effects

The failure of a deep foundation may also due to the failure of its pile cap. The failure movement can be seen in Figure 4.6. Besides that, the failures of a pile may due to the errors in installing the pile into the ground surface. Figure 4.7 shows the problems in installing piles at limestone area.



(Source: Johansson, 2000)

Figure 4.6: Deep Foundation Failure Movement

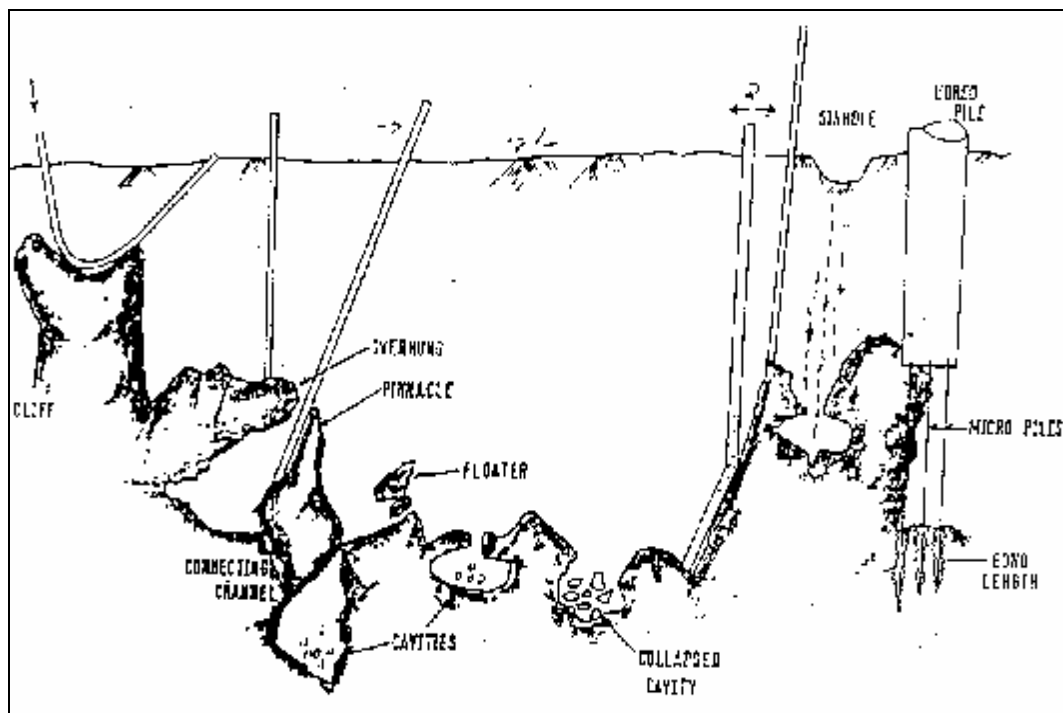


Figure 4.7: Problems in Pile Foundations

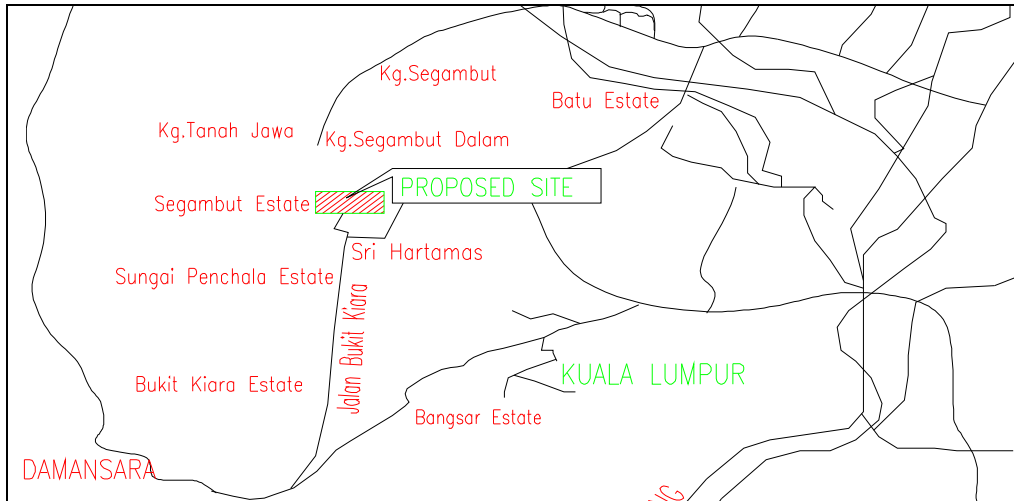
CHAPTER 5

CASE STUDIES

5.1 Case Study 1

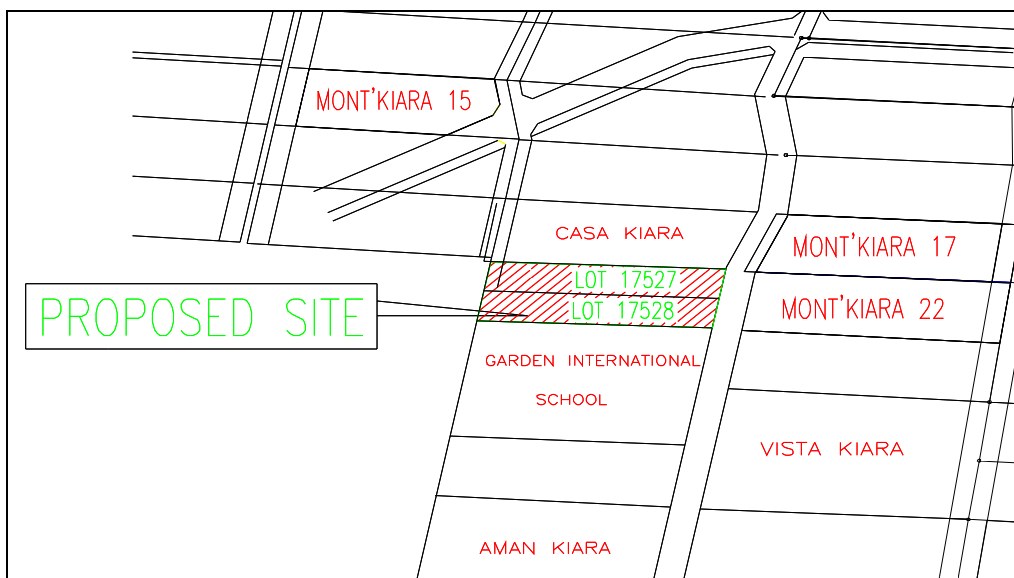
This case study is about the development of 29-storey condominium and a 2-storey service apartment on a 6-storey car park on Lot 17527 and 17528, Mukim Batu, Mont Kiara, Kuala Lumpur. The layout plan of the site is shown in Figure 5.1. Total land area of the site is 2.77 acres. (SP, 2004) The location of the site can be seen in Figure 5.2

There is a sloping ground located at the western portion of the site and the proposed condominium block is located at this area. The car park block will be constructed on an existing partially completed basement located at the east of the site. The highest area is at the western side with RL 80m and the lowest area is at the northern part with RL 59m. The difference of elevation is 21m. (SP, 2004)



(Source: SP, 2004)

Figure 5.1: Layout Plan for Mont Kiara Development



(Source: SP, 2004)

Figure 5.2: Location Plan for Mont Kiara Development

There is an ongoing construction site (Casa Kiara) located at the northern part and a school (Sri Garden International School) located at the southern boundary of the site. In addition, there is a partially completed basement within the site and this basement contains stagnant water of about 1.5m deep. The western part of the site is a sloping ground and some rock boulders were observed here as shown in Figure 5.3. The

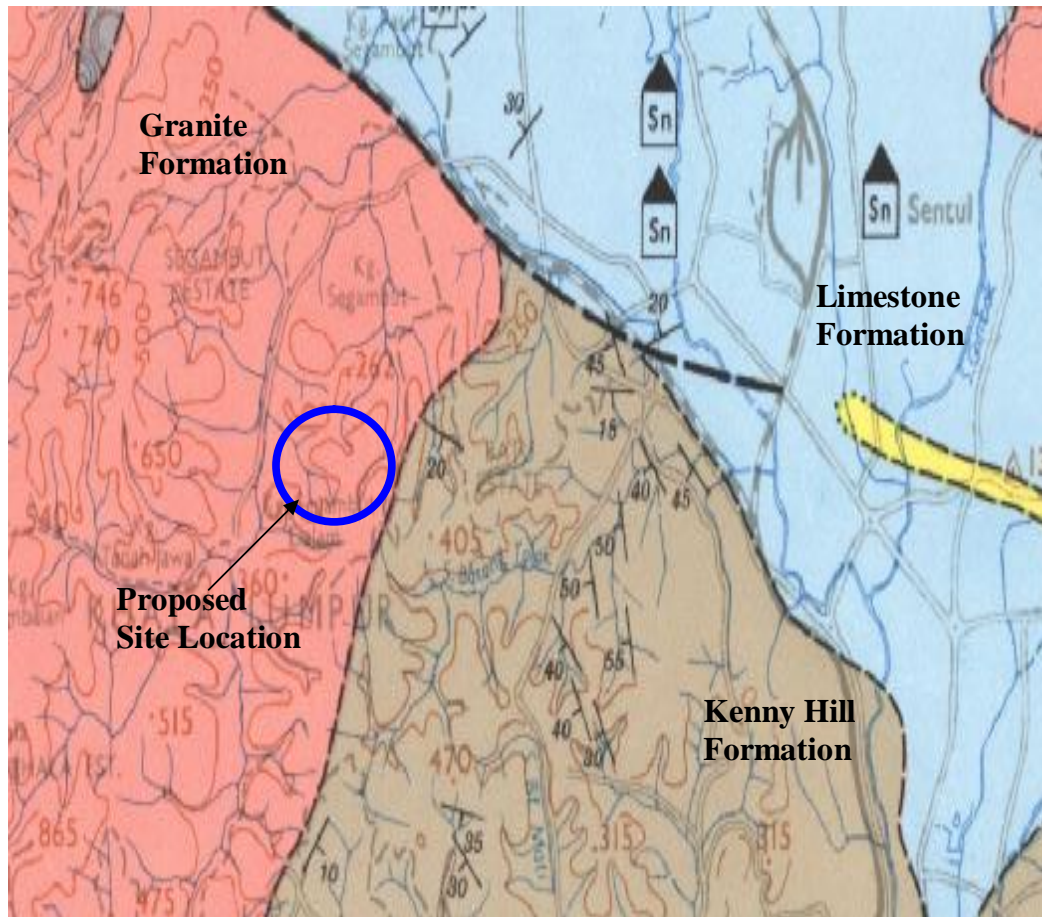
existing of boulders requires careful design and supervision of foundation works to provide a safe foundation for the structure. (SP, 2004) The intermediate boulders may cause false set to deep foundation systems such as driven or jack-in piles. The depths of the bedrock at that site vary from RL 40m to 60m.



(Source: SP, 2004)

Figure 5.3: Rock Boulder

According to the geological map of Selangor, published by Geological Survey Department Malaysia, the site is underlain by Granite formation. A part of the geological map is shown in Figure 5.4.



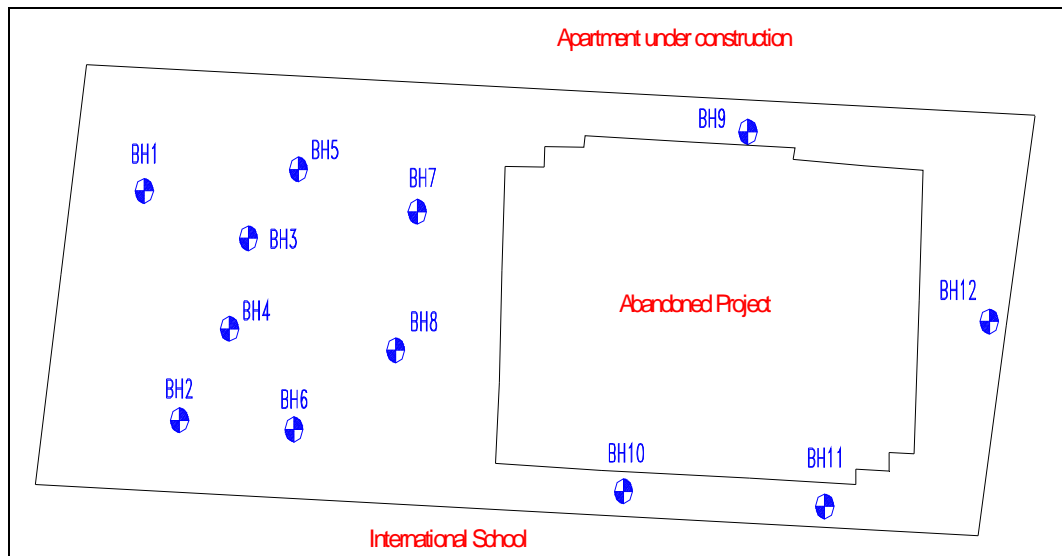
(Source: SP, 2004)

Figure 5.4: Geological Map

A total of twelve boreholes were sunk within the boundary of the proposed site. In addition, 5 observation wells were installed in selected boreholes to monitor the groundwater level. The locations of the borehole are shown in Figure 5.5. The depths of the boreholes range from 14.1m to 33.0m. (SP, 2004) In-situ Standard Penetration Test was carried out in every borehole at 1.5m intervals. Disturbed and undisturbed samples were collected from various depths of the boreholes for laboratory tests. The laboratory tests conducted are the moisture content, Atterberg Limits, Particle Size Distribution, undrained triaxial test and also the soil chemical test.

From the Particle Size Distribution test, the subsoil at that site consists 15% of clay fraction, 30% silt size particles, 40% of sand size particles and 15% of gravel size particles. The soil is mostly belong to sandy clayey SILT group. The natural

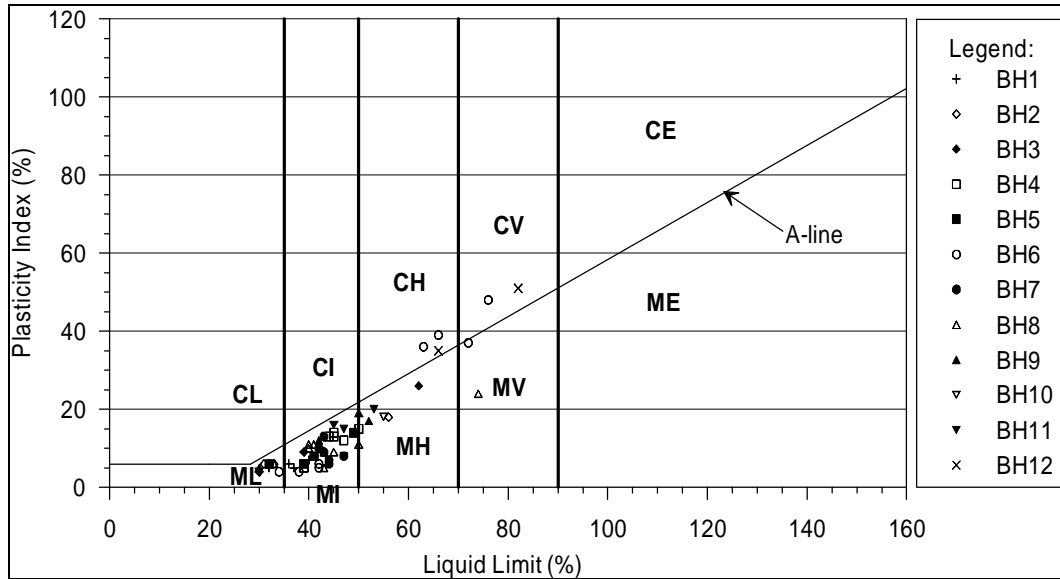
moisture content of the soil ranges from 20% to 34% (SP, 2004) The Atterberg Limits as can be seen in Figure 5.6, indicate that the soil is of intermediate plasticity with Liquid Limit and Plasticity Limit range from 30% to 80% and 3% to 38% respectively.



(Source: SP, 2004)

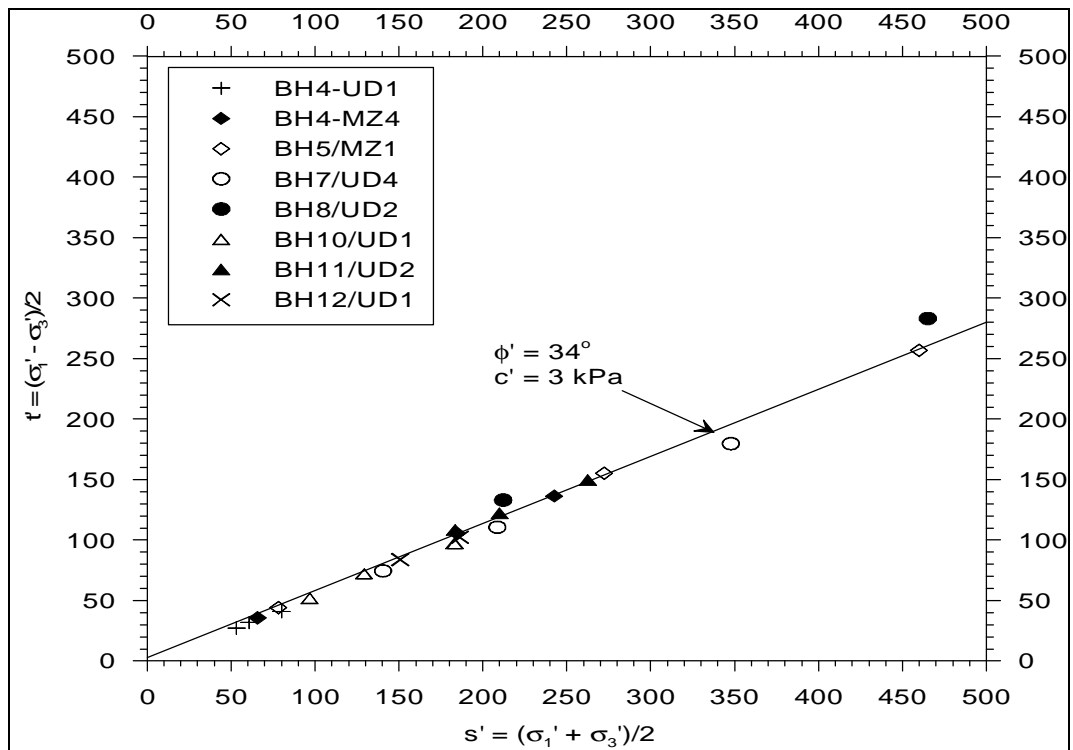
Figure 5.5: Location of Boreholes

8 undrained triaxial tests were carried out to determine the effective strength of the subsoil. The results of the tests are plotted in Figure 5.7. From the graph, the soil shear strength parameters are $c' = 3 \text{ kPa}$ and $\Phi' = 34^\circ$.



