Problematic Soils

In Search for Solution

PROFESSOR DR. BUJANG BIN KIM HUAT

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ABSTRACT

Soils are considered to include all naturally occurring loose or soft deposits overlying the solid bedrock. They are formed from the disintegration and decomposition of rocks and also by decomposition of organic materials. Compared with rocks, soils are softer in terms of strength and more compressible, thus giving more problems to engineering works as compared with rocks. However are soils really a problem, or is it the theory? Perhaps the latter is more valid.

The soils found in Malaysia can generally be termed as tropical soils. They range from residual soils which are normally soils formed by in-situ weathering of parent rocks to soft clays and organic soils, including peat. Soft soils generally have low shear strength and high compressibility. As such, problems like large settlements, flooding and damage to infrastructure due to soil subsidence, foundation failure due to lateral movement of soil, etc, are common. Residual soils on the other hand are generally considered to be better in terms of engineering properties, compared with soft soils, but by virtue of their existence on sloping ground, landslides become a major concern. Over 400 landslides have been reported in the country over the last two decades (more than 30 were major landslides), involving both cut and natural slopes with a total loss of more than 200 lives and billions of Ringgit in damage to property.

In meeting the above mentioned challenges, some research works have been done at University Putra Malaysia. These include proper analysis and design of piles against lateral loading; fundamental understanding on the properties and behavior of soft soils, both soft clays and peat; methods to improve/stabilize the soils; slope assessment system for landslide prediction, bio-engineering techniques to stabilize slopes; and use of waste materials as an economical solution to repair slopes. These works are briefly described in this inaugural lecture booklet.

WHAT IS SOIL?

"Almost all structures are built on soils; those that aren't either float, fly or fall over"

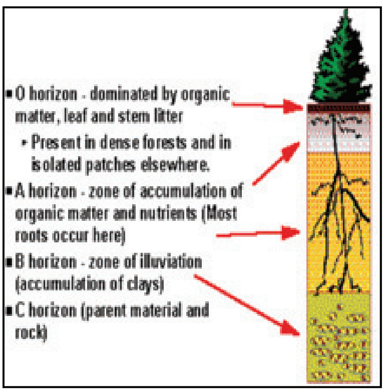
Definition of Soil

Soil is usually considered as the loose agglomerate of minerals, sometimes including organic materials. The term soil is derived from the Latin word '*solium*', meaning the upper layer of the earth crust that may be dug or ploughed. Though it shows a close resemblance in meaning with the definition of soil of agronomists, different professionals define soil differently.

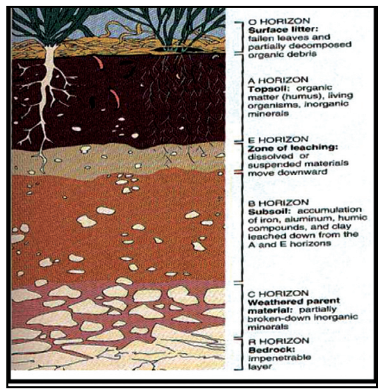
Many professionals, like geologists, agronomists and civil engineers, use soils. Engineers use soil and rock for many purposes; as construction materials, as support to the foundation of structures and also as structural material. According to the civil engineering definition, soil is considered to include all naturally occurring loose or soft deposits overlying the solid bedrock. It is formed from the disintegration and decomposition of rocks, the process that is known as weathering. It may also be formed by decomposition of organic materials.

To a geologist, all the materials mantling the earth's crust are unconsolidated sediments overlying solid bedrocks called regolith. Regolith is roughly equivalent to soil, as used by the engineers. It may include *saprolite*, which is a decomposed rock that is chemically altered and coherent and retaining traces of the original structure of rock.

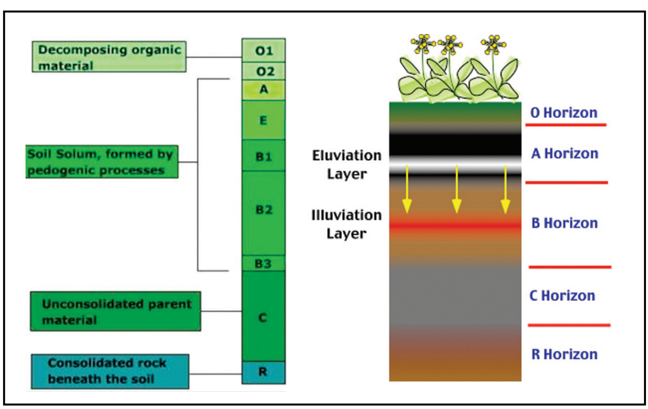
Agronomists, agriculturists and soil scientists are much more concerned with the growth of plants on the earth's surface. According to them, soil is a thin layer of loose surface materials of the earth's crust, which is a portion of regolith or engineering soil, where plants grow. The geotechnical engineers know the material, which is called soil by the agronomists, as topsoil. The topsoil usually contains large quantities of organic materials and is not suitable either as construction material or as a foundation. The topsoil or surface soil is usually removed before the construction of a structure. The nomenclatures used by the different professionals, for soils, are shown in Figure 1.



(a) Agronomist



(b) Geotechnical Engineer



(c) Geologist

Figure 1 Nomenclature used for soil by various professionals

Soil is generally a product resulting from disintegration and/or decomposition of rocks, a process that is known as weathering. Based on their mode of origin, (engineering) soils can be divided into two categories which are - residual soil and transported soil. A third category is organic and peat soil.

How Are Soils Formed?

Compared with rocks, soils are softer in terms of strength and more compressible. As a result soils pose more problems to engineering works, compared with rocks. Further, most construction projects are either built on soil, or out of the soil itself. Hence engineers will normally face more problems from soil engineering, compared to rocks.

Formation of soil can be broadly classified under two categories:

1. Based on the weathering process and
2. Based on the mode of formation.

Based on Weathering Processes

Rocks exposed to the atmosphere will undergo a process of weathering which could be physical, chemical or biological weathering. Weathering is therefore the process of disintegration and decay of rocks resulting from exposure to and the influence of atmospheric agents. Most rocks exposed at the surface are the product of processes which involve elevated pressure and/or elevated temperature. In contrast, in the near surface environment low temperatures prevail and there is little or no confining pressure. Additionally, water and oxygen is abundant. Consequently, the near surface rocks will undergo changes and slowly break down to unconsolidated material or soil.

Weathering is usually a slow process. However, the process of weathering depends on a wide range of different near surface environmental conditions found in different parts of the globe and the depth may vary widely in some of the cases. Depending on the fragmentation phenomenon of rocks, weathering may be classified as physical (mechanical), chemical and biological.

The types and intensity of weathering processes, particularly physical and chemical, depend on the climatic conditions in the area. Temperature and precipitation are the controlling factors and the dominant type of weathering that can be expected in a particular climatic region.

Physical Weathering

Physical weathering (also known as mechanical weathering) includes processes or agents that break down rocks into smaller particles by exerting forces that exceed the strength of the rocks. Physical weathering processes are generally the forerunners of the other two types of weathering and are the main contributors of surface area for chemical and biological attack. The constituents generally remain unaltered and the soil formed has the properties of the parent rock. Usually coarse-grained soils are formed due to physical weathering. The principal agents of physical weathering are unloading effects or stress release, periodical temperature changes, wedging action of ice, splitting action of plant roots, and abrasion of rocks due to wind, water and glacier.

Chemical Weathering

Most rocks originally formed at depths used to be in very different conditions of stress and temperature as compared to that existing at the earth’s surface. When rocks are exposed to the relatively much lower temperatures and pressures at the surface, they become chemically unstable and tend to react with the available constituents of the atmosphere to form more stable new minerals. The exposure of rock surfaces to atmospheric reactants like water, oxygen, carbon dioxide etc. fosters chemical reactions to decompose the rock into soil. This is known as chemical weathering and it is basically a surface phenomenon. When chemical weathering takes place, the parent rock minerals are transformed into new minerals. The soils formed may not have the properties of the parent rock.

Biological Weathering

Biological weathering or biotic weathering has provided a separate entity to describe mechanical and chemical changes of the rocks associated with the direct activities of plants and animals. Plants retain moisture and thus keep the rock surfaces on which they grow, damp. This helps the solution effect to the rocks. The chemical decay of rocks is also aided by the formation of vegetable humus derived from plants and helped by the action of bacteria and fungi. Humic or organic acids are thereby added to percolating rainwater, increasing its acidity or solvent power. Bacteria species may live in aerobic and anaerobic pore spaces of weathered rock and mobilize chemicals like C, N, Fe, S and O, thus assisting the process of weathering. Furthermore, some of the insects secrete acidic enzymes, which also help the chemical disintegration of rocks. The physical breakdown of rocks is also done by the wedging action of plant roots.

Based on Mode of Formation

Based on their mode of formation, soils can be divided into two categories that are residual soil and transported soil. The third category is the organic and peat soil. There are also other special soils such as collapsible and expansive soils.

Soils formed due to the process of in situ weathering of rocks and not transported elsewhere are called residual soils, while those that are transported and deposited elsewhere are termed as transported soils. Transport agents here refer to water, wind, glacier as well as gravitational forces.