(Source: SP, 2004)

Figure 5.6: Plasticity Chart



(Source: SP, 2004)

Figure 5.7: Undrained Triaxial Test Results

Chemical tests were also conducted to determine the aggressiveness of the soil reacting with the construction materials such as concrete and steel. The results from the tests can be seen in Table 5.1.

Borehole no.	Sample no.	Depth (m)	Organic Content (%)	Sulphate Content (%)	Chloride Content (%)	pH Value
BH 1	M4	13.5	0.1	< 0.01	< 0.01	7.3
BH3	M2	12	-	-	-	6.9
BH3	M4	18	-	< 0.01	< 0.01	-
BH4	UD1	3	-	< 0.01	< 0.01	7.4
BH4	M4	21	0.1	0.02	< 0.01	6.6
BH5	M1	10.5	-	-	-	7.2
BH5	M2	13.5	-	-	< 0.01	-
BH5	M3	16.5	-	< 0.01	-	-
BH7	UD1	3.0	-	< 0.01	-	-
BH7	UD2	10.5	-	-	-	6.4
BH7	UD5	19.5	-	-	< 0.01	
BH8	UD1	3.0	0.1	0.02	< 0.01	7.3
BH9	UD1	3.0	-	-	-	7.3
BH9	UD2	6.0	-	-	< 0.01	-
BH10	UD1	4.5	-	< 0.01	-	-
BH11	UD1	4.5	0.2	0.02	< 0.01	6.1
BH12	UD2	6.0	-	< 0.01	-	-

(Source: SP, 2004)

Table 5.1: Chemical Test Results – Case 1

The column loads of the condominium tower block range from 648kN to 26 265kN. (SP, 2004) Since the subsoil encountered at the site is relatively weak and the bedrock is relatively shallow, deep foundation are used to transfer the column load to the bedrock underlying the site.

Since the proposed site is located at a residential area and next to the Sri Garden International School, hence noise and vibration impacts caused by the construction activities are required to be maintained at a minimum level to minimize the

disturbance to adjacent buildings. The jack-in pile system is used as it generates minimum noise and vibration during the pile installation compared to the usual use bored pile. Furthermore, the installation of jack-in pile is faster and economical than the bored pile. Pre-stressed spun piles are adopted for this foundation system as it has better quality control in which lesser piles will be broken during jacking and pile handling and also it has higher lateral resistant.

The jack-in pile capacity is designed to its structural capacity. The length of the pile penetration will be determined based on the capacity that can provide the pile working load with minimum factor of safety of two. Piles are designed as end bearing piles because no intermediate hard layer was encountered. Since the column loads are small and the tower footprint area is relatively small, larger diameter pile is required to avoid congestion of piles within the tower area. Pile of 500mm diameter and grade 80 concrete are adopted for the construction of the foundation. (SP, 2004)

The car park block was constructed on the partially completed basement. The proposed column loads range from 899kN to 10 931kN. Some of the estimated column loads have exceeded the estimated capacity of the existing foundation of the partially completed basement. Therefore additional piles are required to strengthen the foundation of this partially completed basement. Micropiles are chosen because of its cost effective and its ease of construction. (SP, 2004)

The micropiles used are 300mm in diameter and rock socket with 5m length is required to provide a pile capacity of 1 300kN per pile. The additional micropiles and the existing piles are cast under a common larger pilecap to form the permanent foundation for the car park. The capacity of the existing pile is 1 100kN but was downgraded to 80% of its original capacity to cater for the degradation of the pile capacity as the piles were installed long time ago. The number of micropiles required was determined based on the difference between the proposed column load and the downgraded capacity of the existing piled foundation. (SP, 2004)

A careful design and selection of foundation system are needed as this development was built on undulating terrain. The selection will also need to consider the surrounding structures at the site which include a school. The noise level is needed to

be kept as low as possible to minimize the noise disturbance to the students at that school. A good selection will need to consider all those criteria.

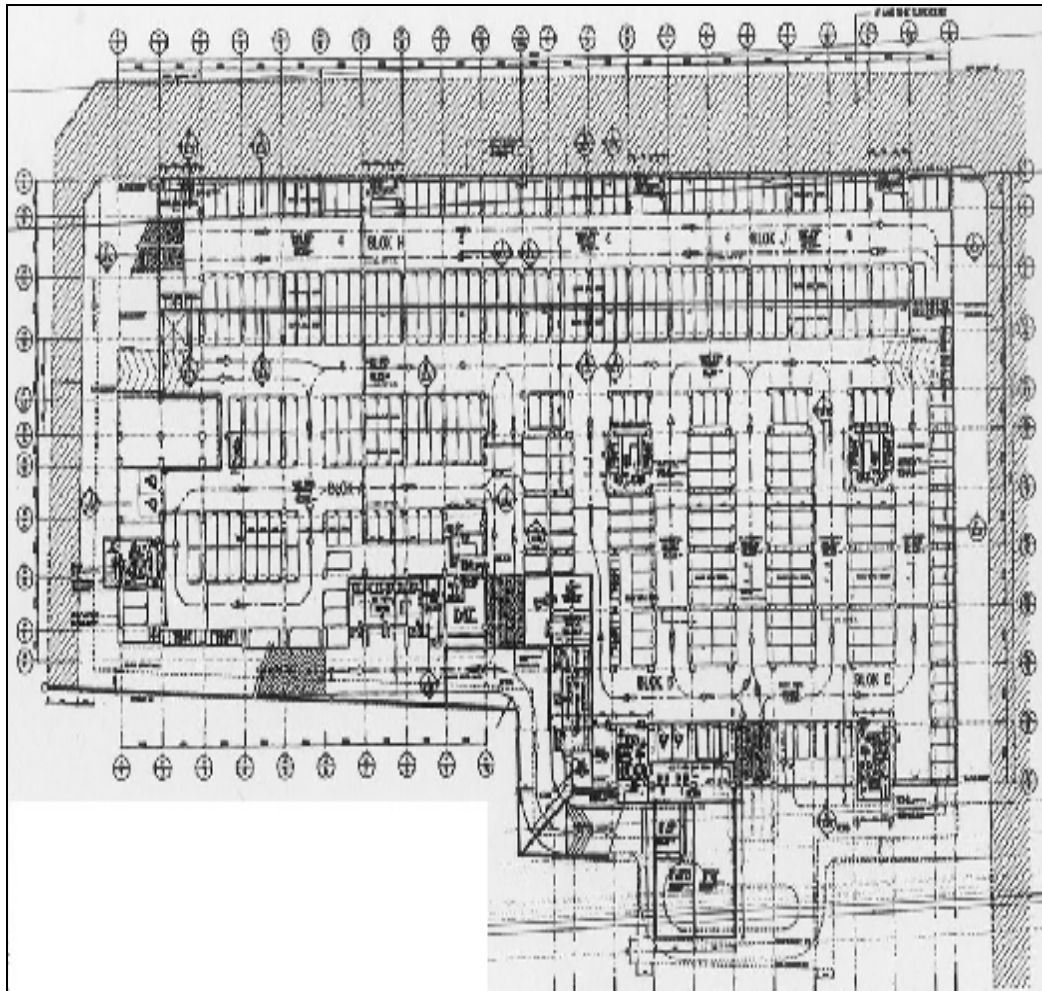
5.2 Case Study 2

Case study 2 is about the development of three blocks of 17-storey apartments in Lot 13, Batu 2 ½ Jalan Cheras, Kuala Lumpur. The location and layout plan of the site are shown in Figure 5.8 and Figure 5.9 respectively.



(Source: B, 2005)

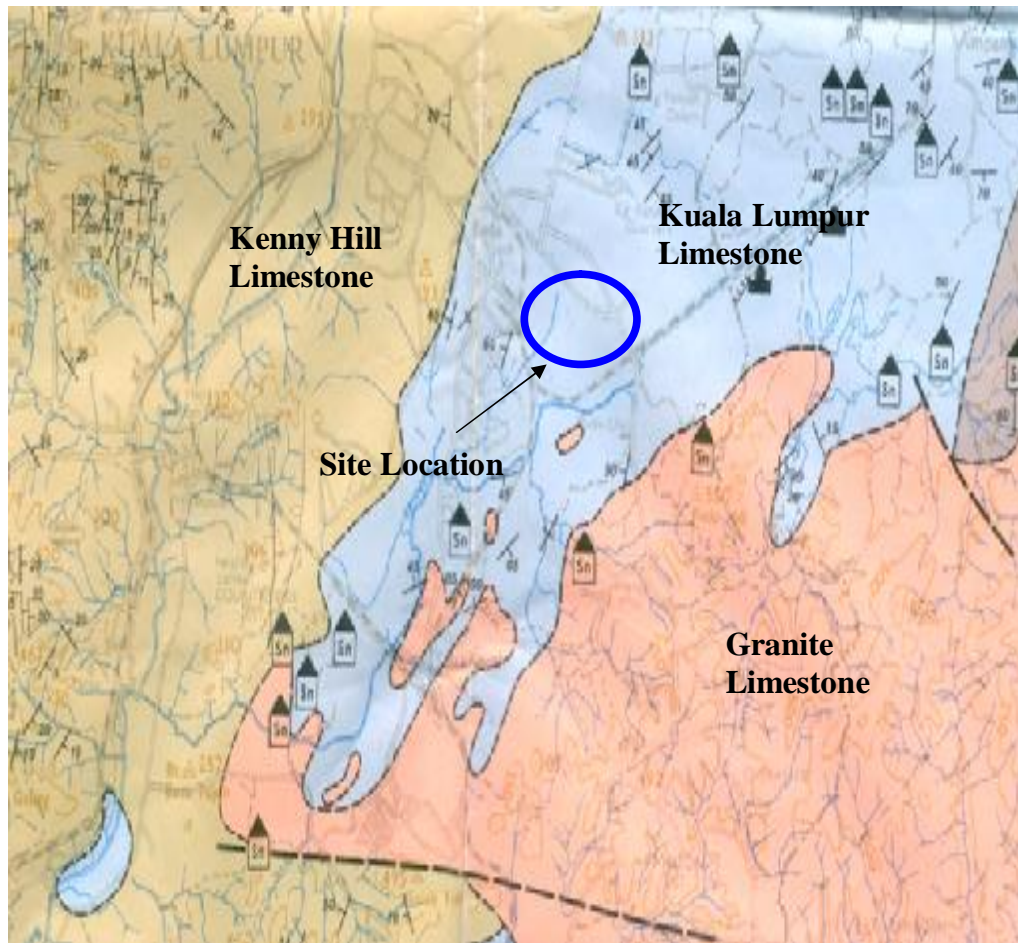
Figure 5.8: Location Map of Jalan Cheras Development



(Source: B, 2005)

Figure 5.9: Layout Plan of Jalan Cheras Development

From the Geological Map of Selangor, the site is underlain by limestone bedrock. The limestone bedrock is covered by alluvial deposits. This river alluvium consists of sand, gravel, clay and some organic materials. The general geology of the site is shown in Figure 5.10



(Source: B, 2005)

Figure 5.10: General Geology of Jalan Cheras Development

The existing ground is generally flat at about 39m and is overlain by mixture of soil and construction debris. The overview of the site is shown in Figure 5.11

The soil investigation is carried out with the boring of 26 boreholes. The locations of these boreholes are shown in Figure 5.12. From the borehole logs (refer to Appendix), we can see that the materials overlying the limestone bedrock consist of soft Silty Sandy CLAY and Silty SAND (B, 2005) The water level varies from RL 34.7 m to RL 39m. The soil testing carried out at this site include the In-Situ Standard Penetration Test, Triaxial Test, moisture content test, Atterberg Limit test and also the particle size distribution test.

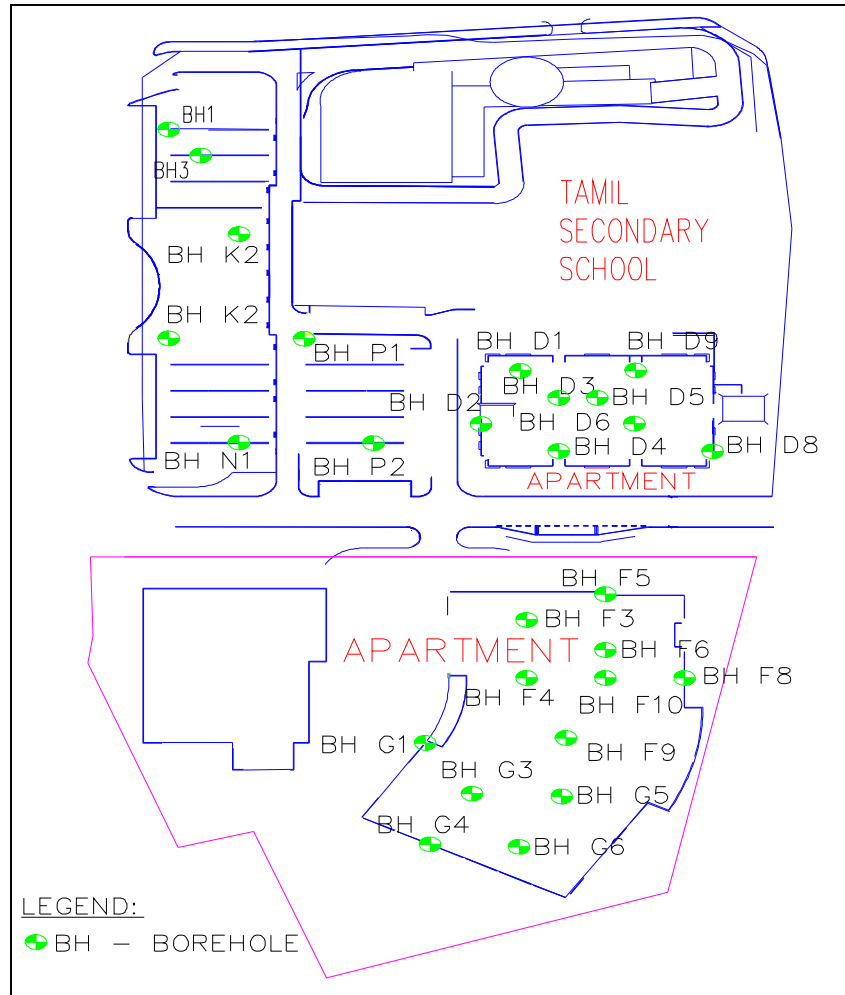


Figure 5.11: Existing Site Overview

The column loads of the apartments range from 2 200kN to 19 950kN. The columns are laid out on 8m square grid. The finish floor level of the lowest basement level is at RL 37.5m. (B, 2005)

Shallow foundations such as pad footing, strips and rafts are used. Since the columns are on wide 8m grids, pad footings are adequate and are recommended for area with shallow bedrock. The pad footings are founded on the limestone bedrock because of the high bearing capacity of rock and thus reducing the size of the pad footings required. The pad footings are socketed 200mm into the limestone bedrock for lateral stability. For practical and feasible construction, the depth of footing is limited to 2m below the basement excavation level. The pad footings are used in areas with not more than 2m depth from the temporary working platform level.

Deep foundation such as bored piles is used in areas where the bedrock is deep and where shallow foundations are not recommended. The bored piles lengths range from 0.4m to about 1.5m. (B, 2005) Bored piles are constructed by boring through soil and rock to form an empty hole. Steel reinforcements are lowered into the bored hole and subsequently, tremie concreting is done to the required cut-off level. The bearing capacity of bored piles is through the shaft friction of soil or rock. The bored piles sizes used in this project are 800mm, 1050mm, 1200mm and 1500mm in diameter (B, 2005)



(Source: B, 2005)

Figure 5.12: Location of Boreholes in Jalan Cheras

Since the site is located on limestone bedrock which is known for its karstic features, cavities problem caused by acidic solution, exist in certain location. Presence of large cavities can endanger the stability of the building structure, therefore cavity probing is recommended to determine the extent of the cavities. Cavities problem can be treated by using compaction grouting, pressure grouting or filling with aggregates and then grout, depending on the size of the cavity (B, 2005)

Generally, grouting is done by replacing the slump or soft materials in the infill cavities with stiffer materials such as cement mortar, grout and aggregates so that the imposed load at the rock roof of the cavity can be transmitted through the stiffer

materials and avoid the collapse of thin roof. Mortar or grout mix or neat cement can be used as filling materials where voids presence in the cavities.