The extent of soils is a three dimensional system: two aerial dimensions and a depth dimension. Whether they are transported or residual deposits, soil shows aerial limitations and changes with depth. Horizontal as well as vertical transition into another soil type may be gradual or abrupt depending on modes of formation and transportation. When vertical changes are caused in transported soil, the resulting layers are called strata; while in the case of residual soil, the layers are usually termed as horizons. The set of horizons, from soil surface to the original or physically unaltered parent rock, is known as the profile.

Residual Soils

Soils which are formed due to the process of in situ weathering of rocks and not transported elsewhere are called the residual soils. Such soils may retain some characteristics similar to that of the parent rock.

Figure 2 shows a general profile of residual (granitic) soils. The transition from fresh (un-weathered) rock, to partially weathered to fully weathered rock (soil) is denoted by grades or zones of I to VI.

In tropical countries like Malaysia, where the chemical and biological weathering is quite intense, residual soils formed may reach thickness of several hundred meters while in cold and arid climate, little or no residual soils will be formed.

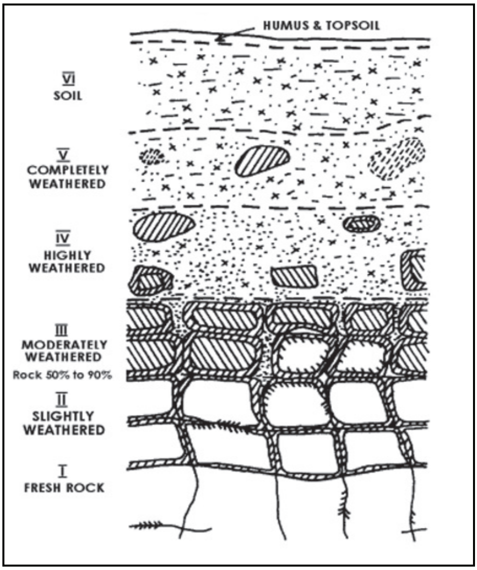


Figure 2 Classification of residual soil based on weathered degree (after Little, 1969)

Types of soils formed depend on the parent rocks. For example sandy residual soils are formed from granite rocks; Igneous and metamorphic rocks result in soils from silt type to gravel; while sedimentary rocks like shale will form residual soil with higher clay content.

Saprolite is residual soil that has not been completely weathered and still contains lumps of rocks and structures of the original parent rock.

Transported Soils

Transported soils are soils that have been transported and deposited elsewhere by agents such as water, wind, glacier as well as gravitational forces. These are the alluvial, glacier, lacustrine and marine, aeolian and colluvial soils.

Alluvial soils, which are also known as fluvial soils or alluvium, are soils that are transported by rivers. These soils are normally found in engineering because many engineering structures are actually built on them. This is because many major towns and cities are located in areas close to rivers, river mouths and deltas where these types of soils are usually found. Upstream of the river, where the flow of water is quite fast, particles of clay and silt are present in the form of suspension. Only sand and gravel will be deposited. However when the water flow becomes slower, say at the river mouth and where the river meets the sea, fine soil particles (silt and clay) will in turn be deposited.

Colluvial soils are soils that are transported downhill by action of gravity.

Special Soils

Special soils include organic soils, peat soils, and collapsible as well as expansive soils.

Organic soils are soils which contain organic matters from accumulation of plant remains. These soils are normally formed when the rate of accumulation exceeds that of decay; say in waterlogged areas such in the swamp. Organic soils are soils with more than 20% organic matter, while, peat is organic soil with more than 75% organic matter. Other components of the soils may include silt and clay.

Collapsible soils (also known as metastable soils) will remain strong and stable as long as they remain dry. However, if they become wet, they will quickly collapse thereby generating unexpected settlement. The process of collapse is sometimes called hydroconsolidation, hydrocompression or hydrocollapse. These soils are usually of sand and silt size particles with honeycomb structures. This type of soils can be found in alluvial, colluvial and residual soils.

WHAT IS TROPICAL SOIL?

Malaysia lies in an area with tropical climate (the Acategory climate in the Köppen Classification System). This climate is extensive, occupying almost all of the continents between latitudes 20°N to 20°S of the equator.

The key criterion for an A category climate is for the coolest month to have a temperature of more than 18ºC making it the only true winterless climate category of the world. The consistent day length and almost perpendicular sun angle throughout the year generates temperatures above 18ºC. Another characteristic is the prevalence of moisture. Warm, moist and unstable air masses frequent the oceans at these latitudes. As a consequence, this climate zone has abundant sources of moisture giving rise to high humidity.

Due to this climatic condition, weathering of parent rocks (igneous, sedimentary or metamorphic), mainly chemical weathering, is the main agent for soil formation in tropical countries like Malaysia. The soils formed by weathering are largely left in place, thereby literally called residual soil, and whose character depends on the parent rock it develops from. For example, residual soil on weathered granite will initially be sandy, as sand-sized particles of quartz and partially weathered feldspar are released from the granite. The partially weathered feldspar grains will gradually over time further completely weather into fine-grained clay minerals. As the resistant quartz does not weather, the resulting soil will have both sand-sized quartz and clay. This will further change over time as this residual soil that develops from granite may become more clayey. However, the influence of the parent rock decreases over the passage of time. After a sufficient time period, the differences in the residual soils from different types of rocks, i.e. igneous, sedimentary and metamorphic, may be obliterated.

The presence or absence of coarse grained quartz in the parent rock becomes the only vestige that survives and has a long-term significance.

In engineering terms, these soils are generally regarded as having poor to average properties. Detailed descriptions of the characteristic and properties of Malaysian tropical residual soils are given by Huat & Ali (2007).

In addition to the residual soils, transported soils are also found in Malaysia, though in terms of coverage but not necessarily in the order of significance, they are less than the residual soils. Transported soils by definition are soils that are formed from materials formed elsewhere that have moved to the present site where they constitute the unconsolidated superficial layer. The physical processes through the operation of their agents of transportation, i.e. mainly gravity and water, dislodge, erode and transport the soil particles to their present location.

Figure 3 shows a simplified soil map of Malaysia. About 70 percent of the country's land area is covered by residual type soils, while the remaining 30% is marked as alluvial, which is under the category of transported soils. By virtue of their location, which is mostly along the coast, these soils are called coastal alluvium. Alluvial soils, which are also known as fluvial soils or alluvium, are soils that are transported by rivers. Marine soils are also deposited in water, but by salt water. These soils are normally found in engineering because many engineering structures are actually built on them. They often fall into the category of soft soils because of their low strength and high compressibility.

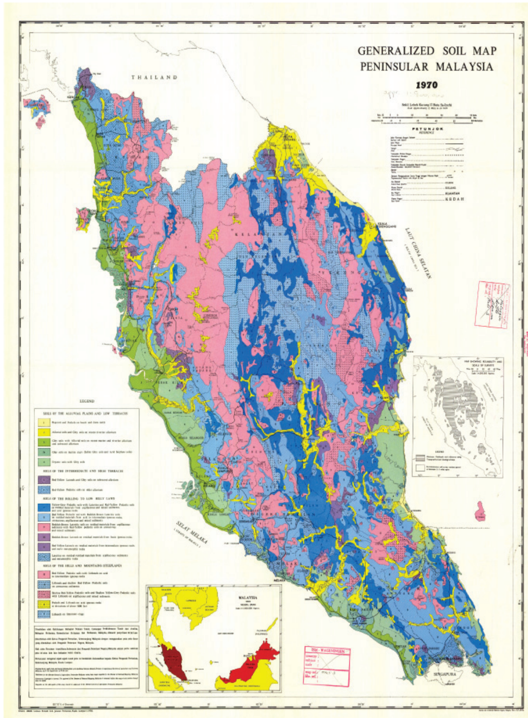


Figure 3 Simplified map of the soil distribution of Peninsular Malaysia

Another type of soil that can be found in Malaysia is the organic soil. By definition, organic soils are soils with more than 20% organic matter while, peat is organic soil with more than 75% organic matter. Peat actually represents an accumulation of disintegrated plant remains, which have been preserved under conditions of incomplete aeration and high water content. It accumulates wherever the conditions are suitable, that is, in areas with excess rainfall and poorly drained ground, irrespective of latitude or altitude. Nonetheless, peat deposits tend to be most common in those regions with comparatively cool wet climate.

Physico-chemical and biochemical processes cause the organic material to remain in a state of preservation over a long period of time. In other words, waterlogged poorly drained conditions, not only favor the growth of particular types of vegetation but also help preserve the plant remains.

Nearly two thirds of the world tropical peat lands, of about 30 million hectares, are found in Southeast Asia. In Malaysia, some 3 million hectares (about 8%) of the country's land is covered with organic soils/peat (Figure 4). In terms of thickness, these deposits vary from just a few meters from the ground surface to tens of meters.