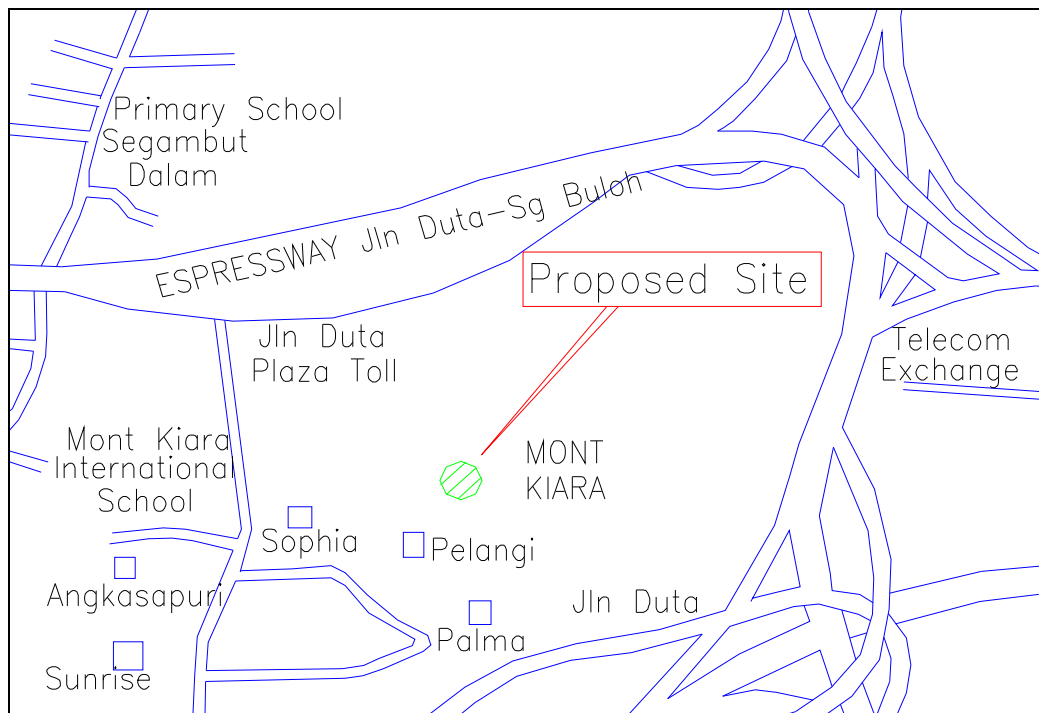
At cavity areas, micropiles are used instead of the proposed bored piles. Micropiles, which are 300mm in diameter, are bored through the cavity and further into the limestone bedrock where no cavities are detected by the micropile boring. (B, 2005) Reinforcement is lowered into the bored hole and followed by grouting.

Since the capacity of micropile is much lower than the bored pile, large numbers of pile and larger pile cap are needed. In addition, if cavity is detected by the micropile boring, cavity treatment is still required to prevent buckling of the reinforcement within the cavity. Compaction grouting is recommended over micropiles due to its cost effectiveness and ease of construction.

The selection of foundation in this case study is mainly due to the soil condition at that site. One of the issues in the case is the presence of cavities at certain areas. This problem must be treated because the presence of cavities may affect the stability of the building.

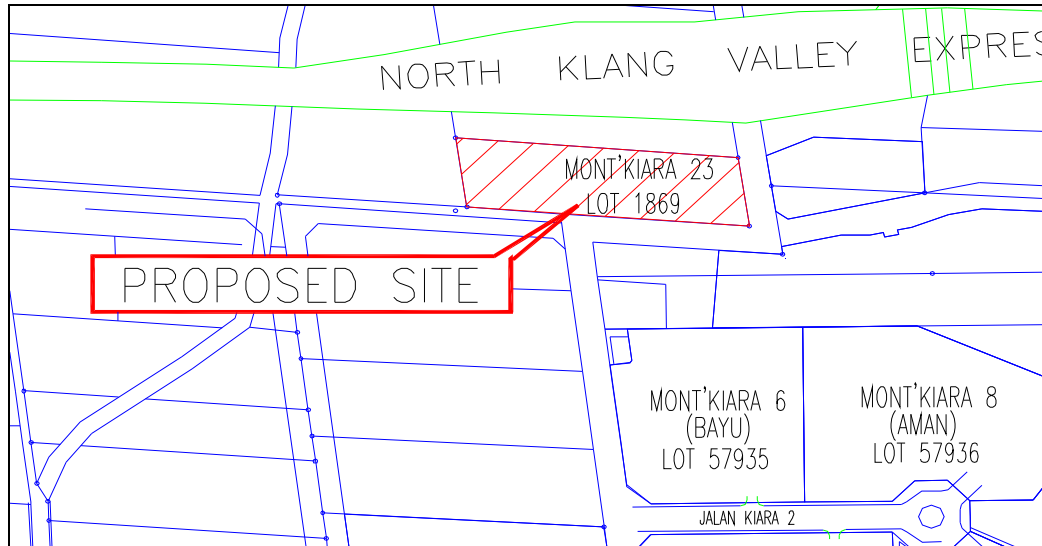
5.3 Case Study 3

This case study is about the development of a 31-storey condominium tower and a 2-storey condominium on top of a 6-storey basement carpark on Lot 1869, Mukim Batu in Kuala Lumpur town. The total area of the site is 2.812 acres. The location and the layout plan of the site are shown in Figure 5.13 and Figure 5.14 respectively. The site is located adjacent to the North Klang Valley Expressway (NKVE).



(Source: AA, 2005)

Figure 5.13: Location of Mukim Batu Development

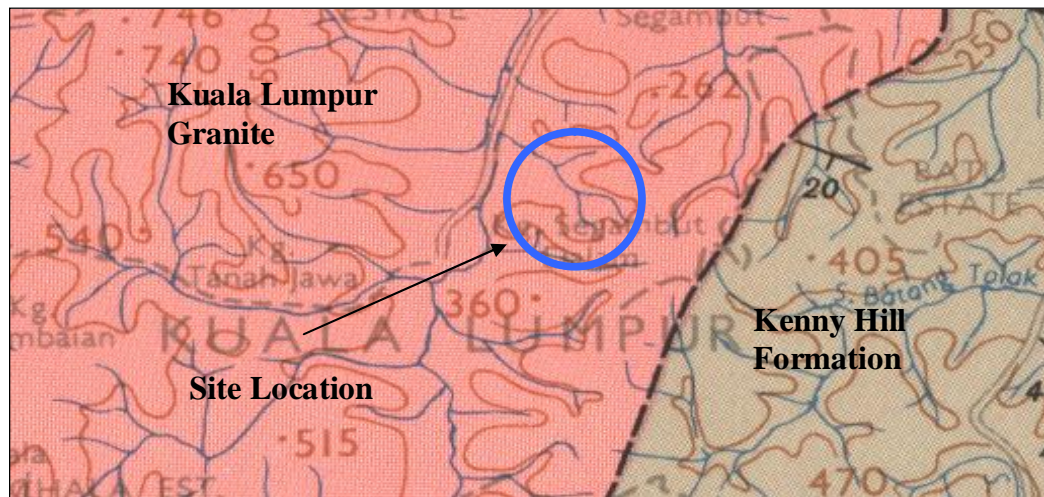


(Source: AA, 2005)

Figure 5.14: Layout Plan of Mukim Batu Development

Based on the Geological Map of Selangor, the site is underlain by the Kuala Lumpur Granite formation. The granitic rock ranges from coarse to very coarse-grained and is pale to grey in colour. The overburden materials, 15 to 45m thick, consist of residual soils which are derived from the weathering of granitic rock. The overburden materials are silty SAND and sandy SILT which are yellowish to reddish in colour. Gravels and cobbles can also be found at certain depths. The general geology of the site is shown in Figure 5.15

A site investigation was carried out with 22 boreholes being sunk 14m to 52m within the boundary of the site. The boreholes profiles are shown in the Appendix. 3 of the 22 boreholes were observation wells, where the water levels at the site are being monitored. The ground water level is approximately RL 57m. Generally, intermediate hard layers present at RL 52m and have a vertical thickness of 3m to 4.5m. (AA, 2005)



(Source: AA, 2005)

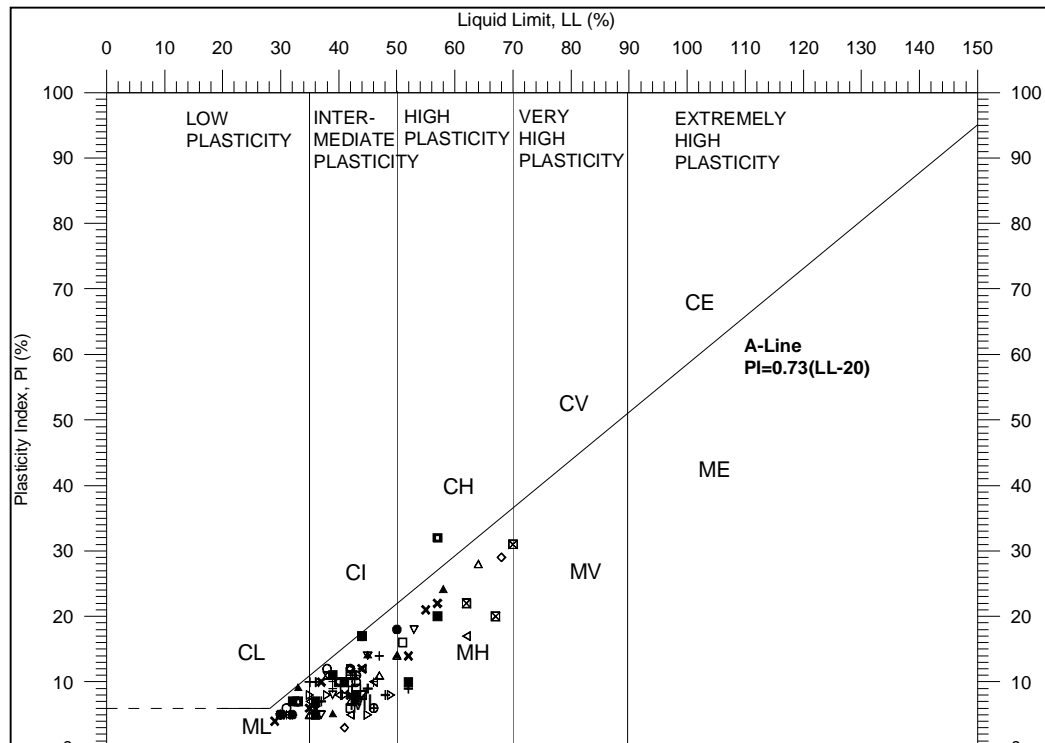
Figure 5.15: General Geology Map of Mukim Batu Development

Relatively softer layers are found under the intermediate layers. The presence of intermediate hard layers makes it difficult to penetrate the soil using driven or jack-in piles. It may result in the piles being falsely terminated on top of the intermediate layer. Consequently, the high building loads are transferred to the softer layers which underlain the intermediate hard layers. This will lead to building settlement as the loads are not properly transferred to the actual founding level. In order to prevent the settlement of the building and to ensure that the piles are terminated at the actual founding level, pre-boring of the localized area with intermediate hard layers are necessary to provide a safe and economical foundation systems for the buildings.

The soil testing carried out at the site are the Atterberg Limit, Triaxial Test, soil chemical test and other. The Atterberg Limit test as shown in Figure 5.16 indicates that the soils which are silty in nature are of intermediate plasticity with Liquid Limit and Plastic Limit range from 30% to 68% and 3% to 32% respectively. (AA, 2005)

The density of the soil ranges from 17.8 kN/m³ to 20.2kN/m³. A total of 10 undrained Triaxial tests were carried out on the undisturbed soil samples and the effective soil parameters found are $c' = 3\text{kPa}$ and $\Phi' = 32^\circ$. The result can be seen in

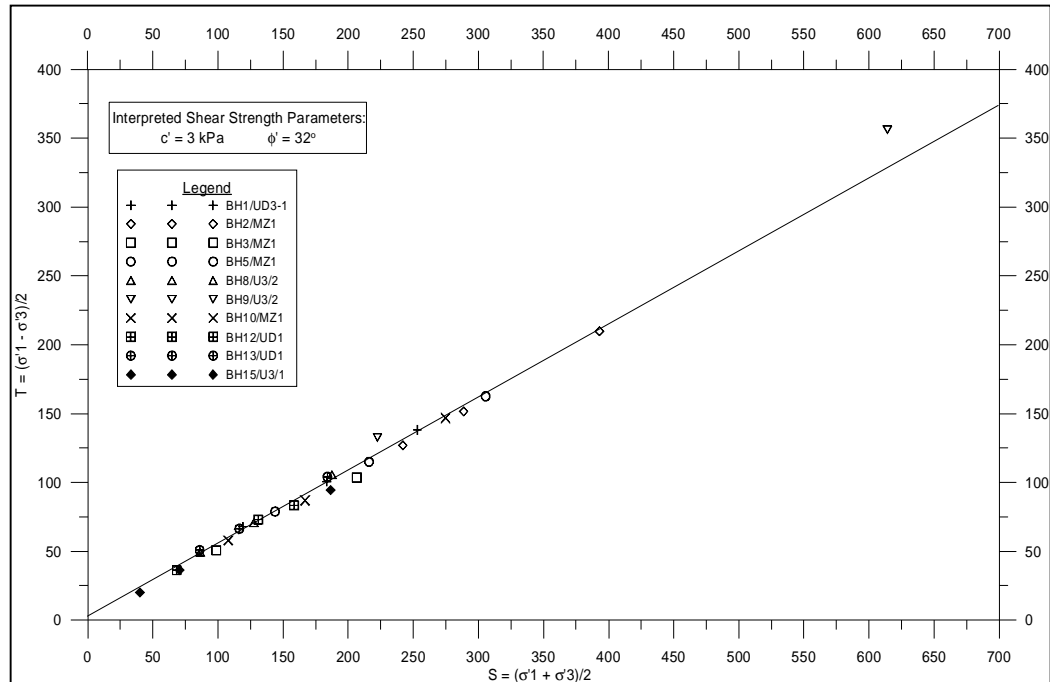
Figure 5.17. The compression test carried out showed that the compression strength of the rock cores ranges from 13MPa to 65MPa. (AA, 2005)



(AA, 2005)

Figure 5.16: Atterberg Limits Result

Based on the soil chemistry tests as can be seen in Table 5.2 indicate that the soil is of 5.3 to 7.6 pH values. The total Sulphate and Chloride content range from 0.01% to 0.03% and 0.01% to 0.05% respectively. (AA, 2005) Overall, the soil can be classified as non-aggressive soil. Thus, normal Portland cement can be used for these soil conditions and hence can also be used for the foundation works.



(AA, 2005)

Figure 5.17: Undrained Triaxial Test Result

Borehole	Sulphate Content (%)	Chloride Content (%)	pH Value
BH-1	0.01	0.05	5.6
BH-2	-	0.03	5.4
BH-3	<0.01	0.05	5.8
BH-4	<0.01	0.03	5.3
BH-5	<0.01	0.04	5.7
BH-6	<0.01	0.05	5.6
BH-7	<0.01	0.05	6.7
BH-8	0.01	<0.01	6.7
BH-9	<0.01	0.03	5.4
BH-10	<0.01	0.05	5.6

(AA, 2005)

Table 5.2: Chemical Results – Case 3

During the construction of the buildings, the site was unoccupied. The eastern part of the site was bare vegetation and formed a steep decline, from RL 73m to RL 17m, leading to the lowest point of the site. The western portion is located in higher ground and was covered by thick undergrowth. (AA, 2005)

The selection of foundation in this project depends on many factors such as the soil profile, intensity of column loads, building settlement, topography, disturbance to surrounding site area and other. Generally, footings and rafts are suitable for low-rise building on cut ground and piled foundations are suitable for medium to high rise buildings.

Since the columns loads of these 2 condominiums are high and also the stiff soil stratum which is about 20m to 40m below the proposed finished levels for the tower block and about 10m to 25m below the proposed finished levels for the carpark block, jack-in piles are recommended for both the structures. The pre-stressed spun piles are used for these structures as the piles can provide an allowable structural capacity of about 88kN to 2100kN with sizes range from 300mm to 500mm diameter with grade 80 concrete. (AA, 2005) The pre-stressed spun piles have better quality control and higher lateral resistance of the piles which can reduce the pile damage and wastage during pile installation and handling.

The site is located within a residential area with a school nearby, therefore the piles are chosen to be installed by using a large hydraulic jack to push the piles into the ground to set at the final founding levels. A total jack-in load of 2.2 times of the working axial pile capacity was used to jack the piles into the ground until it sets in hard layer and to ensure that all the installed piles have achieved the required factor of safety of two. The estimated pile lengths are between 8m to 36m depending on the bearing stratum.

500mm diameter spun piles are used for the 31-storey condominium tower block and are designed to its full bearing capacity. However, in order to design the piles to its full capacity within the localized areas where intermediate hard layers were detected, pre-boring work is required.

Meanwhile, for the 2-storey condominium on top of the carpark, a combination of 350mm and 400mm diameter spun piles is used. The capacities of the piles are downgraded to 80% of their structural capacities to accommodate for the negative skin friction on the piles. The allowable working load shall be reduced to 900kN, 1200kN and 1500kN for the diameter of 350mm, 400mm and 500mm piles respectively. Since the carpark are seated on filled ground of 6m, the lowest floor of the carpark are designed as suspended base. (AA, 2005)

5.4 Case Study 4

This case study is about the development of two 35-storey serviced suites and 3-levels of basement carpark on Lots 43, 44, 133 and 135, Seksyen 58, Jalan Ampang, Kuala Lumpur. The location of the site is shown in Figure 5.18.



(Source: WT, 2005)

Figure 5.18: Location of Jalan Ampang Development

The site is located in the hub of Kuala Lumpur city in the intersection of Jalan Ampang and Jalan Sultan Ismail. The site is surrounded with hotels (Concorde Hotel and Renaissance Hotel), a monorail station (Bukit Nanas Monorail Station) and a

tourism centre. (WT, 2005) Before the construction begin, the site was originally used as parking spaces as shown in Figure 5.19 and Figure 5.20

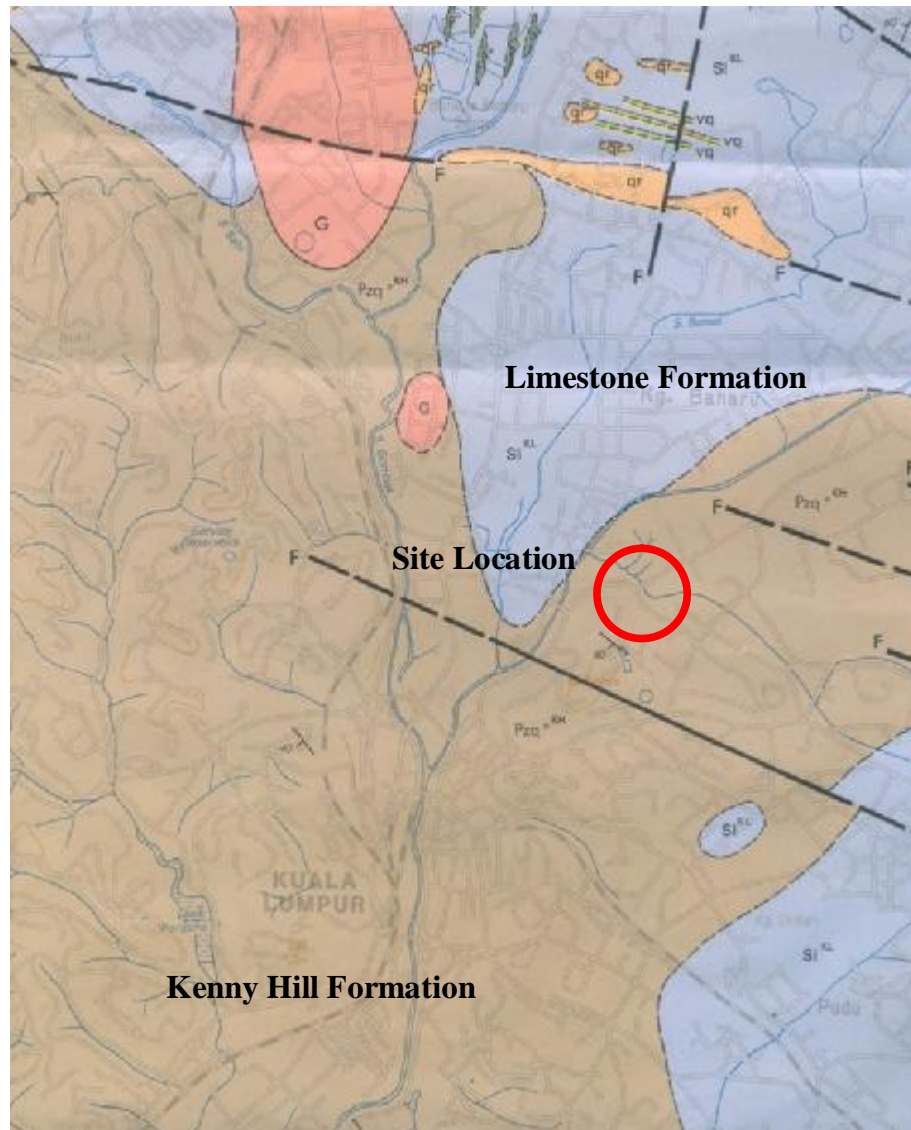


Figure 5.19: Overview of Existing Site (From Monorail View)



Figure 5.20: View of Existing Site

The geological map of Selangor indicates that the site is underlain by Kenny Hill Formation. The sub-soil consists of sandy SILT with some layers of silty SAND. (WT, 2005) The general geology of the site is shown in Figure 5.21



(Source: WT, 2005)

Figure 5.21: General Geology of Jalan Ampang Development

The site is generally a flat ground with an existing level of RL 32.5m. A soil investigation is carried out using 10 boreholes. The boreholes were sunk using rotary wash boring. Field testing such as the Standard Penetration Test (SPT) and in-situ permeability tests were done in the boreholes. The selected disturbed and undisturbed soil and rock samples were collected at various depths for further soil testing such as the moisture content, Particle Size Distribution, Atterberg Limit test, Triaxial test and soil chemical tests.

From the subsoil profile shown in Appendix, the subsoil consists of Kenny Hill residual soil which consists of very loose to loose sandy materials from the depth 5m to below 60m. No bedrock was encountered in the boreholes up to 60m depth. The Atterberg Limit test indicates that the soil is of low percentage of fine materials (silt or clay) with low to medium plasticity. The Liquid Limit is between 30% and 40% and the Plasticity Limit is between 5% and 10%. The natural moisture content of the soil is less than 35%. The density of the overburden soil ranges from 18kN/m³ to 21kN/m³. (WT, 2005)

The Particle Size Distribution indicates that the soil consists mainly of SILT and SAND. The in-situ permeability test carried out indicated that the soil is from 10⁻⁵ to 10⁻⁶ m/s which is also in the range of sand-silt. 10 undrained triaxial tests were carried out to determine the shear strength parameters of the soil. The chemical tests done indicated that the soil is of 4.5 to 8.5 pH values and the amount of organic present is less than 0.1% which is insignificant. The total sulphate and chloride content are less than 0.01%. Therefore, the soil can be classified as non-aggressive soil.

Due to highly sensitive buildings around the site, further testing such as Static Load Test, High Strain Dynamic Load Test, Pile Integrity Test, Extensometer and Sonic Logging Test are necessary to verify the performance of the foundation system.

Bored-pile foundation system is used for this development, which utilizes a combination of large diameter piles that are 900mm, 1500mm, 1800mm, 2500mm and 3000mm diameter to support the loads from the columns, walls and lift cores. (WT, 2005) The following criteria have been adopted for the pile foundation design:

- The thickness of the lowest basement slab is designed to 900mm thickness to overcome groundwater pressure.
- Individual piles are designed to achieved settlement design working load lesser than 12.5mm and the settlement of the test pile at the failure load shall be equal to 10% of the effective pile diameter.
- The concrete grade for all the bored piles shall be 35Mpa.

Based on BS8004:1986, when Grade 35 concrete is used for the bored piles, the allowable compressive strength of the pile under working load shall not exceed 25% of the specified concrete cube strength result at 28 days. Therefore, the allowable compressive stress for all the piles under working load is limited to 8.75Mpa. (WT, 2005)

The allowable structural capacity for various pile sizes are shown in Table 5.3. A factor of safety of 2 is adopted in bored pile design to accommodate for the skin friction effect.

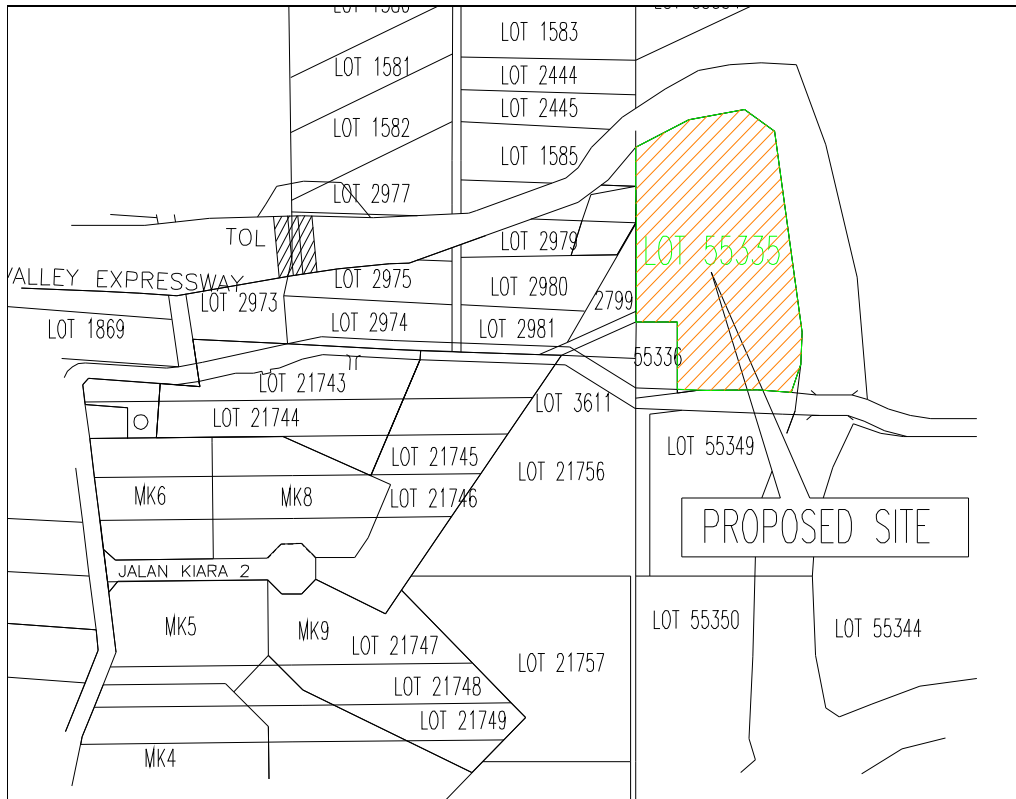
Pile Diameter (mm)	Pile Type	Allowable Structural Capacity (kN)	Adopted Working Capacity (kN)
900	P900	5566	4770
1500	P1500A	15462	7500
	P1500B	15462	13500
1800	P1800	22266	20000
2500	P2500A	42951	27000
	P2500B	42951	32000
	P2500C	42951	38500
3000	P3000A	61850	44800
	P3000B	61850	51000
	P3000C	61850	58500

(Source: WT, 2005)

Table 5.3: Allowable Structural Capacity for Various Pile Sizes in Malaysia.

5.5 Case Study 5

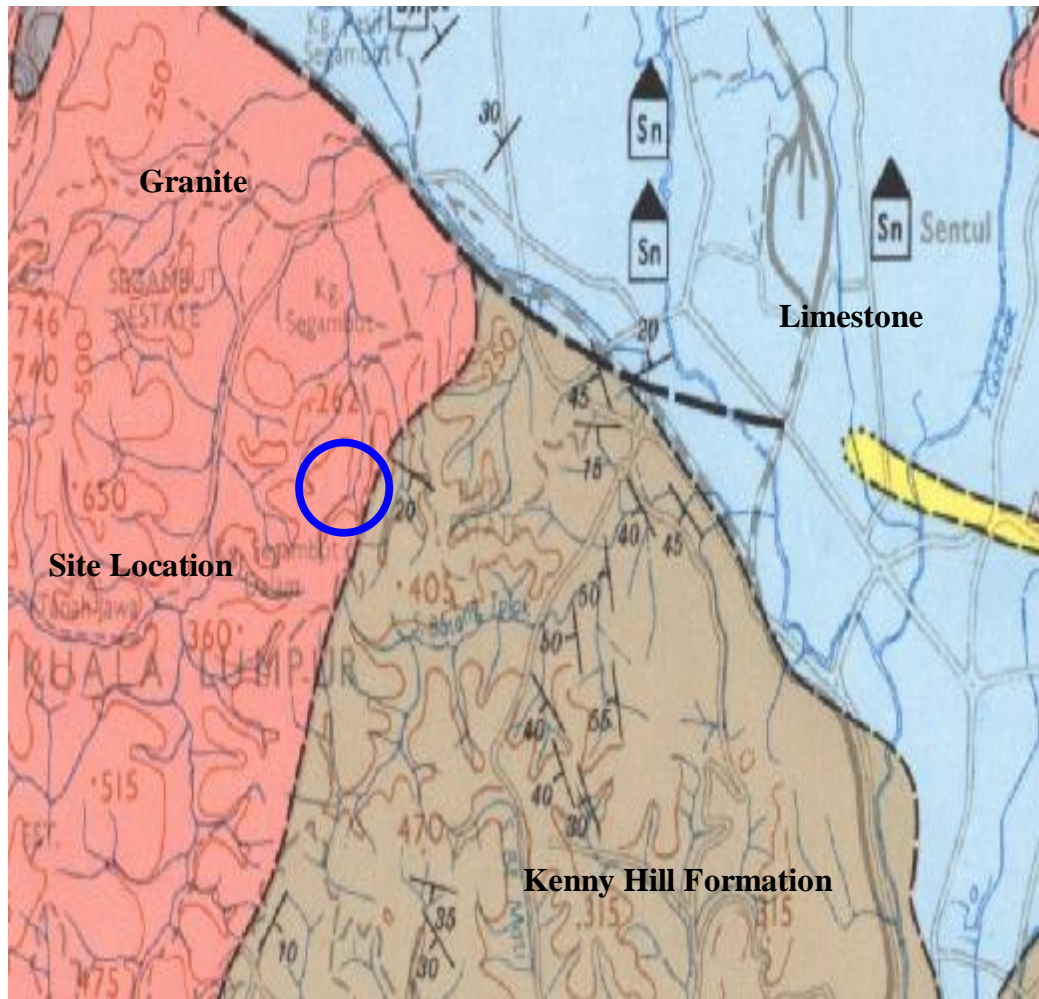
This case study is about the development of 5-storey shoplots on Lot 55335 in Mont Kiara, Kuala Lumpur. The layout plan for the site is shown in Figure 5.22. The total land area of the site is 12.71 acres. The site is located on an undulating terrain that requires cut and fill earthworks, erection of earth retaining structures and formation of cut and fill slopes. (SC, 2005)



(Source: SC, 2005)

Figure 5.22: Layout of Shoplots Development

According to the Geological Map of Selangor, the site is underlain by granite and is close to Kenny Hill Formation. A part of the geological map of that area is shown in Figure 5.23



(Source: SC, 2005)

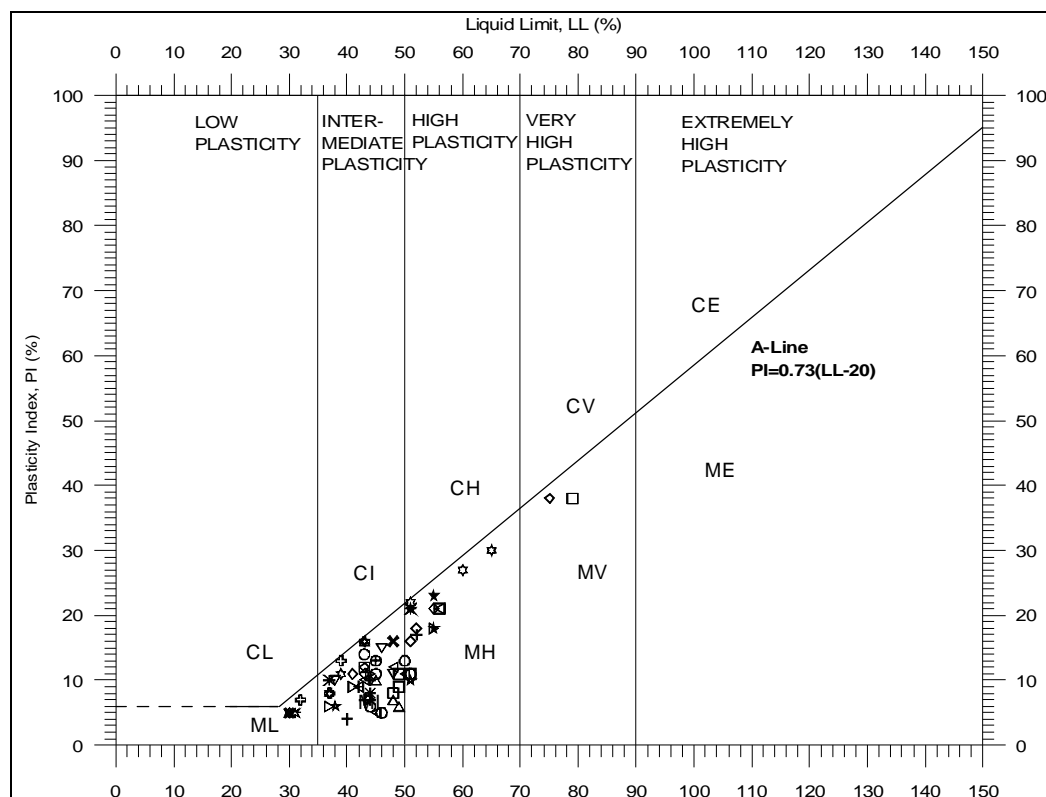
Figure 5.23: Geological Map of Shoplots Development

The northern and eastern parts of the site are surrounded by the New Klang Valley Expressway (NKVE). There is an ongoing development at the southern and western part. The highest area, RL 104m is at the western part and the lowest area, RL 54m is at the northern part of the site. Cut slopes and earth fills are observed at several areas at the site. (SC, 2005)

17 boreholes were sunk within the boundary of the site. The final depths of the boreholes range from 16.5m to 50m. In-situ Standard Penetration Test (SPT) was carried out in every boreholes at 1.5m intervals. Disturbed and undisturbed soil samples were collected from various depths for laboratory testing. The tests

conducted were moisture content test, Atterberg Limits Test, Particle Size Distribution, soil chemical tests and undrained Triaxial test.