Figure 4 Simplified map of organic soils of Malaysia

WHY IS SOIL A PROBLEM?
(Soils of Malaysia in Particular)

"*In reality soil is not exactly a problem, the theory is*"

In general almost all civil engineering projects are constructed on earth, and some may even be constructed from earth materials itself, such as embankments and dams. All these projects will therefore require some geotechnical engineering input. Among issues that need to be addressed by an engineer in this field are (Coduto, 2007):

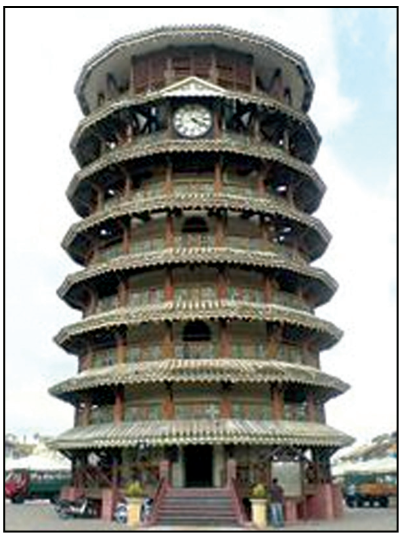
- Is the soil or rock beneath the construction site capable to safely support the structure that is going to be built?
- If foundation is required to support the structure, the question would be – what type of foundation and its method of design.
- Suitable methods of soil improvement to improve the properties of in situ soil if their engineering properties are too poor, like soft clays and peat.
- If retaining structures are required, what would be the best type and how to design them?
- Are the proposed slopes or existing natural slopes stable? If not, how can they be stabilized?
- Impact on the project from natural calamities such as earthquake.

Problems Associated with Soft Ground

Many towns and cities are actually located close to river mouths, deltas, lakes or alluvial flats. These are usually areas with soft compressible soils. These soils usually have low undrained shear strength with high void ratio (high compressibility). Other areas that bring many problems to engineering are swampy and peaty areas where the soils have high organic content.

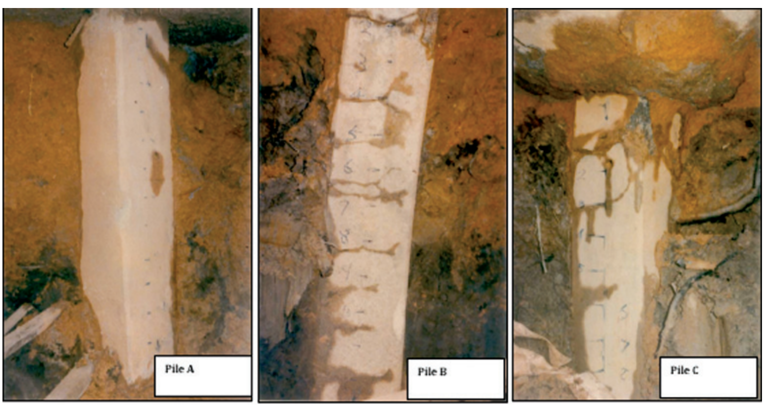
Constructions on soft ground, such as building embankments for roads, railway or landfill for housing projects may bring about problems associated with instability during construction, and long term and persistent settlement.

Figure 5 provides a pictorial view of some of the problems associated with soft grounds in Malaysia.



(a) Malaysia's '*leaning tower*', Teluk Intan, Perak.
The lean was due to differential settlement of the tower foundation

(b) Flooding and damage to infrastructure in Sibu Town (Sarawak), both attributed to subsidence of peat upon which most of the town was built



(c) Bridge pile failures, attributed to soil lateral movement (Gue & Tan, 2003)

(d) Abutment tilt and gap between bridge decks due to foundation failure on soft ground (Gue & Tan, 2003)

(e) Pile broken due to excessive lateral movement of soft soil in open excavation during construction of a lift shaft for the Majestic Hotel, Malacca, 2007

Figure 5 Problems associated with soft ground

Landslides

Landslides are a serious geologic hazard common in most countries around the world. Globally, landslides cause billions of dollars in damage and thousands of deaths and injuries each year.

Some slopes are susceptible to landslides whereas others are more stable. Many factors contribute to the instability of slopes, but the main controlling factors are the nature of the underlying bedrock and soil (material properties, planes of weakness such as beds, joints or factures), the configuration of the slope, the geometry of the slope, and groundwater conditions. Landslides are typically associated with periods of heavy rainfall. Antecedent rainfall can cause minor landslides, especially for cases of shallow landslides in residual soils. In areas which experience forest and brush fires, a lower threshold of precipitation may initiate landslides.

In Malaysia, the annual rainfall can reach as high as 4500 mm. This combined with high temperatures around the year causes intense chemical weathering and formation of thick residual soil profiles which in certain locations can reach up to 100 m in depth. With these sets of climatic and geological conditions, combined with other causative factors, landslides are among the most destructive natural disaster forces in Malaysia. Since 1975, some 400 landslides have been reported in Malaysia (more than 30 were major landslides), involving both cut and natural slopes, with a total loss of more than 200 lives and billions of Ringgit in damage to properties.

Most landslides occur during or following short periods (<3hr) of intense rainfall (when total rainfall >70 mm) or longer periods (>1 day) with somewhat continuous rainfall (Raj, 2004).

Figure 6 shows some of the major landslides in Malaysia in recent years.

(a) Collapse of the Highland Tower, Ulu Kelang, Selangor in December 1993. 49 lives were lost, only 2 survived, a maid and a baby girl. Fortunately most of the occupants were at-work at the time of the disaster. The remaining 2 blocks have been left unoccupied till today

(b) Debris flow during heavy down-pour at the Karak Highway – Genting Highland slip road (road to one of Malaysia's top hill resort destinations) in June 1995 killed 22 whilst 23 others were injured

(c) Deep seated rotational slide of an 'engineered' anchored slope at Gua Tempurung, Kampar, Perak in January 1996. 1 person died

(d) A bungalow house at Taman Hill view, Ulu Klang, Kuala Lumpur dilapidated by landslide in November 2002. 8 people were killed

(e) Rock fall at Bukit Lanjan, New Klang Valley Expressway, Selangor in November 2003. A major rock fall in the country which led to closure of one of the main gateways to Kuala Lumpur for more than 2 months

(f) Landslide at Kampung Pasir, Ulu Klang, Kuala Lumpur in May 2006. 4 were killed when one longhouse was buried under the avalanche of earth debris

(g) Landslide on March 22, 2007, Precinct 9 Putra Jaya after a period of heavy rain. 23 cars were buried under the rubble. 1200 people were evacuated (Photos courtesy of the New Strait Times)

(h) Landslide at Kampung Cina, Kapit on December 27, 2007 during a heavy downpour. 12 houses were destroyed involving 17 families (76 people). 9 were injured, 4 were lost

(i) Landslide at Jalan Semantan Kuala Lumpur during a heavy downpour on December 4, 2008. Several parked cars were damaged (photo courtesy of Klue)

(j) Landslide at Bukit Antarabangsa, Kuala Lumpur (an up-scale residential area) on December 6, 2008. 5 people died (photo courtesy of Google)

(k) Taman Awana Cheras landslide on November 18, 2009 following a period of heavy rain

Figure 6 Some recent landslide incidences in Malaysia

Note: All materials summarized in above articles can be found in detail in the following reference books published by Universiti Putra Malaysia Press:

1. (2004). *Organic and Peat Soil Engineering*. University Putra Malaysia Press. ISBN 983-2871-08-5.
2. (2007). *Essential Soil Mechanics for Engineers*. University Putra Malaysia Press. ISBN 983-3455-55-7.
3. (2007). *Ground Improvement Techniques*. University Putra Malaysia Press. ISBN 983-3455-56-5.
4. (2008). *Landslides in Malaysia: Occurrences, Assessment, Analysis and Remediation/Preventive Solutions*. University Putra Malaysia Press. ISBN 978-967-5026-39-3.

MEETING SOME OF THE CHALLENGES
(Some Recent Research Works Done at University Putra Malaysia)

Pile Support Problems due to Lateral Movement of Soil

As soft soils move laterally past piles, passive lateral pressures may be applied to the piles. In a number of design situations, piles have to be designed for the effect of lateral soil movement. These include piles in or near an embankment built on soft clay (Figure 7), bridge abutment in soft ground, piles adjacent to an excavation, piles in unstable slopes and piles in a marginally stable riverbank. These piles are called *passive* piles. The design of such piles may be based on the assumption that forces from moving soil will act against the piles and ‘squeeze’ past the piles. As these piles will experience additional stress and strain, failure to assess the effect in design will result in unacceptable pile movement or stress, or both. In Malaysia, design of passive piles is still quite new to many practicing engineers. Problems of pile support due to lateral soil movement, though not uncommon in Malaysia, are sometimes not recognized or well understood.

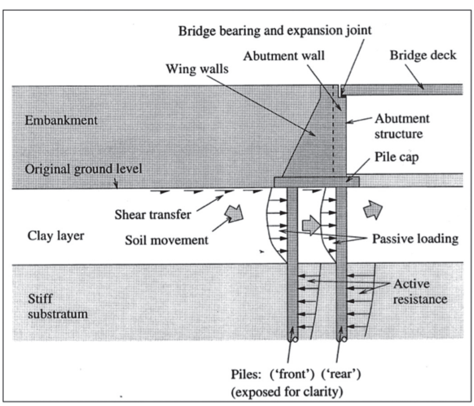


Figure 7 Passive pile in an embankment subjected to horizontal loading from moving soil past the piles (Springman, 1989)

During construction of deep foundations, it is the norm of practice that excavation/earthworks are done prior to the installation of piles. Nevertheless, in many cases, due to space constraint, installation of piles have to be carried out prior to excavation work. Open excavation can be carried out without temporary support up to a limited depth. For excavation depth which exceeds the critical depth, normally temporary retaining structures such as sheet piles will be necessary. This temporary retaining structure is also crucial to protect adjacent existing structures against lateral soil movements arising from the excavation.

The issue of piles within an excavation is a relevant problem in practice. From a behavior viewpoint, such piles are known as passive piles, which will be subjected to predominantly lateral load from horizontal soil movement and upward ground movements (base heave) and so can have tensile forces induced in them as well as bending. Tension cracks in piles are commonly observed where excavation induced bending moment in the pile exceeds the pile's cracking moment capacity. A combination of tension and bending can be dangerous for pre-cast or bored concrete piles and failure could be a real possibility. Figure 8 shows a case study of the Majestic Hotel, Malacca (February 2007) where a pile group failed due to excavation in soft marine clay.

Figure 8 Pile failure in an open excavation

A study to this effect was carried out from 2006 till present with the main objective being to understand pile response to passive loading based on a laboratory scale model and 3D FEM using Plaxis 3D Foundation software. A suite of design charts for predicting passive pile response in open excavation, especially in soft clay underlain by a hard layer, was then developed for practical use.

1-g laboratory simulation of piles in excavation

For this study a model test tank with the dimensions 300 mm (W) x 800 mm (L) x 600 mm (D) was fabricated with an 8mm thick acrylic resin wall with each corner reinforced with steel plate as shown in Figure 9. Aluminium alloy tubes of 12.3 mm in outer diameter with a wall thickness of 0.81mm were used as the model pile. The model test pile was instrumented with strain gauges. Each pile had strain gauges in total five on each side of a neutral axis.

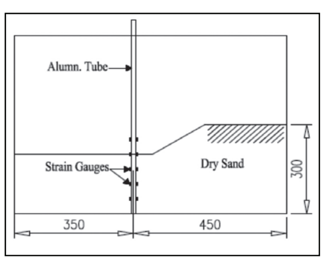
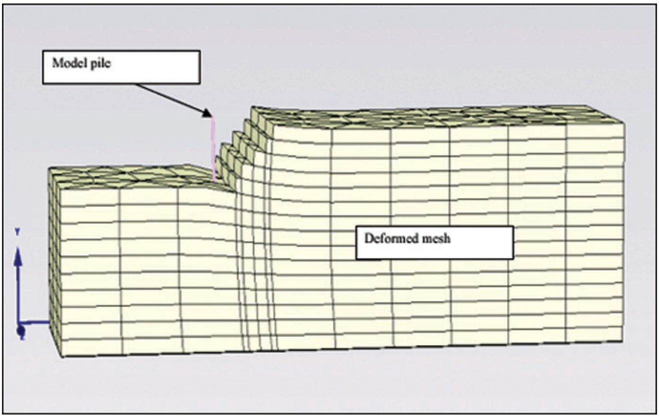
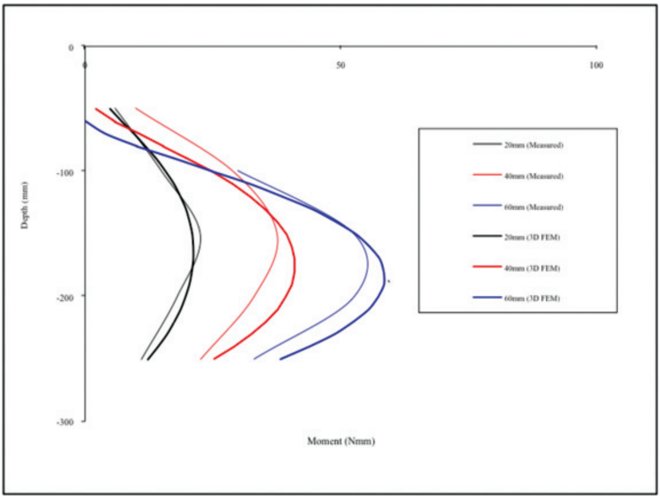


Figure 9 1-g model test (tank and pile, and set-up for a single pile in open excavation)

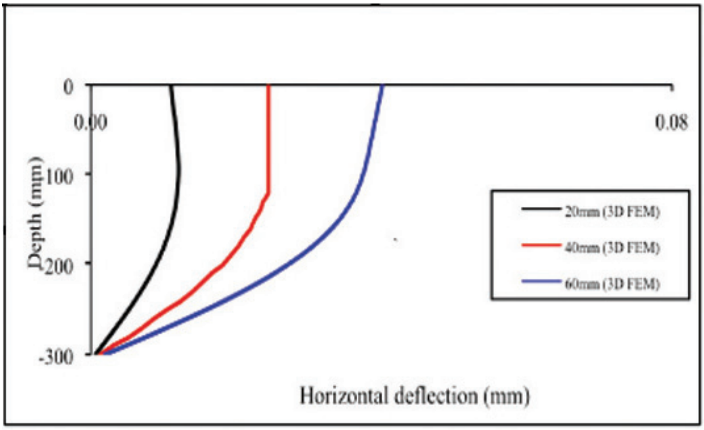
Figure 10 shows results of the FEM model by PLAXIS 3D Foundation. Both the experimental and FE results were apparently in good agreement.



(a) Deformed mesh of the single pile in open excavation



(b) Pile bending moment



(c) Pile deflection

Figure 10 FE model, bending moment and horizontal deflection of pile

Case Study: Passive Piles Failure in Open Excavation

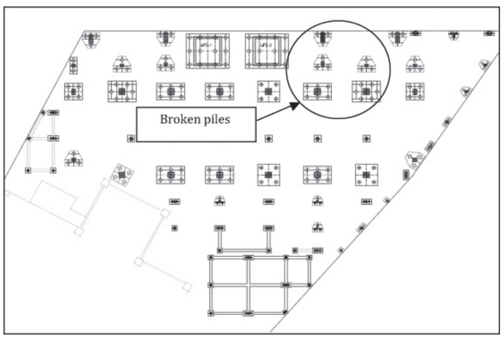
The Majestic Hotel is located in Malacca, the capital of a state of the same name - one of the fourteen Malaysian states. Situated on the South-Western Coast of Peninsular Malaysia, facing the Straits of Malacca, it lies between the states of Negeri Sembilan and Johor. Malacca is situated approximately 150 km south of Kuala Lumpur and 260 km north of Singapore. Originally, Majestic Malacca was as a mansion, dating back to the 1920s, and even after refurbishing works, it still remains at the heart of the hotel. A new building had been built to house 54 spacious rooms and suites.

The site was located directly on a 5 to 7 m thick marine clay site adjacent to a river in Malacca. Generally, the subsoil profile of the site could be divided into three main strata namely; very soft marine clay, very stiff clay and hard silt with intermittent 3 to 4 m thickness of medium dense to very dense gravel ranging from depths of 14.5 to 32 m.

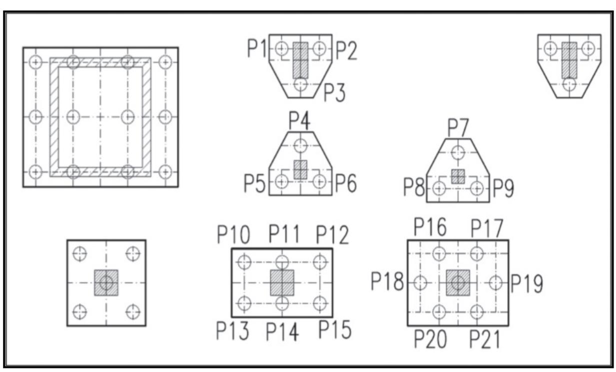
The top 300 mm layer of soil showed SPT 'N' value of 4-6 blows, followed with a SPT 'N' value average of zero with thickness of 5.0 to 7.0 m. Undrained samples collected at depths of 3.0 to 4.5 m were subjected to undrained unconsolidated tests in the laboratory which gave the undrained shear strength of 16 kN/m^2 to 20 kN/m^2. The unit weight of the very soft marine clay was about 16 kN/m^3 to17.5 kN/m^3. The liquid limit of the soil was high, mostly at about 55 to 75%, while the Plasticity index ranged from 20 to 45%. The soil consisted of a very high percentage of silt and clay; in the range of 70 to 90%. Figure 11 shows the excavated very soft marine clay below.

Figure 11 Soft marine clay revealed during excavation work carried out in January 2007

The subsequent layer of soil was made up of a layer of stiff to very stiff silt. Generally, this layer had a thickness of 4 to 12 m. After this layer, a layer of hard silt was encountered with intermittent layer of 3 to 4 m medium to very dense gravel. The superstructure was designed to be supported by end-bearing piles comprising of a group of 300 mm diameter spun piles with nominal thickness of 60 mm. The piles were terminated at the hard layer on the very stiff silt or medium to very dense gravel layer. The excavation works started after the piles installation on January 2007 and by the end of the month, excessive deviation was observed, leading to the cracked pile group shown in Figures 8 and 13. Figure 12 shows the foundation layout indicating the location of the broken piles.