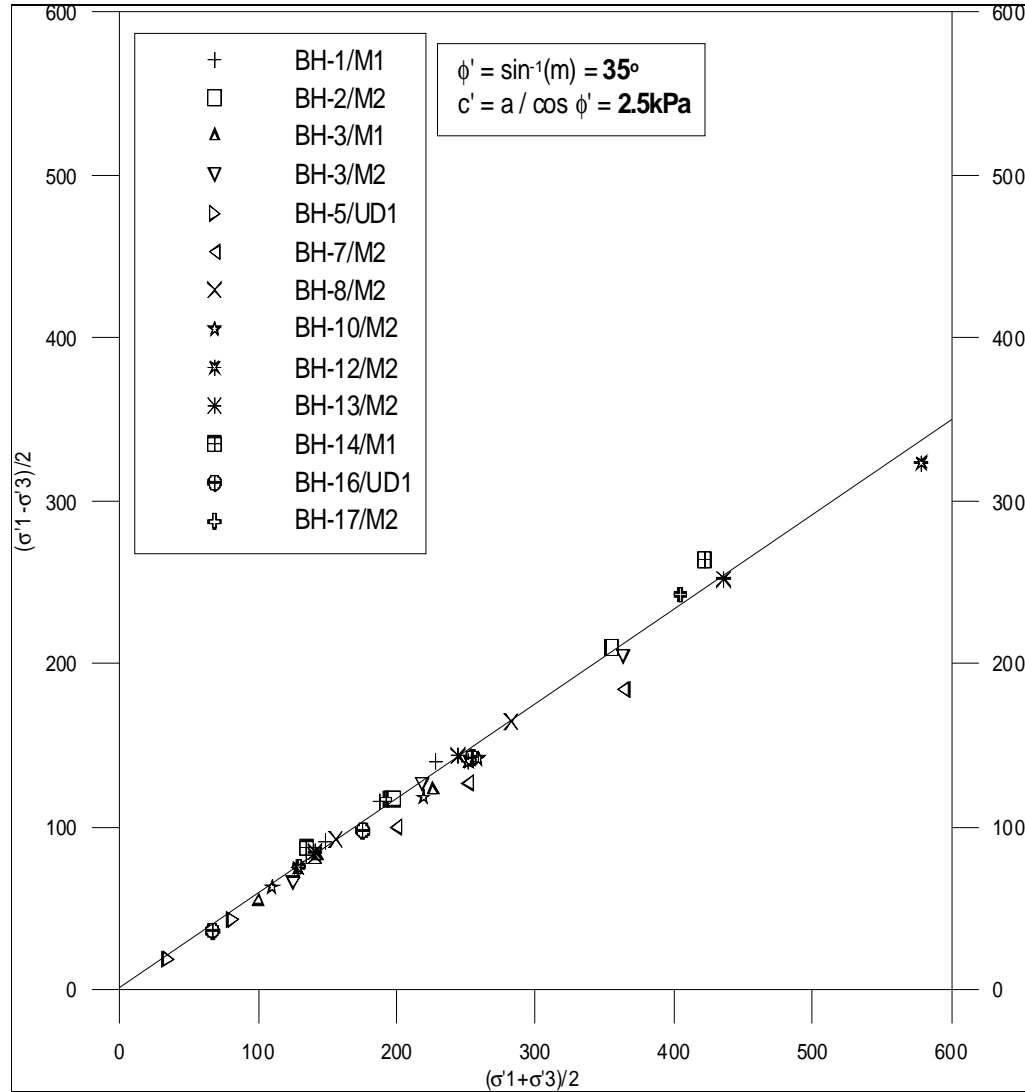
The groundwater was measured from the boreholes and was found to be 1.1m below the existing ground level. The lowest groundwater, 16.8m was at the western side of the site. The Atterberg Limits test result as shown in Figure 5.24 indicates that the soil is of intermediate plasticity, with Liquid Limit and Plastic Limit range from 30% to 80% and 3% to 38% respectively.



(SC, 2005)

Figure 5.24: Atterberg Limit Result

The natural moisture contents of the soil samples range from 17% to 43% meanwhile the density of the soil is from 17.5kN/m³ to 20.5kN/m³. 13 undrained Triaxial tests were carried out to determine the shear strength of the subsoil. The shear strength found parameter found are $c' = 2.5 \text{ kPa}$ and $\Phi' = 35^\circ$. (SC, 2005) The test result is shown in Figure 5.25



(SC, 2005)

Figure 5.25: Undrained Triaxial Test Result

Soil chemical tests were carried out to determine the amount of aggressive chemicals in the soil that will react with the construction materials such as concrete and steel. Based on the chemical result shown in Table 5.4, the soil samples are slightly acidic with pH values ranging from 5.4 to 6.8. The organic content in the soil is negligible, that is less than 0.01%. The chloride content found was also small which is less than 0.01%. The total sulphate content in the soil is between 0.01% and 0.06%, which is also insignificant. Therefore, the soil can be classified as non-aggressive soil based on BS8110:1985.

Depth (m)	Organic Matter Content (%)	Sulphate Content (%)	Chloride Content (%)	pH Value
3.0	0.1	-	-	5.4
4.5	-	0.01	0.01	-
6.0	0.1	-	-	6.1
7.5	-	0.06	0.01	5.5
9.0	0.1	-	0.01	-
10.5	-	-	0.01	-
11.0	-	-	-	6.3
12.0	-	0.03	-	6.8
13.5	-	-	0.01	-
15.0	-	0.02	-	-
16.5	-	-	0.01	-
17.5	-	0.01	-	6
18.0	-	0.03	-	-
21.0	-	-	0.01	-

(SC, 2005)

Table 5.4: Chemical Test Results

Intermediate hard layers were detected varying from RL 55m to RL 73m with vertical thickness of 1.5m to 9m. (SC, 2005) The presence of intermediate hard layer may pose difficulties for driven piles as the piles may not be able to penetrate through the intermediate layer. This will produce a false set and may cause the piles to settle when the building loads are transferred to the softer soil beneath the intermediate hard layers. Therefore, additional investigation is needed to determine the extent of the intermediate hard layers so that proper pre-boring can be adopted to ensure the performance and safety of the building.

The selection for the foundation system depends on many factors such as soil condition, amount of column loads, site topography, amount of building settlement and other. The shoplots can be considered as medium rise buildings. Due to the large amount of fill and the presence of stiff subsoil bearing stratum, driven pre-stressed spun piles are chosen as the building foundation system. The pre-stressed spun pile has better quality of control and higher lateral resistant than the reinforced concrete

pile. The diameters of the spun piles used were 300mm and 450mm, depending on the amount of column loads. (SC, 2005) The proposed system will transfer the column loads to the foundation by column-pilecap-pile system.

The rows of the shop lots were divided into separate building to prevent differential settlement due to significant differences in pile lengths used in the foundation system. Since the piles will be used in filled ground, negative skin friction of the pile need to be considered during the pile design. Negative skin friction occurs when the subsoil subjected by fill settle with time more than the down drag forces caused by the pile which acts onto the pile. This will increase the amount of load needed to be carry by the piles.

Negative skin friction will affects the carrying capacity of the piles and also the performance of the piles. The amount of load from the building and negative skin friction may exceed the designed pile capacity. This will cause large settlement and damages to the buildings. One of the methods to solve this problem is to downgrade the pile capacity or to preload the ground to eliminate future subsoil settlement under the fills. The effects of negative skin friction can also be reduced by applying a slip coating, which involves an application of bituminous layer on the pile surface forming a slip layer between the pile surface and the settling soil.

As presented earlier, the subsoil consists of SILT and SAND. When the subsoil is subjected to fills, majority of the settlements are immediate, therefore the effect of negative skin friction is small. However, it is not practical to eliminate the risks due to negative skin friction because of the high earth fills that can reach a maximum of 25m. The capacity of the spun piles was downgraded to about 30% of the original spun pile working capacity to accommodate for the negative skin friction. (SC, 2005)

CHAPTER 6

COMPARISON OF CASE STUDIES

Generally, all the case studies are about the development and the construction of medium to high rise buildings for commercial or residential uses. A comparison between the five case studies will be made in this chapter. Comparison will be made in several aspects such as the variation of soil at the site locations, the type of foundation systems used, the factors governing the foundation selection and also some of the major issues involved in those case studies. The different development and usage of the buildings is shown in Table 6.1

Case	Project	Usage
1	29-storey condominium and 2-storey service apartments	Residential
2	17- storey apartments	Residential
3	31-storey condominium	Residential
4	Two 35-storey serviced suites	Residential
5	5-storey shop lots	Commercial

Table 6.1: The Different Type of Case Studies

6.1 Site Conditions

Although all the sites in the case studies are located in Kuala Lumpur but they are underlain by different type of soil formation such as Kuala Lumpur Limestone, Kenny Hill Formation and Granite Formation. Two of the cases are underlain by granite formation, one by limestone formation, one by Kenny Hill Formation and the last case, the combination of granite and Kenny Hill Formation. These formations will affect the selection of foundation system at that particular site. For example, at the limestone area, false set of piles may occur due to the presence of cavity problem at that area. Therefore, certain consideration and precaution are needed in the design and construction of foundation system at this area.

The site conditions vary from one case to one case because no ground conditions are similar in the nature. There is a sloping ground at the western part of the site in the first case and the difference of elevation is 31m. The ground condition in case 2 is generally flat and is underlain by mixture of soil and construction debris. In case 3, there is a 15 to 45m thick overburden of granitic rocks at that site. The site in case 4 is generally flat ground and the site in case 5 is located on an undulating terrain with 50m different in elevation. This are requires a maximum 25m of cut and fill of earthworks.

6.2 Soil Testing

The soil testing carried out in Kuala Lumpur are basically the same in every cases. The soil tests conducted to find the engineering properties of the soil are Particle Size Distribution, moisture content, Atterberg Limit, triaxial test and Standard Penetration Test (SPT). Chemical test were conducted to find the pH values and the degree of aggressiveness of the soil. Disturbed and undisturbed soil samples are collected at various depths for this purpose. Several of the soil borehole logs can be seen in the Appendix.

6.3 Foundation System

Different foundation systems are used in the different cases because of the variation in ground or soil and the site surrounding conditions. The foundation system used in the case studies are shown in Table 6.2

Case	Type of Foundation	Foundation System
1	Deep	500mm jack-in pre-stressed spun piles 300mm micropiles
2	Shallow Deep	2m deep pad footing 800,1050,1200 and 1500mm bored piles 300mm micropiles
3	Deep	300,350,400 and 500 mm jack-in pre-stressed spun piles
4	Deep	900,1500,1800,2500 and 3000mm bored piles
5	Deep	300 and 450mm driven pre-stressed spun piles

Table 6.2: Foundation System for Case Studies

6.4 Foundation Selection

As been discussed in Chapter 3, the selection of a suitable foundation system for a certain project is governed by a lot of factors. The two main factors identified are site condition and site surrounding area. In case 1, the site is located near a school and residential areas, therefore jack-in piles are used. The piles installed using this system will produce minimum noise and vibration impact during pile installation. Pre-stressed spun piles of 500mm in diameter used in this case have a higher lateral resistant and better quality control as lesser piles are broken during the pile installation. Micropiles of 300mm diameter are recommended for the construction of the car park block in case1. These micropiles act as additional support to the existing foundation system. Micropiles are chosen because they are cheaper and easy to construct.

For case 2, pad footing of 2m deep is used at area with shallow bedrock because the bedrock has sufficient strength to carry the building loads. Meanwhile, at the area with deeper bedrock, deep footings such as bored piles are recommended. At areas where cavity problem occurred, 300mm micropiles are used. The micropiles cost lesser compared to the treatment of cavity area with grouting.

Jack-in pre-stressed spun piles with diameter range from 300mm to 500mm are recommended in case 3 because of the large amount of loads to be supported by the foundation system. Since the site is underlain by hard layer, that is limestone formation, 2.2 times the working axial pile capacity jack-in load is recommended to install the piles.

The foundation system used in case 4 is influenced by the amount of loads supported by the building. Larger diameter bored piles are used to support the loads from the columns, walls and the lift cores of the building.

Since shoplots are considered as medium rise building, deep foundation is chosen. Driven pre-stressed spun piles are recommended in case 5 because there is a large amount of fill at the site.

6.5 Main Issues

The main issue involved in case 1 is the presence of rock boulder in the western part of the site area. This rock boulder will affect the construction of the foundation system. Since this rock boulder is located next to a school, it cannot be blast. Blasting process will produce a loud noise and vibration impact that will disturbs the students studying at that school. Besides that, it will also disturb the residents living nearby the site. Furthermore, the cost of blasting a rock is very expensive. Therefore, in this case, it is not recommended to blast the rock boulder.

The site for development in case 2 is located on a limestone area. As been discussed in Chapter 5, cavity may occur in limestone area. The roof of cavity may collapse

and hence affecting the foundation system built on this area. This problem may be treated by grouting but this solution is not economical. Therefore, micropiles are adopted at area where the cavity exists. However, if cavity is still detected during the installation of micropiles, compaction grouting is recommended.

The presence of intermediate hard layers in case 3 makes the pile installation more difficult. The piles may be falsely terminated on top of this layer. If softer layer exist below the hard layer, then the building may settle because the loads are transferred to the softer layer instead of the actual founding level. Therefore, pre-boring is required to determine the extent of the hard layer in this site.

The site in case 4 is located in the centre of Kuala Lumpur city and is surrounded by hotels. Bored piles are recommended instead of using jack-in piles because of the large diameter of piles to be used. The biggest diameter piles used for this project are 3000mm. Since this site is located near to other high rise buildings, further testing such as Sonic Logging Test is required to ensure the safety of the building constructed.

Intermediate hard layers also present in case 5. Again, this hard layer will affect the setting of the piles. Besides that, since this site requires a lot of fill, a maximum fill of 25m, negative skin friction should be considered during the pile design. Negative skin friction occurs when the filled soil settles with piles movement. This problem can be solved by either preloading the ground to eliminate future soil settlement or by downgrading the pile capacity. When pile capacity is reduced, larger piles will be needed. Besides that, the effect of negative skin friction can be reduced by applying a slip coating such as bituminous coating on the piles. A slip layer will then be formed between the pile and the surrounding soil.

CHAPTER 7

CONCLUSIONS

This study aims to provide the readers with some views on the current foundation practices used in Kuala Lumpur, Malaysia. It also aims to investigate what are the foundation system and the factors governing the foundation system at the selected case studies.

This research project had successfully reached its objectives through the literature reviews and the case studies conducted. The conclusions that can be made from this project are follows:

- The soils in Kuala Lumpur area are derived from the two main sources that are the Kenny Hill Formation and Limestone Formation. Only a small area of the city is covered by granite formation.
- The engineering properties of soil at the site will influence the design and construction of foundation system. Therefore, an investigation of the soil

properties is essential and it can be carried out using different types of soil testing available nowadays. The soil testing available in Malaysia are triaxial test, particle size distribution, moisture content, Atterberg Limit test, soil chemical test and others.

- Since the buildings built nowadays are either medium or high rise, deep foundation such as piles is recommended. Piles are used to transfer the loads to a greater depth for a better support. The piles used in the case studies are bored piles, micropiles and pre-stressed spun piles.
- The selection of a safe and cost effective foundation system for a particular building at particular site is governed by a lot of factors. These factors are all important and shall not be omitted just to make the design and construction of foundation faster.

Further Research Work

A further comprehensive study can be made to make this research project more valuable. Several works can be made in the future such as:

- Focusing in a one type of soil condition and determine what is the foundation system that best suits the soil condition.
- Establish the relationship between the foundation system and the site soil condition.
- Focusing in other states in Malaysia.

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APPENDIX A
PROJECT SPECIFICATION

University of Southern Queensland
Faculty of Engineering and Surveying

ENG 4111/2 Research Project
PROJECT SPECIFICATION

FOR: LEOW Jia Hui
TOPIC: Investigation of Soil Foundation in Malaysia.
SUPERVISOR: Dr. Jim Shiau, Faculty of Engineering and Surveying.
SPONSORSHIP: None

PROJECT AIM: This project aims to investigate the types of soil encountered in the design and construction of building foundation in Malaysia. Their suitability for using as the foundation soil will be examined.

PROGRAMME: Issue A, 17th March 2005

Background/Review

1. Comprehensive study on the types and the distribution of foundation soil in the state of Selangor, Malaysia.
2. Investigate the material properties of the soil through local soil testing.
3. Understand the foundation design principle, approach and challenge faced in the local consulting firm for difficult soil.

Five Case Studies

4. Identify the soil-relating problems at each construction site.
5. Investigate the methods used to improve soil strength at chosen sites.
6. A comparison of soil conditions and solutions for foundation design and construction among cases

Suggestions and Conclusions.

AGREED : Student : _____ Supervisor: _____

LEOW Jia Hui

Dr. Jim Shiau

Date : _____

University of Southern Queensland
Faculty of Engineering and Surveying

ENG 4111/2 Research Project
PROJECT SPECIFICATION

FOR: LEOW Jia Hui
TOPIC: A Study Of Building Foundations In Malaysia.
SUPERVISOR: Dr. Jim Shiau, Faculty of Engineering and Surveying.
SPONSORSHIP: None

PROJECT AIM: This research project main aim is to review the current building foundation practices in Kuala Lumpur, Malaysia.

PROGRAMME: Issue B, 10th June 2005

Background/Review

1. Determine the types and distribution of soil available in the state of Kuala Lumpur, Malaysia.
2. Investigate the material properties of the soil through local soil testing.
3. Understand the foundation design principles, approach and challenge faced by the local consulting firm in building foundation.

Five case studies,

4. Identify the problems at each construction site.
5. Investigate the type of foundation used at that site and the reasons for that selection.
6. A comparison of the soil conditions, and foundation system among the cases.