(a) Location



(b) Numbering

Figure 12 Foundation layout plans of the broken piles; (a) location, and (b) numbering

Figure 13 Broken piles P1, P2, P3, P5 and P6

Figure 14 illustrates the 3D FEM of the broken piles using Plaxis 3D Foundation. The excavation was simulated in phases as shown in Figure 15.

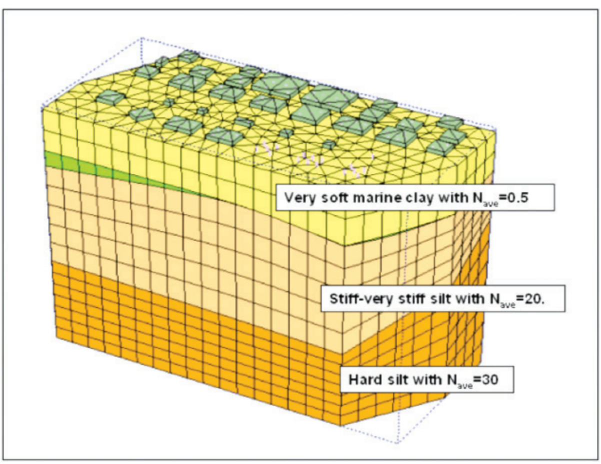
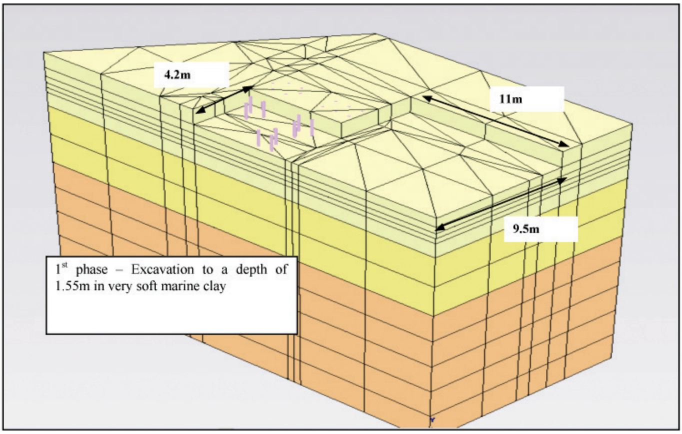
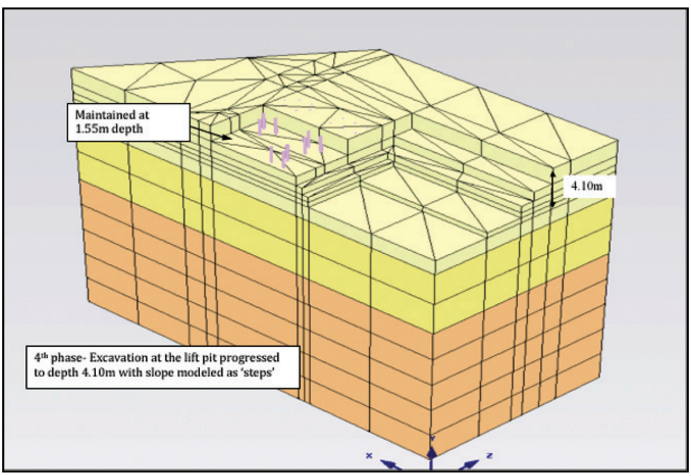


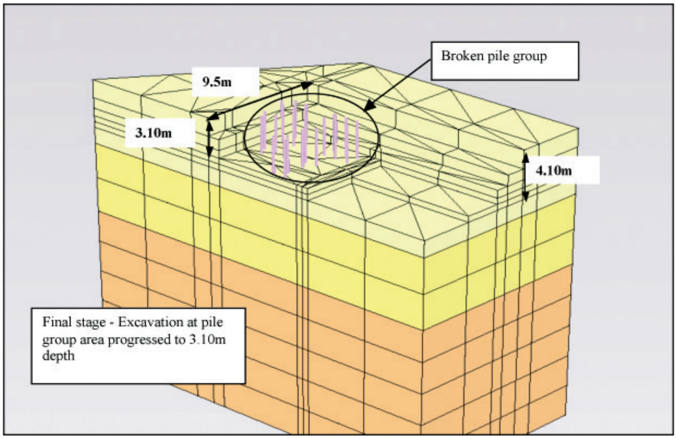
Figure 14 3D FEM model of the pile and foundation



(a) 1st excavation phase



(b) 4th excavation phase



(c) Final excavation phase

Figure 15 Excavation simulation

Hardening Soil model was applied as the soil constitutive model. Some basic characteristics of the model included stress dependant stiffness parameter according to a power law (*m*), plastic straining due to primary deviatoric loading (E_s), plastic straining due to primary compression (E_{oed}), unloading/reloading Young's Modulus (E_{ur}), Poisson's ratio (ν) and failure parameters made according to the Mohr-Coulomb model (c, φ, ψ). The depth of each layer corresponded to the information from the nearest borehole.

During execution of each phase, the response of the piles was compared against the pile cracking moment of 20.4 kNm. The pile which exceeded the cracking moment was observed from the initial stage until the end of the staged construction as summarized in Table 1. Pile response which exceeded the cracking moment was termed as cracked and those exceeding 80% of the cracking moment was assumed as critical piles. There were only four piles which remained as un-cracked piles.

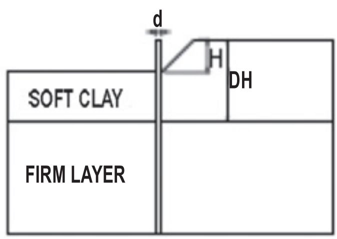
Table 1 Pile maximum bending moment from FE modeling

Pile no	Comment
1	60% of Cracking Moment- UNCRACKED
2	83% of Cracking Moment- CRITICAL
3	80% of Cracking Moment- CRITICAL
4	CRACKED
5	CRACKED
6	85% of Cracking Moment- CRITICAL
7	CRACKED
8	CRACKED
9	CRACKED
10	CRACKED
11	83% of Cracking Moment- CRITICAL
12	CRACKED
13	CRACKED
14	CRACKED
15	CRACKED
16	80% of Cracking Moment- CRITICAL
17	CRACKED
18	74% of Cracking Moment- CRITICAL
19	CRACKED
20	CRACKED
21	CRACKED

The results at the end of the excavation phase showed that 70% of the modeled piles had either reached the critical cracking moment or exceeded the pile's cracking moment, and were therefore confirmed as broken piles. Further excavation on site showed that cracked piles were found near to the transition layer of the very soft clay and very stiff silt, confirming that the location of the crack occurred at the location of maximum moment from the 3D FE analysis.

Design Charts for Predicting Passive Pile Response in Open Excavation

A single pile subjected to an open excavation in soft clay underlain by stiff clay was modeled using 3D FEM modeling with a H-S model, in undrained conditions. The geometry of the 3D FE model was also chosen to be wide enough such that boundary conditions were minimized. A few of the parameters involved in developing the design charts were based on the pile properties, soil properties and excavation configuration.



Excavation was carried out up to depths of 1m, 1.5m and 2m. For each of the depths, three slope configurations were analyzed, namely 1:1, 1:2 and 1:3. The depth of the hard clay layer from the excavated level will be represented by the *depth function*, D as shown in the diagram above.

Spun piles of commonly used sizes of 300mm dia (60mm thick), 450mm dia (70mm thick) and 600 mm dia (90 mm thick) were modeled by using the *Embedded Pile* option in PLAXIS 3D. The sizes of the piles area were adapted from ICP Piles under Class A (effective Pre-stress = 4.0N/mm^2). The Young Modulus of the pile was inputted as 30,000 MPa, as normally used for concrete.

The constant soil properties inputted in the model were basically the soil unit weight, γ = 18 kN/m^3 with friction angle, ϕ = 0 in an undrained condition. The soil Young modulus, E_u corresponds to 300c_u, which was in the range of 150- 400c_u(Poulos & Davis, 1980) which was suitable for soft clay under lateral loadings. Regardless of E_u, the H-S model required the effective stiffness parameter, E' when the calculation mode was set to undrained. Hence, a simple correlation was used where $E'=E_u/1.15$ as proposed by Ong *et. al.* (2006). For simplicity, the stiffness parameter was shown as E_u.

Three soil strength parameters, c_u, suitable for soft clay were chosen i.e. 10 kPa, 20 kPa and 30 kPa, which would correspond to each respective soil stiffness parameter as described. Therefore, the resulting three cases of E_u were 3000, 6000 and 9000 kPa. The undrained shear strength for the hard layer was taken as 250 kPa, which would correspond to E_u=75,000 kPa. The Hardening-Soil model was used and verified by back-analysis of published centrifuge data and a case study. The value of *m* was taken as 0.5 and Poisson ratio,ν, was as per the default setting of 0.2.

Figure 16 below shows the proposed geometry for the parametric study. The excavation will be modeled as 'incremental steps' which will form the intended slope. The results from the parametric study are still under progress.

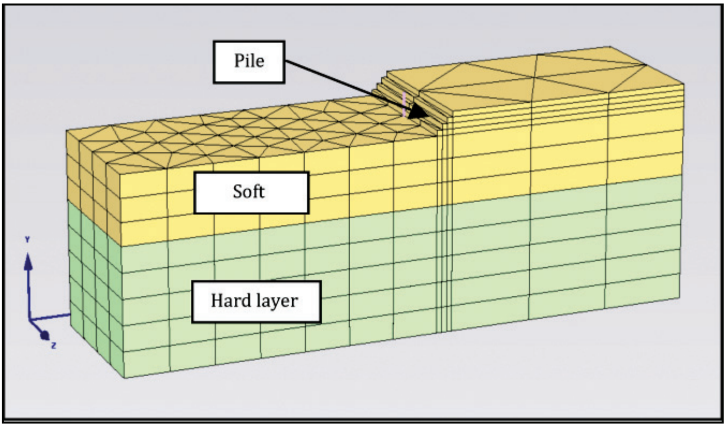


Figure 16 Proposed geometry for the 3D FEM model for the parametric study

Summary

The purpose of this work was to study the response of passive piles in open excavation by numerical modeling using the Plaxis 3D Foundation. The study showed that the advance version Plaxis software was capable of simulating a real case study of passive piles failure in one excavation. Initially, a laboratory scale test was carried out using loose sand to simulate the open excavation with the presence of a single pile represented by an aluminium tube. Back-analyses using the FE package showed satisfactory results. Subsequently, a case study of pile group failure was modeled to predict the response of the piles and results compared against the pile's cracking moment. Following this, a configuration was proposed for parametric study based on some parameters, namely soil properties, pile properties and excavation configuration. The results of the parametric study are still under progress and will be available soon.

Acknowledgement

This work was funded by the Ministry of Higher Education, under the Fundamental Research Grant Scheme 02-01-07-048FR - *Analysis of Lateral Movement and Development of Interaction Factors for Pile Group under Lateral Loadings.*

Note: Further details on materials summarized in this Section can be found in the following publications:

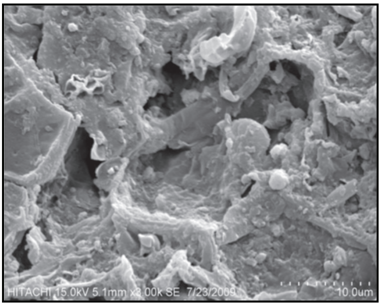
1. (2009). A Case Study of Passive Piles Failure in Open Excavation. *Deep Foundation Institute Journal.* 3(2)50-57
2. (2009). Modeling of Passive Piles – An Overview. *Electronic Journal of Geotechnical Engineering* 14P
3. (2009). A Review of Basic Soil Constitutive Models for Geotechnical Application. *Electronic Journal of Geotechnical Engineering*. 14J
4. (2008). Numerical Modeling of Laterally Loaded Piles *American Journal of Applied Sciences.* USA:NY, 5(10) 1403-1408

Stabilization of Peat

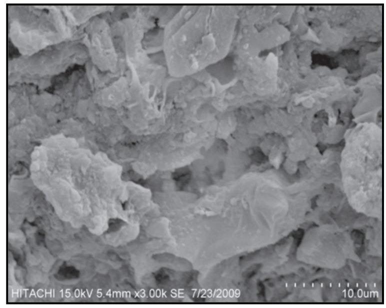
(What a sinking feeling……)

Peat actually represents an accumulation of disintegrated plant remains, which have been preserved under conditions of incomplete aeration and high water content. These peat or peaty soils are extreme examples of problematic soils. They are subject to instability, such as localized sinking and slip failure, and massive primary and long-term settlement when subjected to even moderate load increase. Organic soils are soils with more than 20% organic matter, while, peat is organic soil with more than 75% organic matter. Peat or peaty soils generally have very high natural water content, which can be in excess of 1000%, compared with mineral soils (sand, silt and clay), whose values in the field may range only between 20 to 80% . In terms of unit weight, peat is both low and variable compared with mineral soils, being related to the organic content, mineral content, water content and degree of saturation. The average unit weight of peat is typically just slightly higher than water. Amorphous peat has a higher bulk density than fibrous peat. For instance, in the former it can range up to 10 kN/m^3, whilst, in fibrous (woody) peat it may be half this figure. These *soils* are generally so highly compressible that in their natural condition they can only carry very little load. The values of the compression index of tropical (amorphous) peat can be as high as 5 to 10 compared with soft clays which is only 0.3 – 3. In terms of strength, their values are equally low. The undrained shear strength of (amorphous) peat is in the range of 3-15 kPa only, which is generally much lower than those of mineral soils. The *soils* may also change chemically and biologically with time. For example further humification of the organic constituents would alter the soil mechanical properties such as compressibility, shear strength and hydraulic conductivity. Lowering of ground water may cause shrinking and oxidation of peat leading to humification with consequent increase in permeability and compressibility.

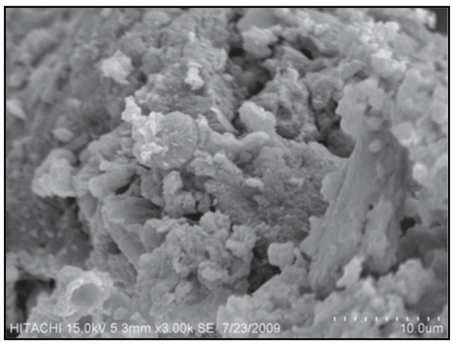
Peat structures, in their various aspects, also affect their engineering behavior. Scanning Electron Microscopy images as shown in Figure 17 can be used for visual appreciation of the soil microstructure which can range from fibrous or fibric (least decomposed), to semi-fibrous or hemic (intermediate) and amorphous or sapric (most decomposed) (ASTM Standard D 5715).



(a) fibrous peat



(b) hemic peat



(c) sapric peat

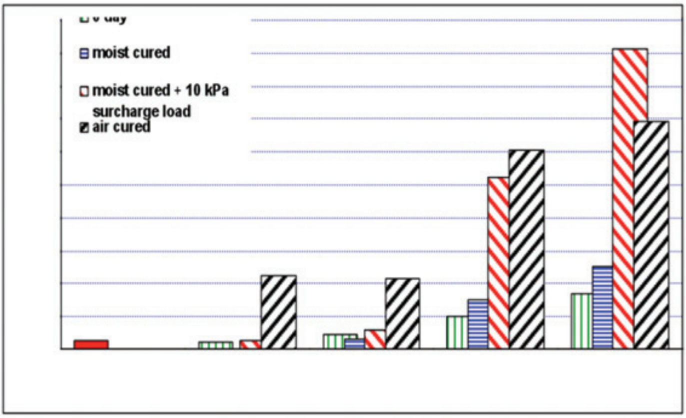
Figure 17 Scanning Electron Micrograph of peat (samples from Kampong Jawa, Kelang)

Arguably, it is probably best to leave peat lands to serve as forest reserves especially in times when global warming is a major concern, as they are the most efficient terrestrial ecosystems in storing carbon, critical for biodiversity conservation and play a key role in water resources management. However, the vastness of the peat lands and their occurrence close to or within population centers and existing cropped areas make some form of infrastructure development inevitable. These would be in the form of road crossings, and in some instances, housing developments that encroach into the peat lands, as land becomes more and more scare. To simulate agriculture development for instance, basic civil engineering infrastructures such as irrigation and drainage, water supply, roads, farm buildings, etc are required. All these call for better and greater understanding of the soil characteristics to make the construction more manageable. It is therefore critical to clearly understand the characteristics of peat. The high water tables, lack of topographic relief and dynamics of their soil properties, which set them apart from mineral soils. Criteria based on mineral soils cannot be generally applied to peaty soil. To meet this challenge, a series of studies have been carried out since 1997, from characterization to understanding the mechanical (engineering) properties of peat to stabilizing peat using chemical additives and electro-osmosis. In 2004 we published a book entitled *Organic and Peat Soil Engineering, ISBN 983-2871-08-5*, which has since become one of the main reference books on peaty soil engineering in the country. Earlier works on characterization and engineering properties of organic soils and peat are available in this book.

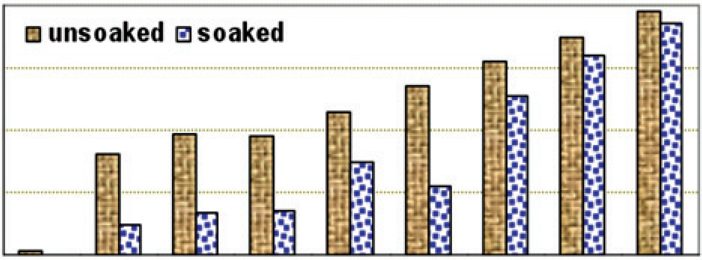
In 2004 we started work to look into the possibility of improving the mechanical properties of peat in terms of (shear) strength gain and improvement in compressibility characteristics. The use of chemical binder (OPC - ordinary Portland cement) and a series of other additives such as polypropylene fibers, steel fibers, silica fume (micro silica), ground granulated blast furnace slag and fly ash were investigated for both cases of shallow (surface) and deep stabilization.