Suggestions and Conclusions

AGREED : _____

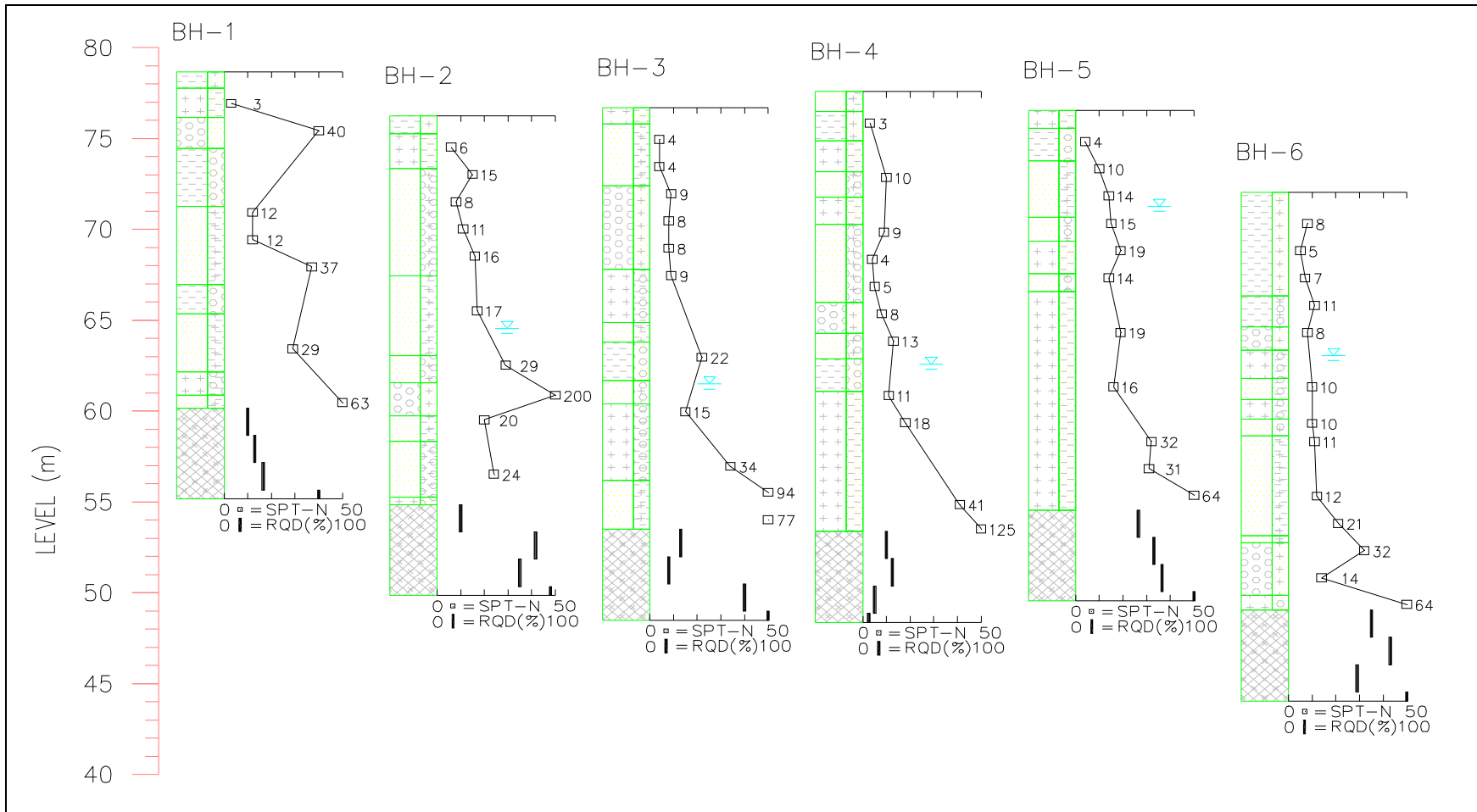
LEOW Jia Hui

Dr. Jim Shiau

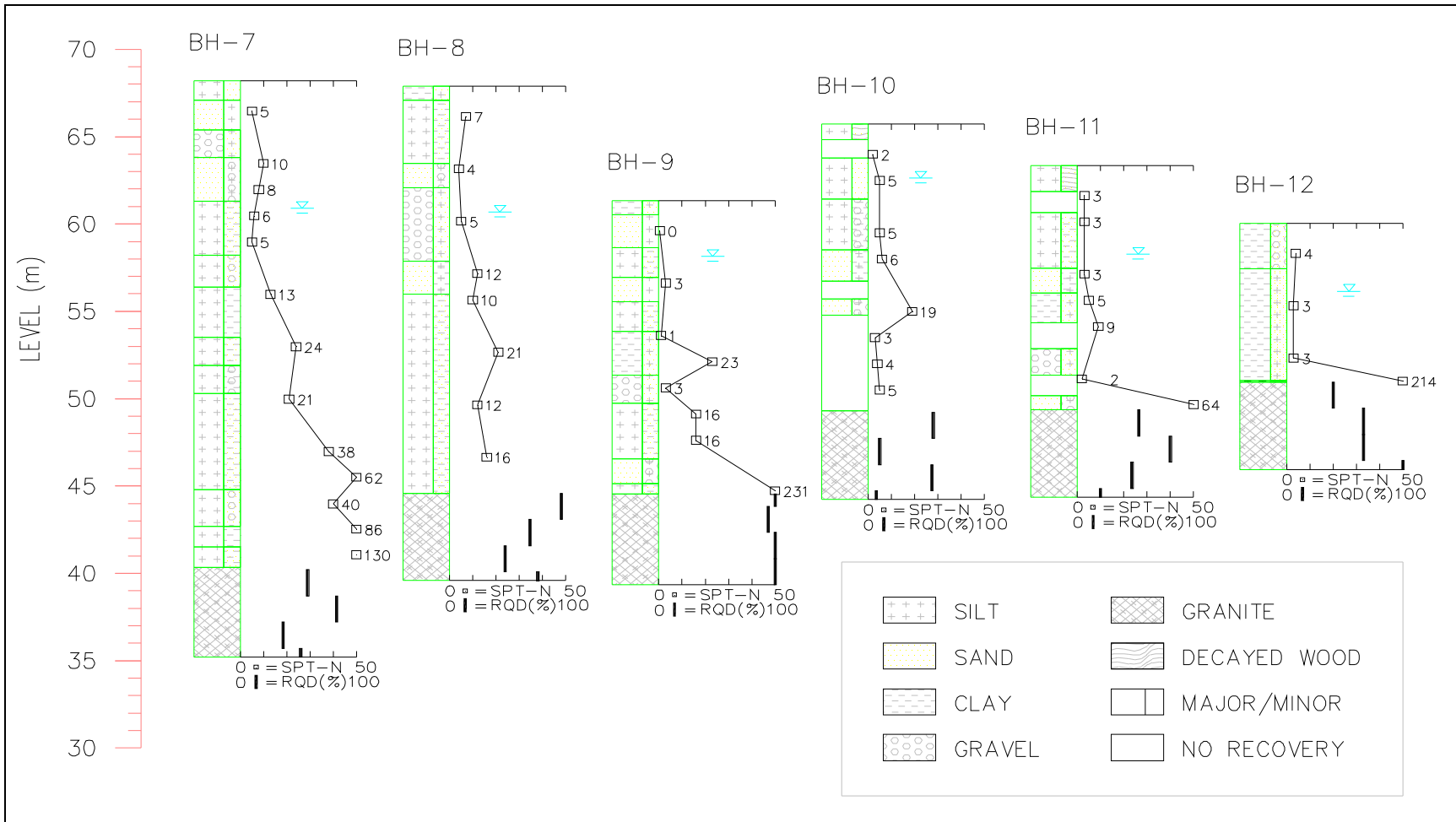
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APPENDIX B