Shallow (Surface) Stabilization

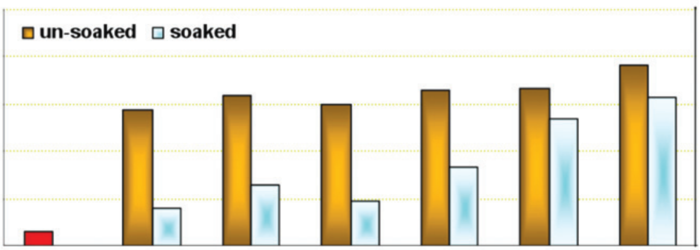
The effects of adding various percentages of chemical binders (OPC) from 5 to 50% in terms of wet weight of peat and other additives, polypropylene fibers, steel fibers, silica fume (micro silica), ground granulated blast furnace slag and fly ash, on the strength of fibrous peat were investigated. The soil strengths were studied in terms of unconfined compressive strength (UCS) and California Bearing Ratio (CBR). The effect of sample curing – moist cured, air moist cured with surcharge and air cured was investigated. The results are shown in Figure 18. In general adding the chemical additives proved to have increased the soil strength substantially. Addition of additives was also beneficial, except for blast furnace slag, and fly ash where there appear to be no additional gain. Air curing was found to be generally better than moist curing. The best additive was apparently the polypropylene fibers (about 0.15% or 1.5 kg polypropylene fiber per cubic meter of peat at its natural water content).



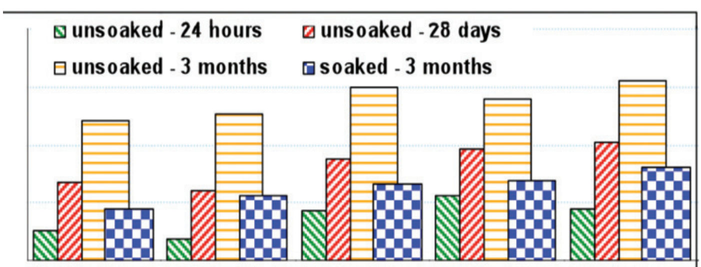
(a) Unconfined Compressive Strength of untreated fibrous peat and peat treated with OPC after 90 days curing



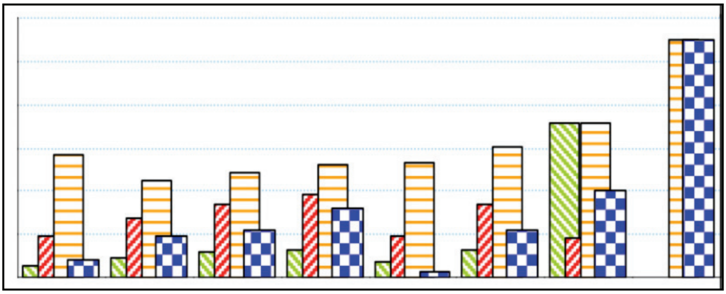
(b) CBR of untreated fibrous peat and peat treated with OPC and polypropylene fibers after 90 days curing



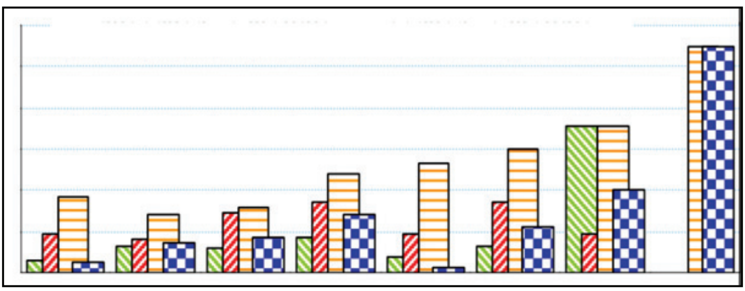
(c) Unconfined Compressive Strength of untreated fibrous peat and peat treated with OPC and silica fume after 3 months curing



(d) CBR of untreated fibrous peat and peat treated with OPC, polypropylene and steel fibers after 3 months curing



(e) CBR of untreated fibrous peat and peat treated with OPC and blast furnace slag after 3 months curing



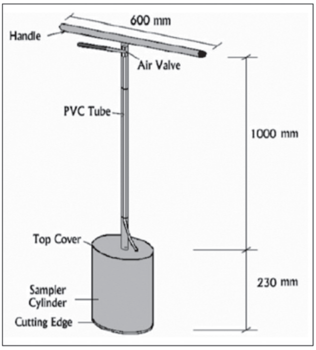
(f) CBR of untreated fibrous peat and peat treated with OPC and fuel ash after 3 months curing

Figure 18 Effects of OPC and additives on strength of fibrous peat.

Deep stabilization

Deep stabilization involved installing 'columns' of peat added with OPC and other additives into pre-bored holes set in peat samples. This stabilization technique is akin to that of cement or sand columns applied to soft clays. Peat with cement columns were then tested in triaxial for strength and Rowe cell for compressibility. Both these tests required 'undisturbed' samples of peat to be obtained from the field. This proved to be challenging as peat is not only very soft in nature but exists in waterlogged areas. A special sampler was designed and fabricated to overcome this problem.

The sampler consisted of a thin hollow cylindrical tube 150 mm in diameter (internal) and 230 mm high, as shown in Figure 19. The upper part of the cylindrical hollow body is fitted with a cover plate.



(a)

(b)

Figure 19 Peat sampler: (a) schematic diagram, and (b) various components of the sampler

The lower part of the cylindrical tube had a sharp edge to cut roots as the auger is slowly rotated and pushed into the peaty ground during sampling. The thin tube was fitted with a valve which is left open during sampling to release both air and water pressure. The valve is then closed prior to withdrawal of the tube with the peat sample enclosed, thus providing a vacuum effect to help keep the sample in place. The handle was formed of a 600 mm cross bar and the stem of 1000 mm height. Soon after the sampler is withdrawn the cylindrical tube is sealed with paraffin wax.

The cement column technique involved casting in place a 20 mm diameter column (which gave an area ratio of 0.16) inside a 50 mm sample as illustrated in Figure 20. The column was pre-bored using a 20 mm PVC tube with a sharpened lower edge.

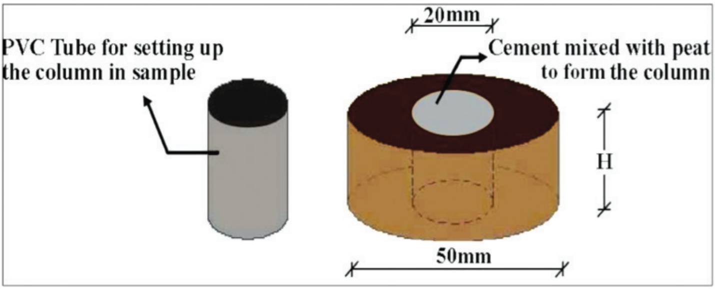
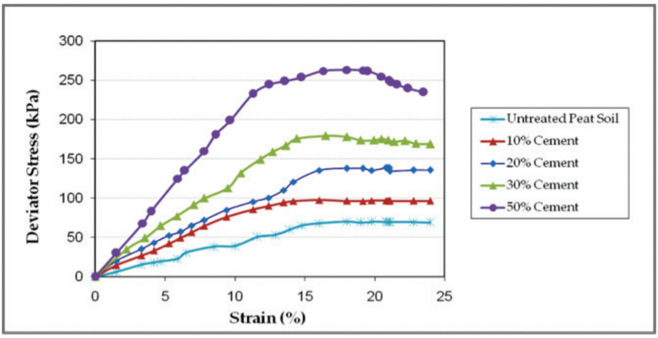


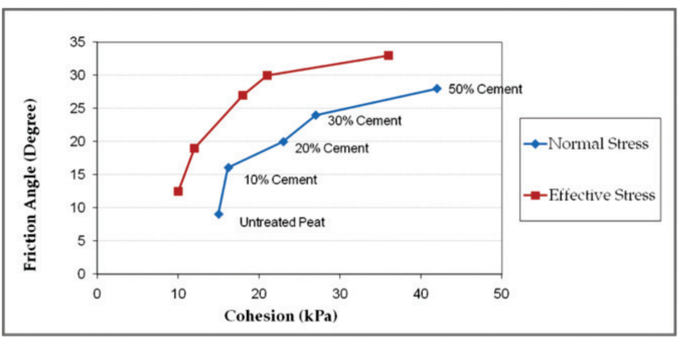
Figure 20 Cement column being formed in undisturbed peat

The cement column was formed by adding peat with varying percentages of cement (OPC), the proportions of cement to peat ratio chosen are (100:0), (75:25) and (25:75) compacted inside a PVC tube. The column was cured for 45 days in a soaking basin, before being inserted inside a pre-bored hole as shown above. Sample height for the shear strength and compressibility tests (Rowe cell) was 100 mm and 20 mm respectively.