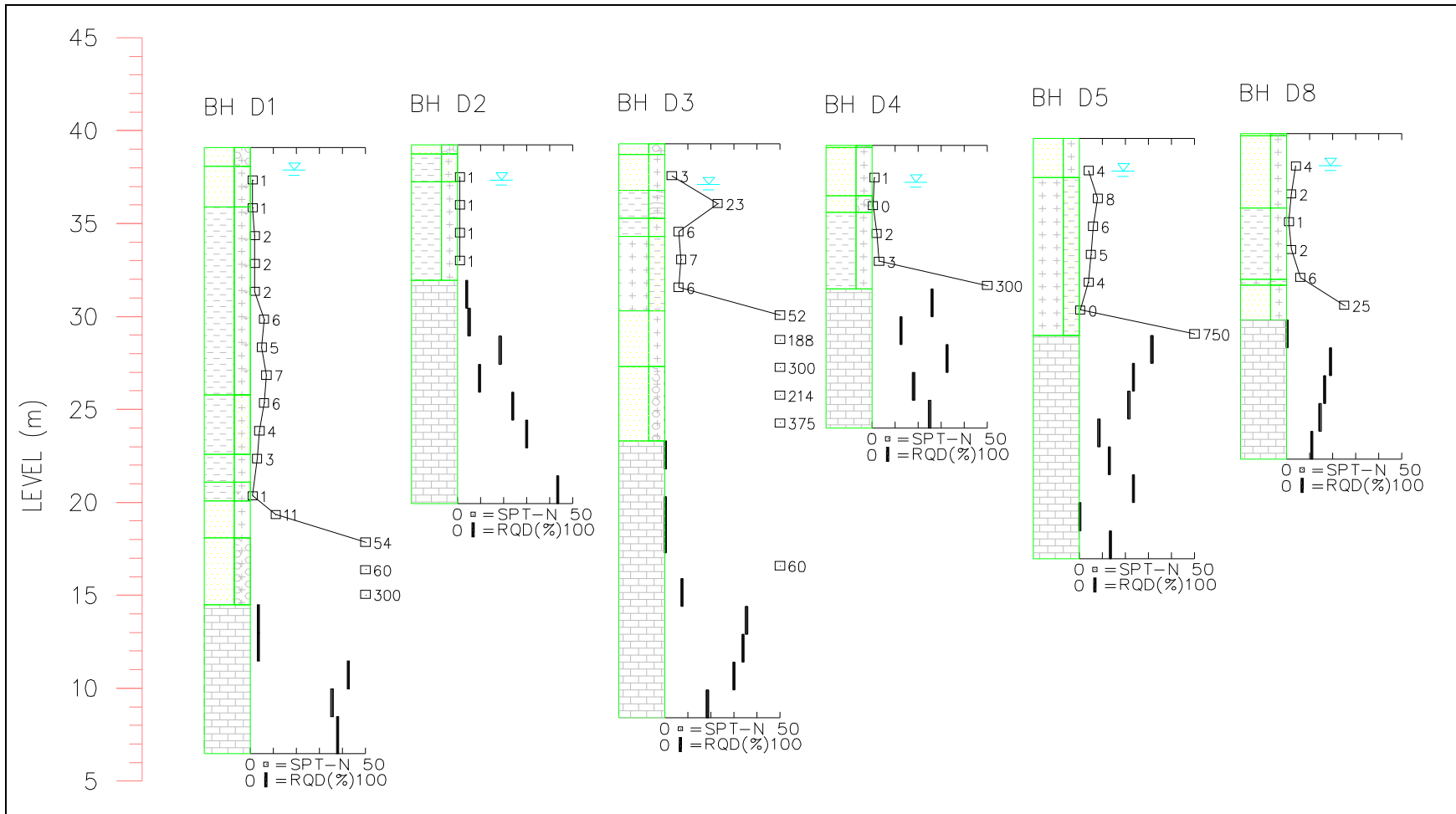
CASE STUDY BOREHOLES DATA



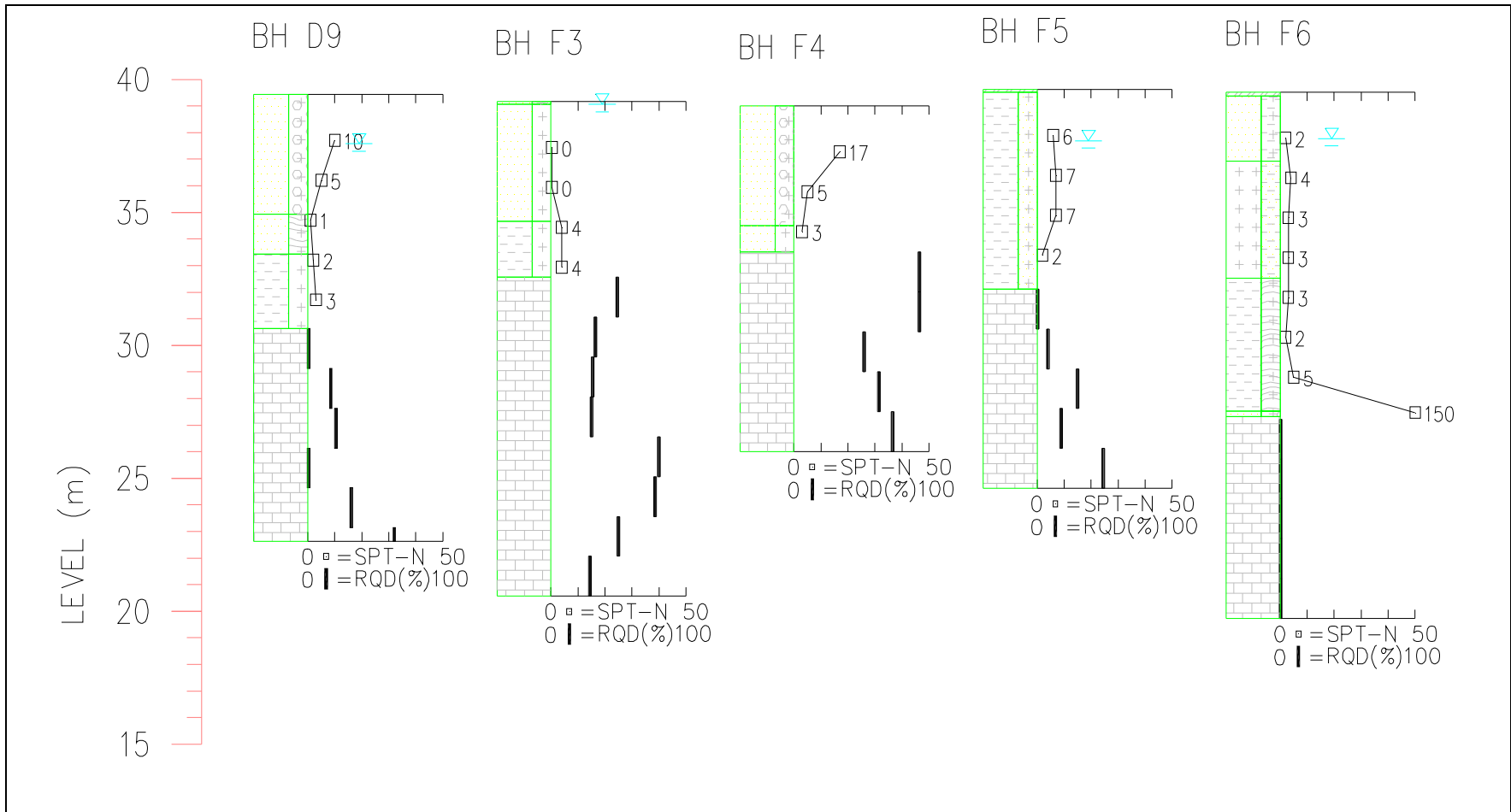
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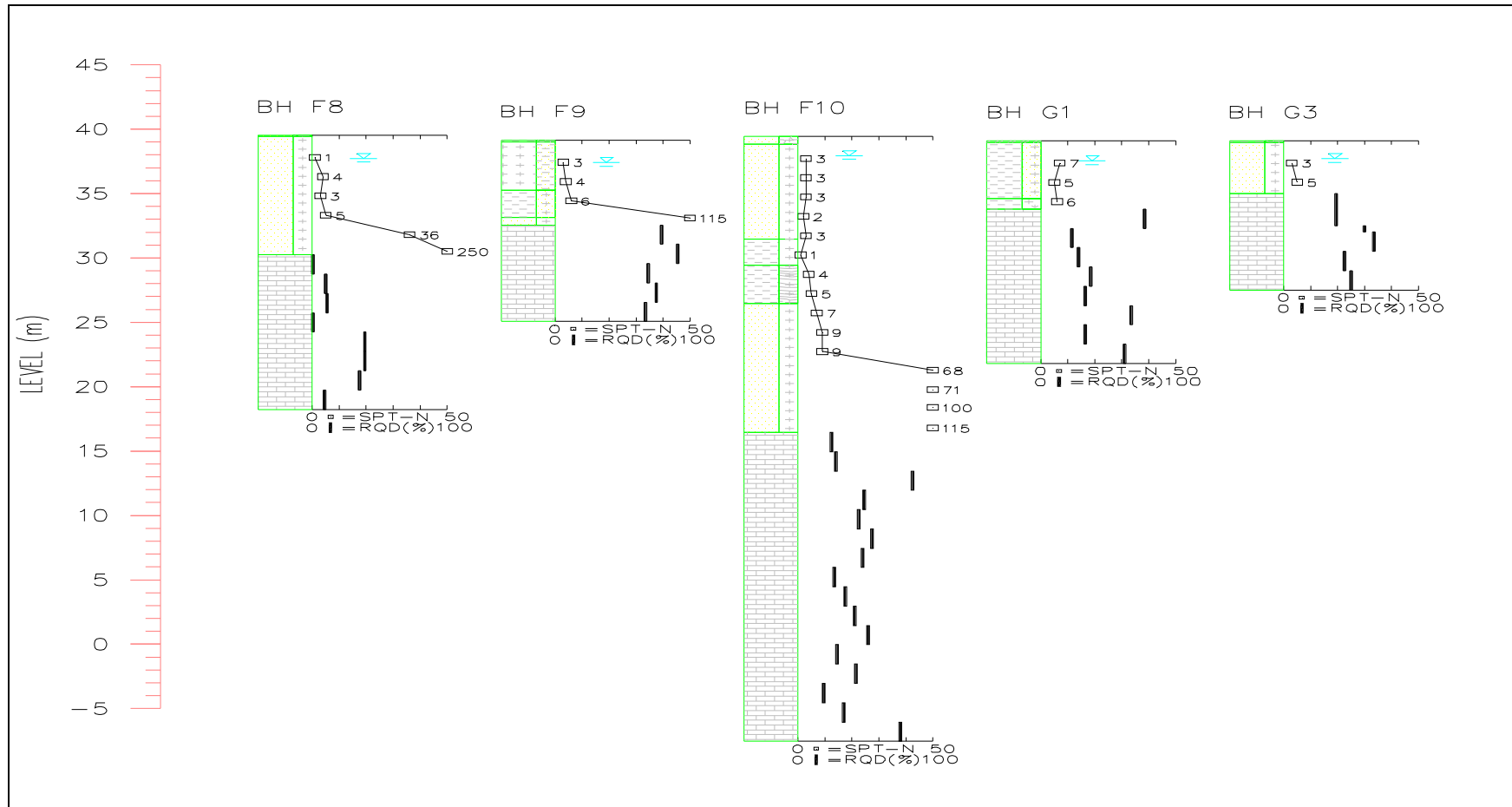
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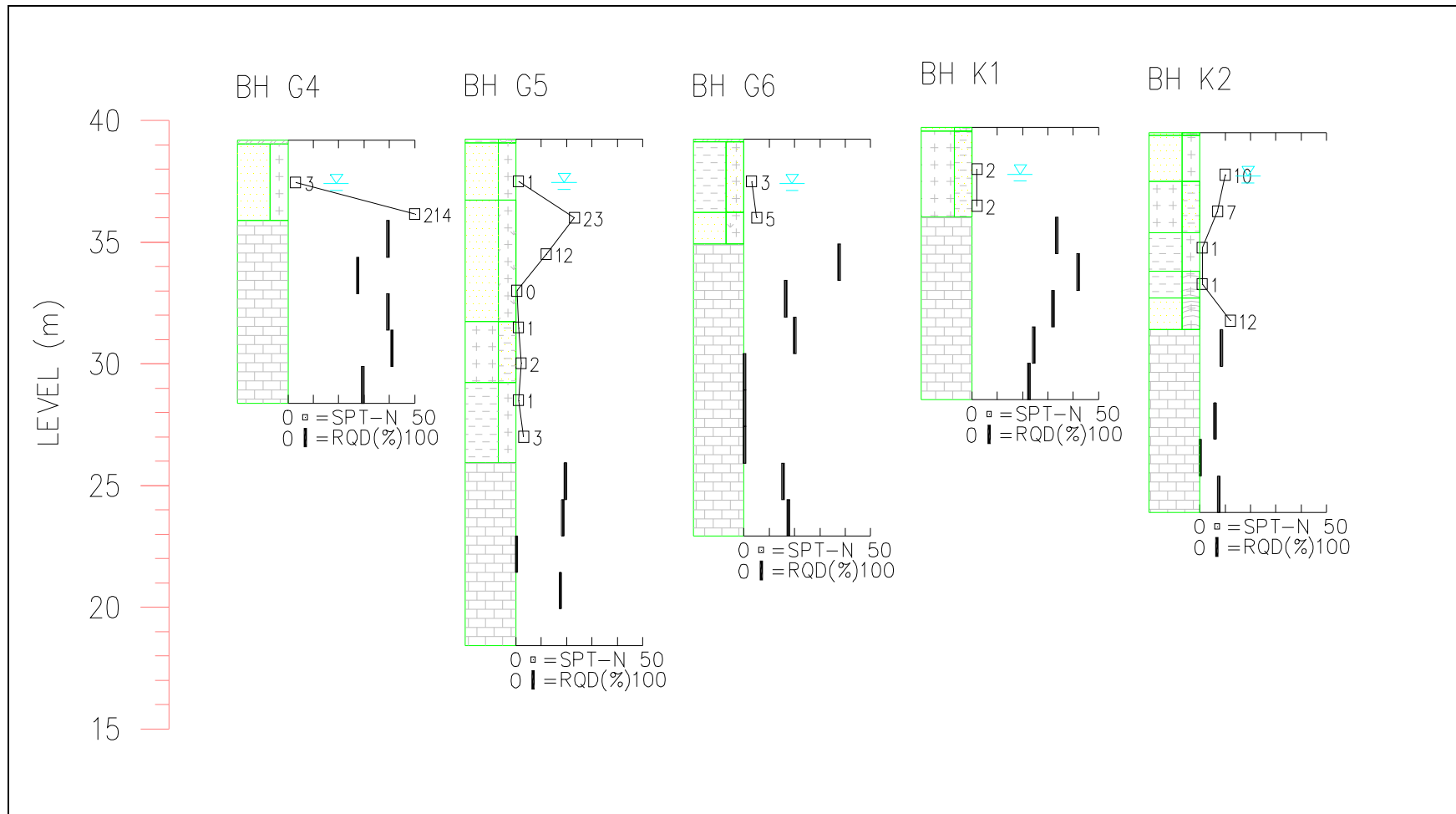
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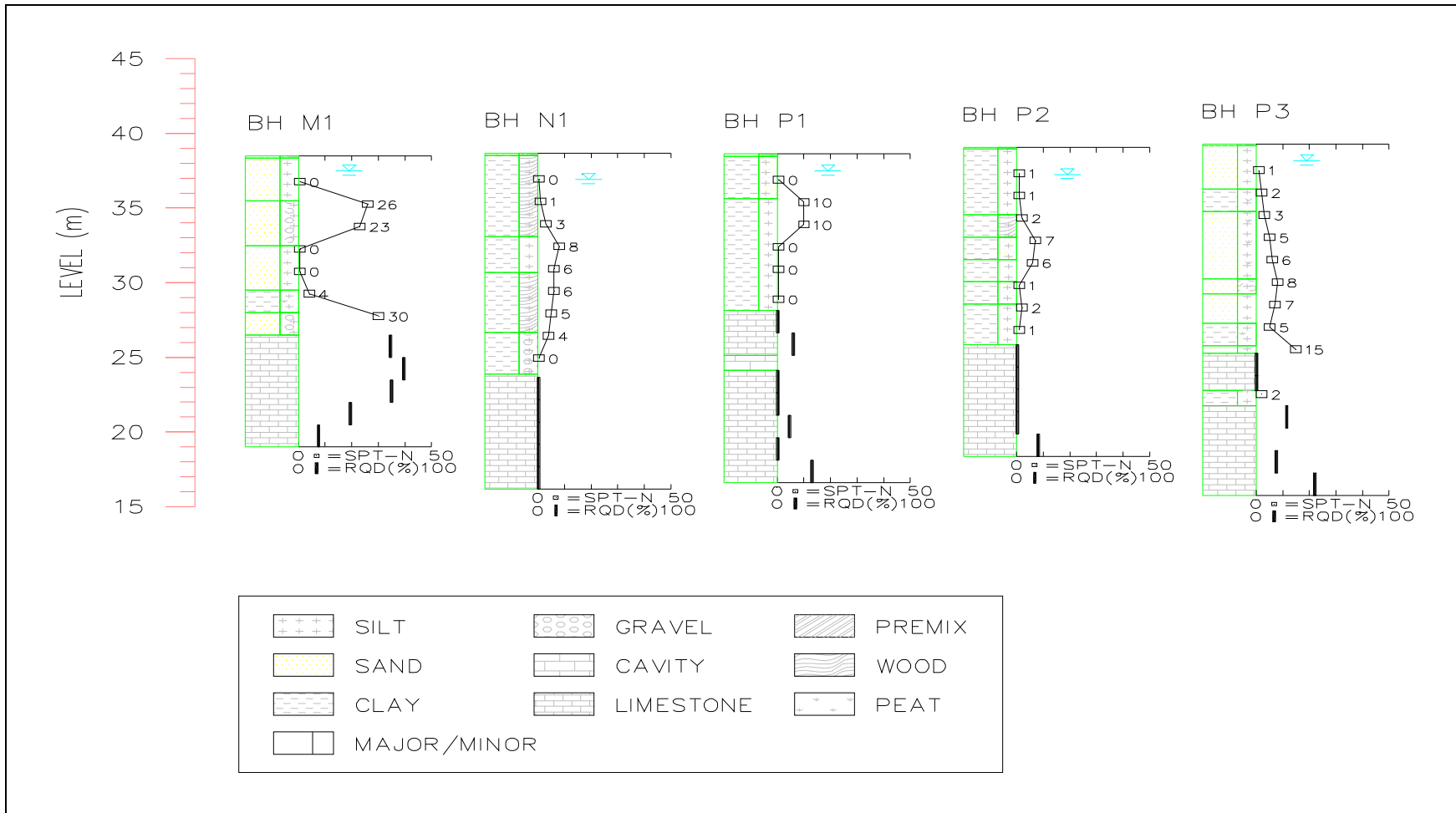
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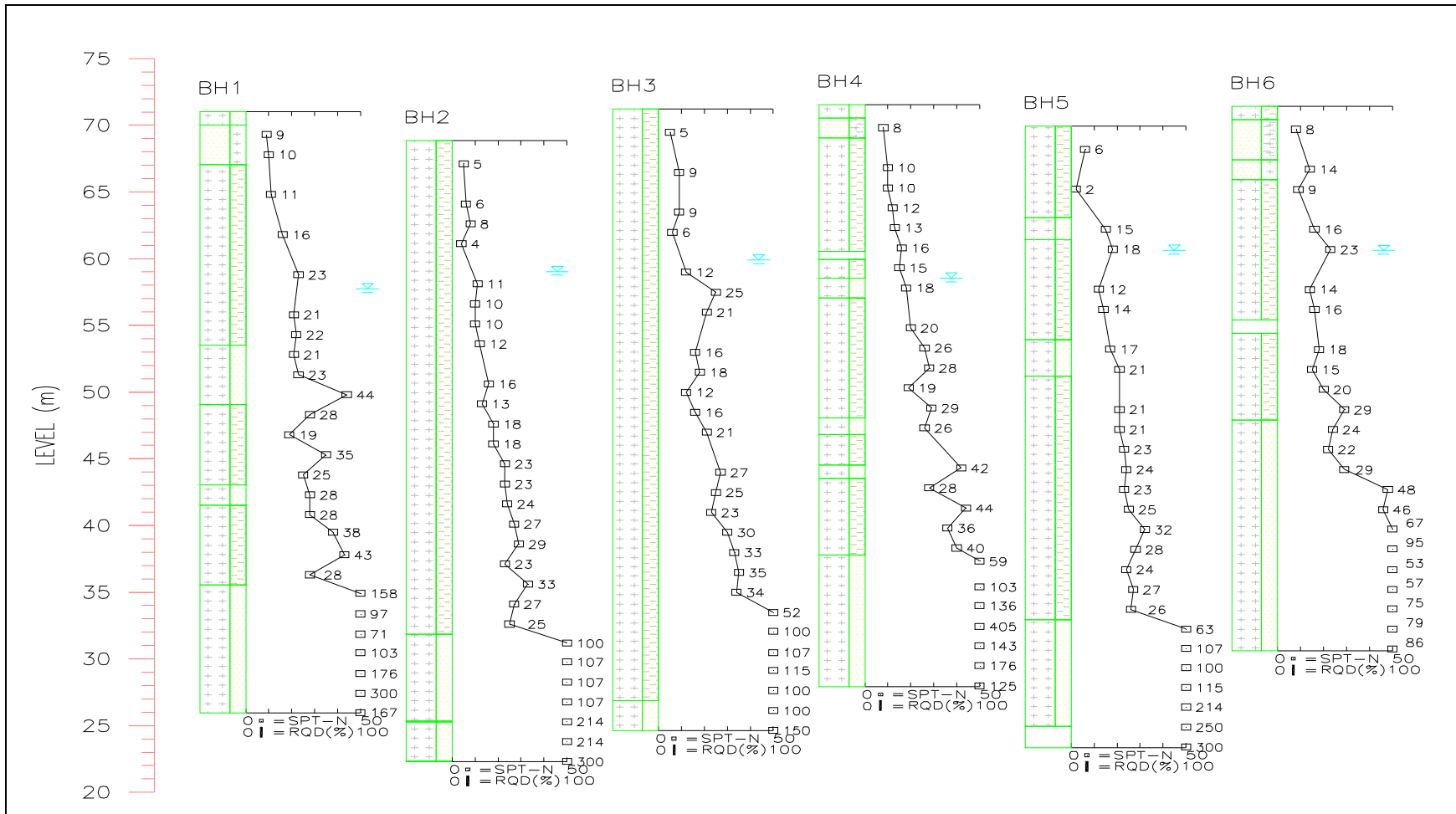
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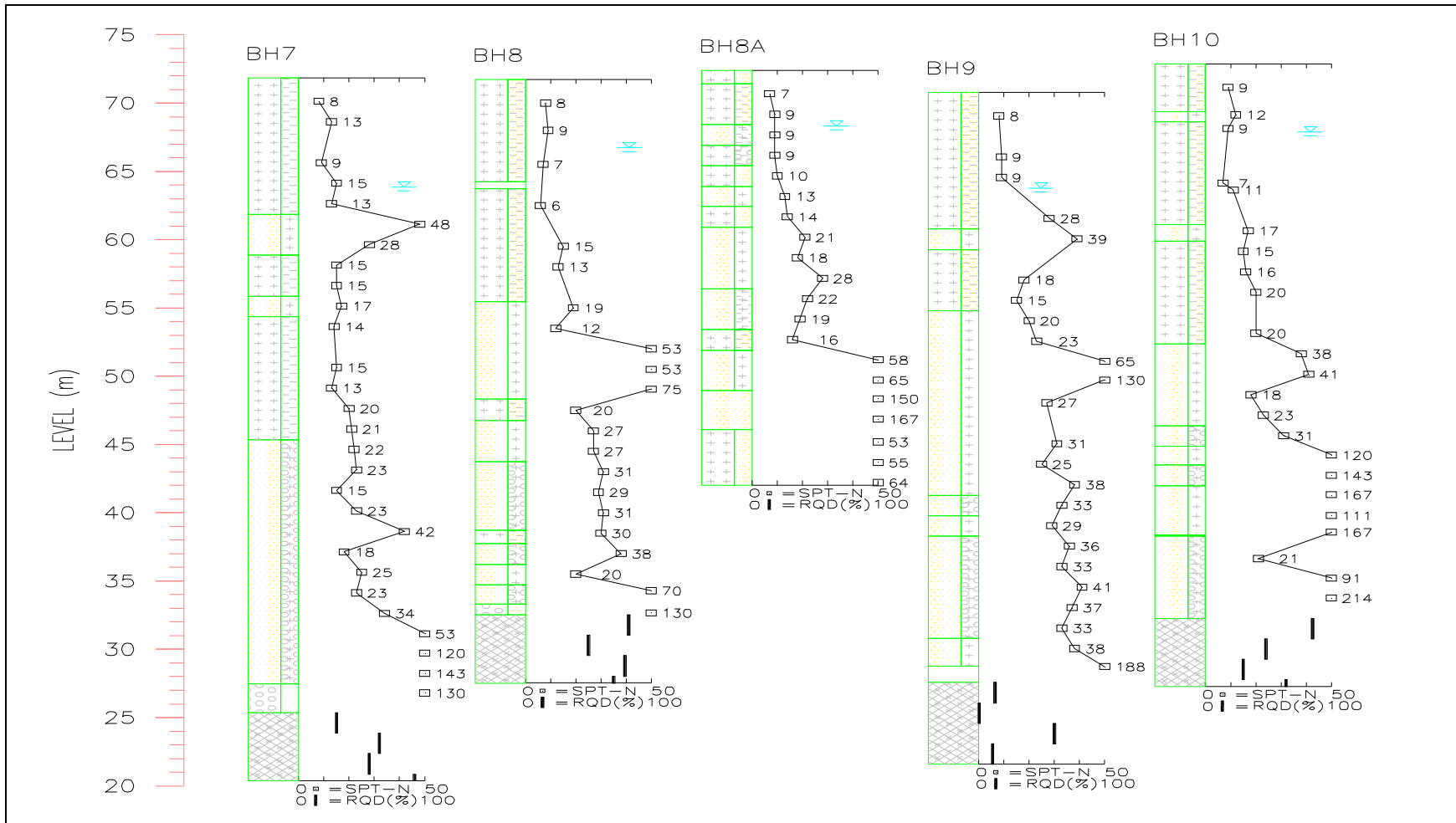
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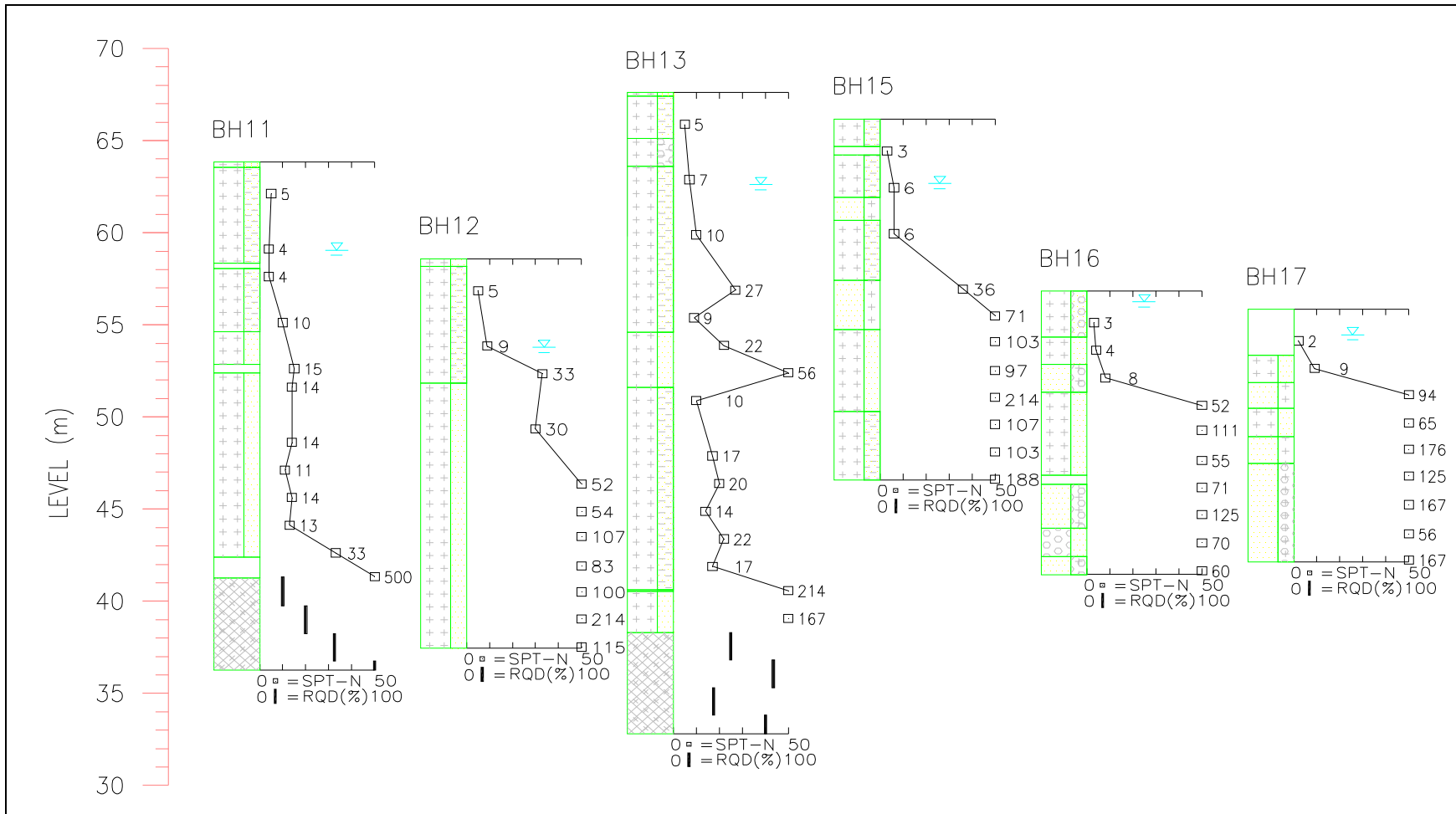
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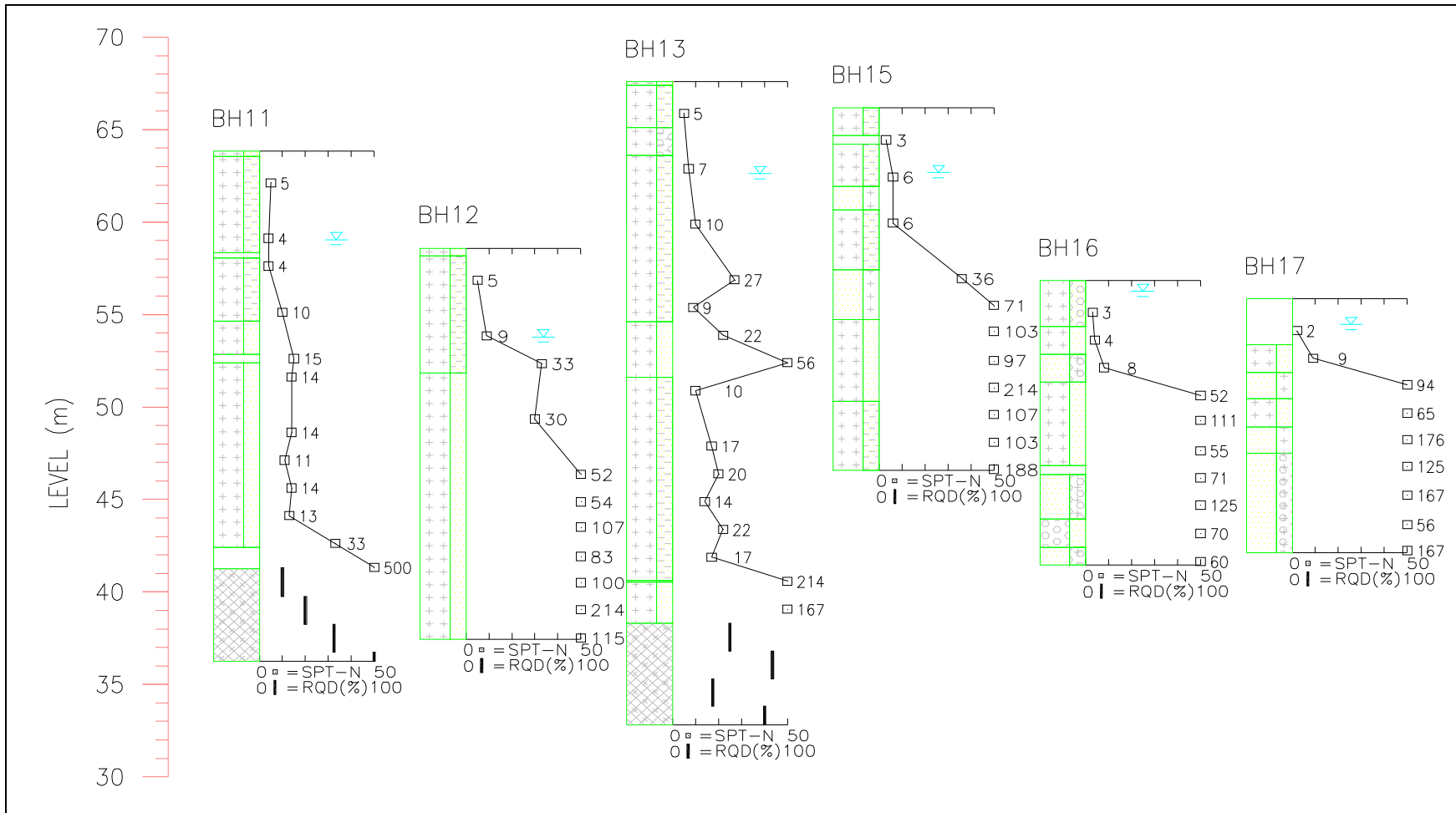
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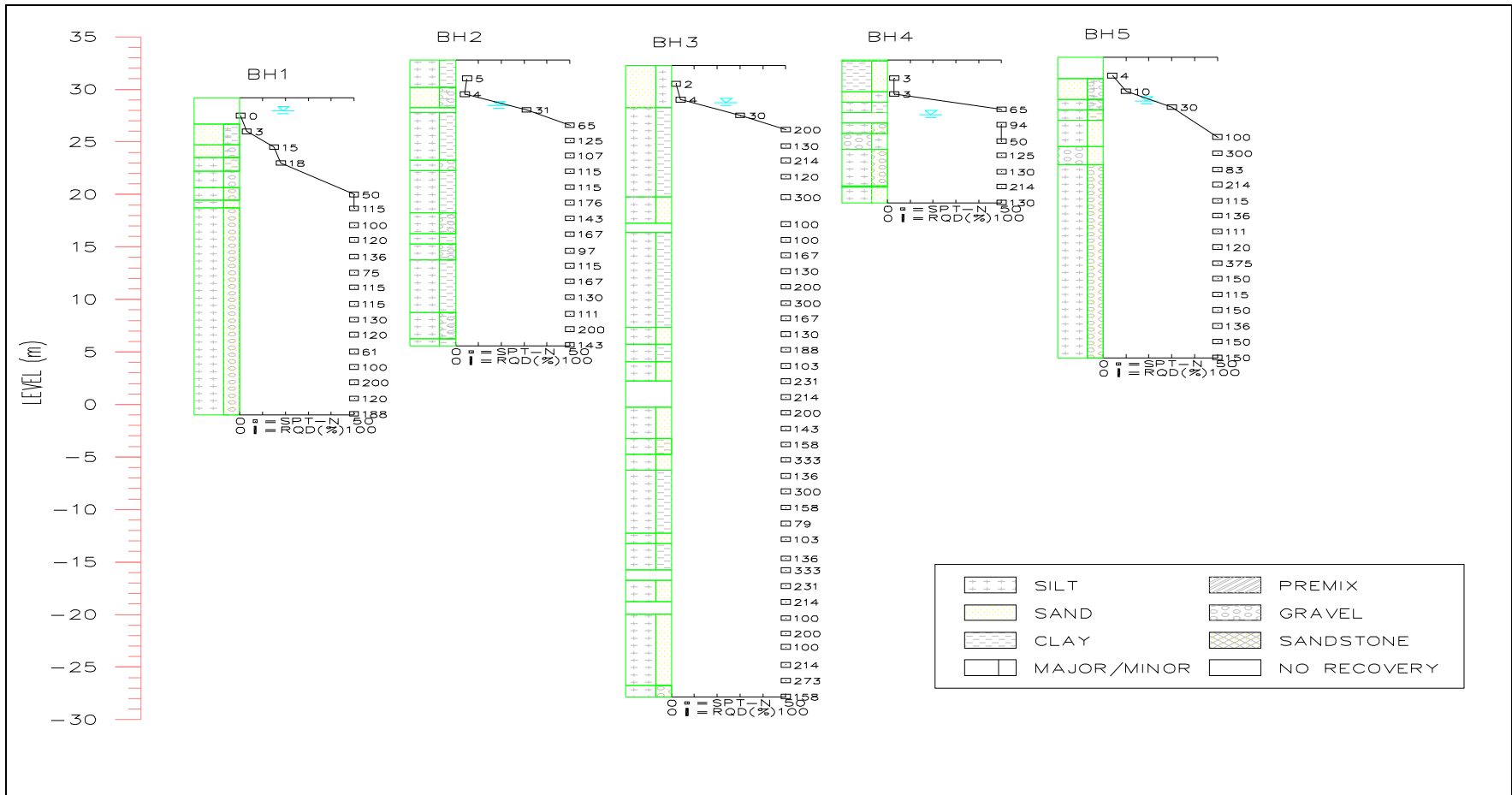
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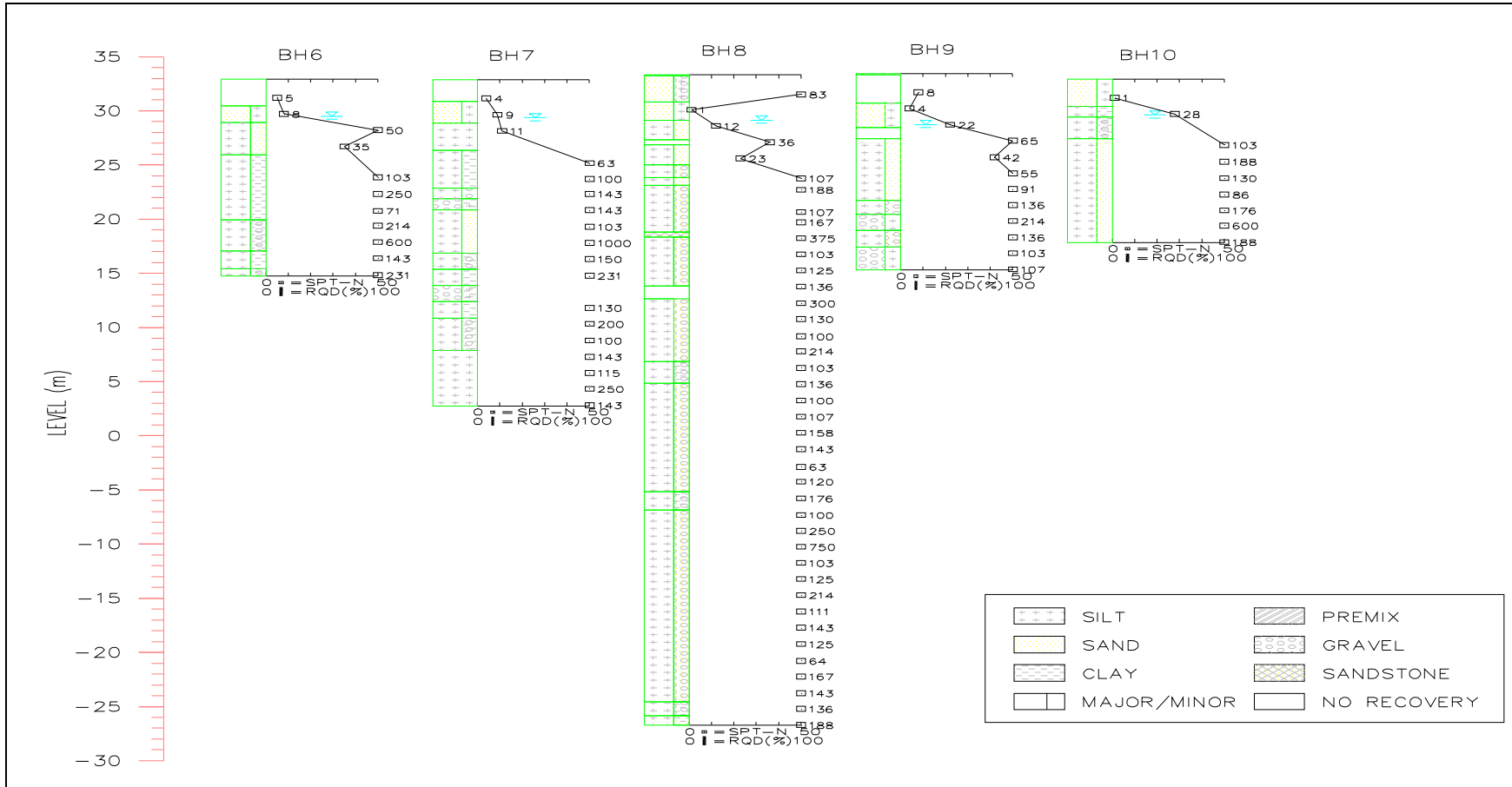
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Borehole Data for Case Study 3- Sheet 4



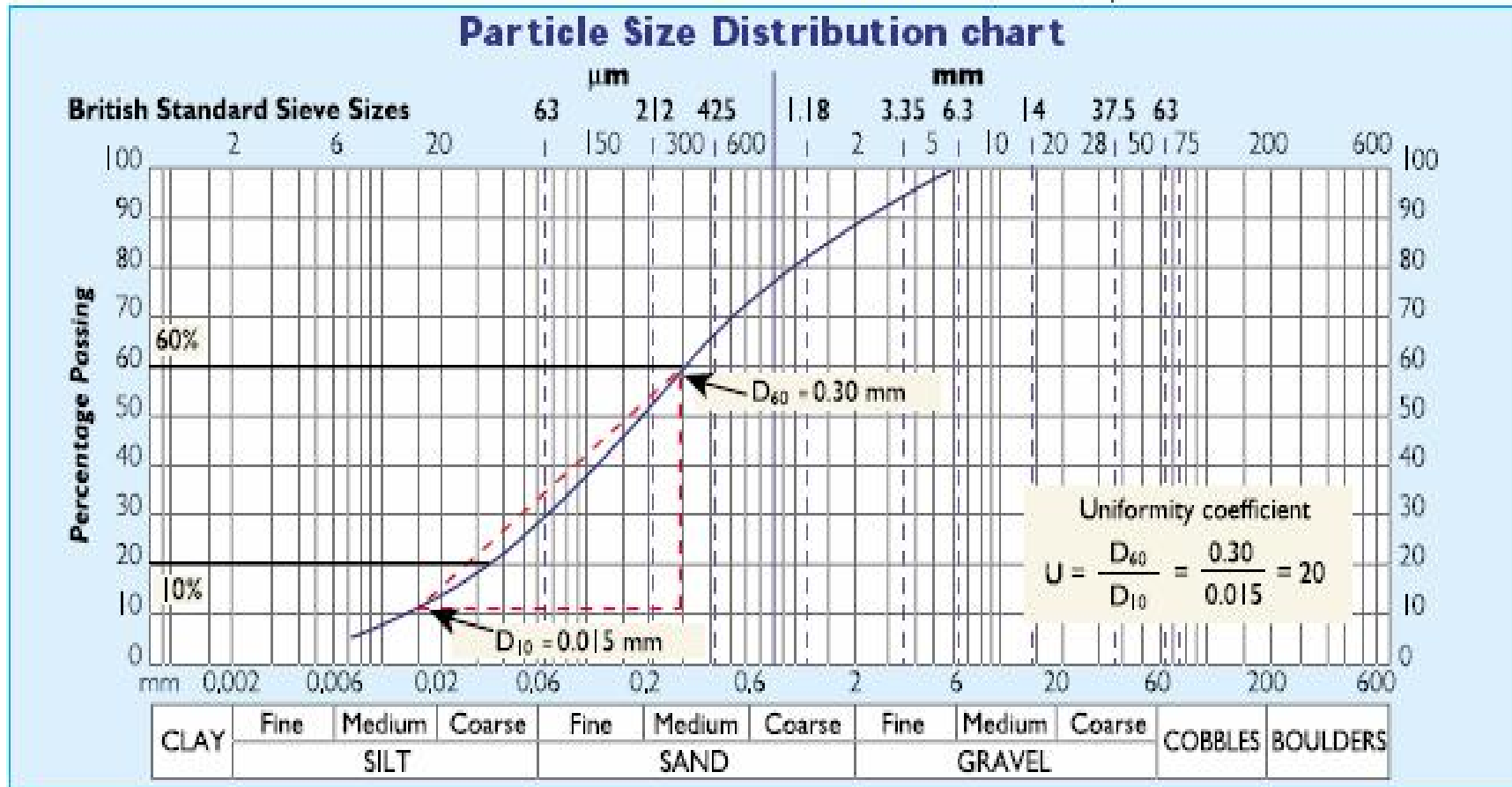
Borehole Data for Case Study 4 – Sheet 1



Borehole Data for Case Study 4 – Sheet 2

APPENDIX C

PARTICLE SIZE DISTRIBUTION



(Source: ELE International)

Particle Size Distribution Chart