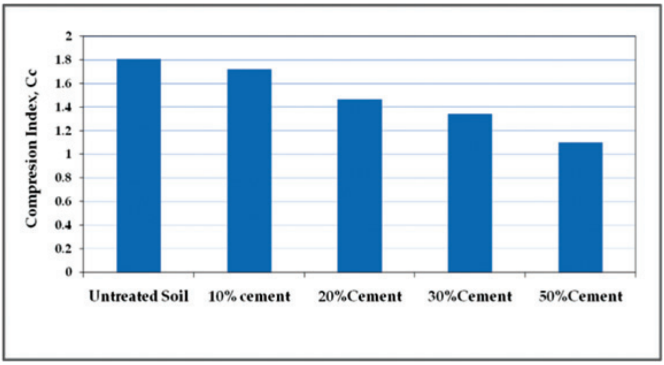
Figure 21 illustrates the effect of the cement column on the shear strength and compressibility of peat, measured with triaxial and Rowe cell tests respectively.



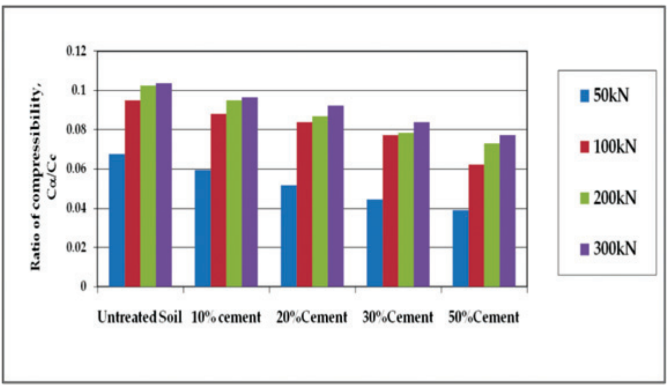
(a) Stress-strain curves of peat with cement column



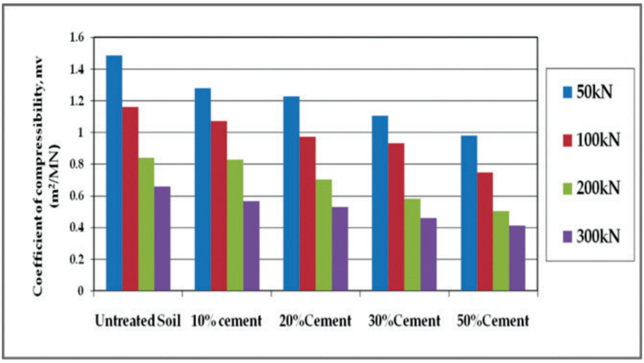
(b) Shear strength parameters of peat with cement column



(c) Compression index of peat with cement column



(d) Secondary compression (re-compression) index of peat with cement column



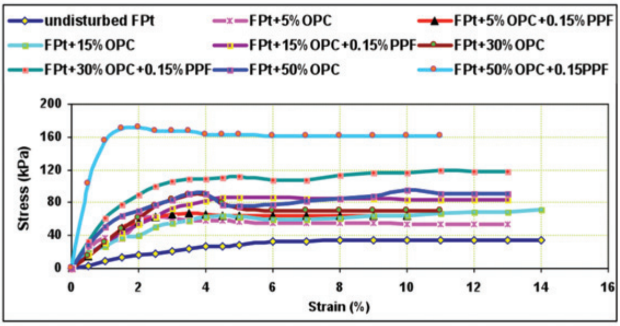
(e) Coefficient of compressibility of peat with cement column

Figure 21 Effects of precast cement columns on peat

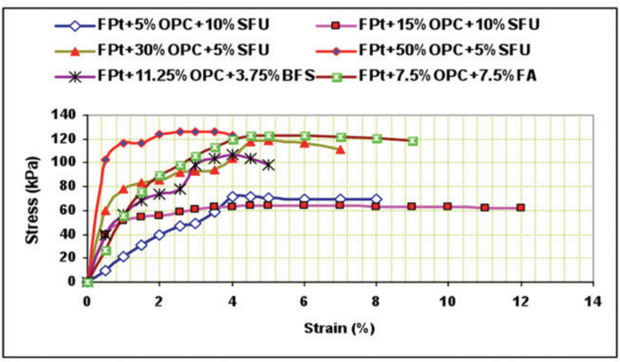
In general it was found that the cement column significantly improved both the shear strength (100% increase) and compressibility of peat, in particular the column with higher dosage of cement. At low cement content, the treated soil showed a ductile stress-strain relationship. With an increase in the cement content, the relationship changed from a ductile to a brittle one with an increase in the samples' strength. Sapric peat showed a well defined peak stress at failure as compared with hemic and fibric (fibrous) peat. The peat gained strength with the addition of cement and probably the presence of cations initiated the flocculation because of the cementation phenomena between the soil particles and cement. When tricalcium silicate (C_3S) and dicalcium silicate (C_2S) are mixed with water, calcium ions are quickly released into the solution with the formation of hydroxide ions. The degree of humification, or decomposition can greatly affect strength development. As humification progresses, the soil's pH, mineral content, bulk unit weight and cation exchange capacity increase (Hampton & Edil, 1998). This high cation exchange (CEC) capacity and humification may also be the reason for high increase in the strength of sapric peat compared with hemic and fibric (fibrous) peat. Factors affecting stabilized organic soil such as peat therefore depended upon: the water content; physical, chemical and mineralogical properties; nature and amount of organic content and the pH of pore water.

Adding additives to cement as well as air curing has been shown to be beneficial to peat. Additives (polypropylene fibers, steel fiber and silica fume) were then added to the cement column. The water content was set at peat optimum moisture as found from the Proctor compaction test. The mixture was then compacted into molds and left to dry. Figure 22 illustrates the process. Figure 23 shows the beneficial effects of installing the air-cured cement column on the shear strength (expressed in terms of stress-strain ratio) of fibrous peat.

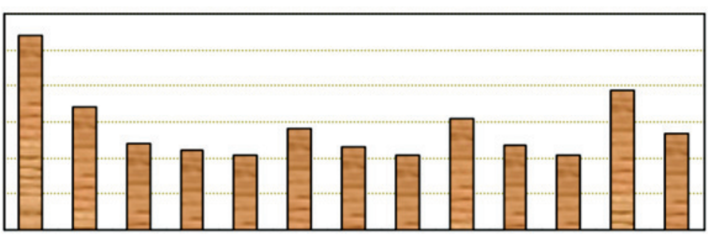
Figure 22 Making *precast* peat column



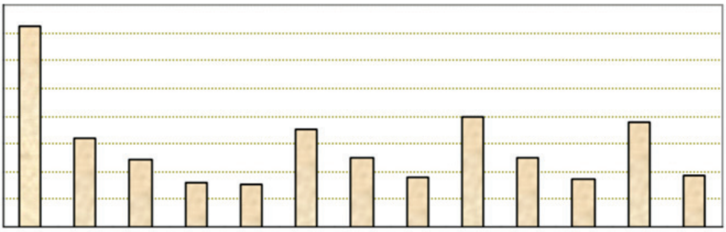
(a) Stress-strain of untreated peat and peat stabilized with OPC with or without polypropylene fiber columns



(b) Stress-strain of untreated peat and peat stabilized with OPC and additives (silica fume, blast furnace slag and fly ash) columns



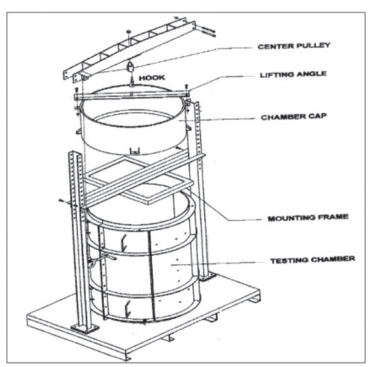
(c) Compressibility ratio of untreated peat and peat treated with precast cement columns



(d) Recompression ratio of untreated peat and peat treated with precast cement columns

Figure 23 Strength and compressibility of untreated peat and peat treated with pre-cast cement columns

To simulate field conditions, tests were carried out at larger scale. To do this, we had to fabricate a special test chamber 1.6m high and 820 mm in diameter, as shown in Figure 24. Instrumentation included pore pressure gauge, LVDT, horizontal strain gauge and pressure cell. During each test, vertical deformations at both sides of the plate load, pore pressures at the top and at the bottom of the tested column, pressure cell at the bottom of the column, and applied load on the plate load by means of load cell and hydraulic jack were recorded using TM loggers.



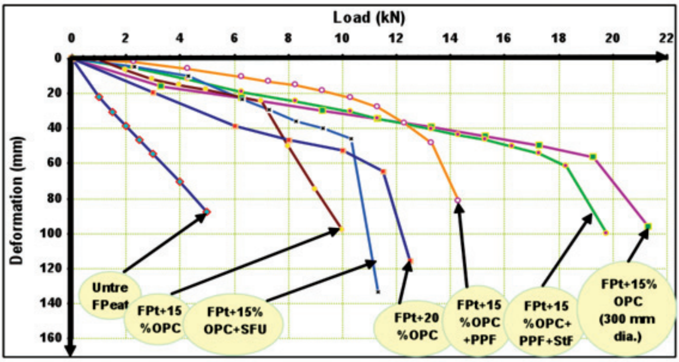
Photograph of the test tank
(Diameter = 820mm, Height = 1600 mm)

Line diagram

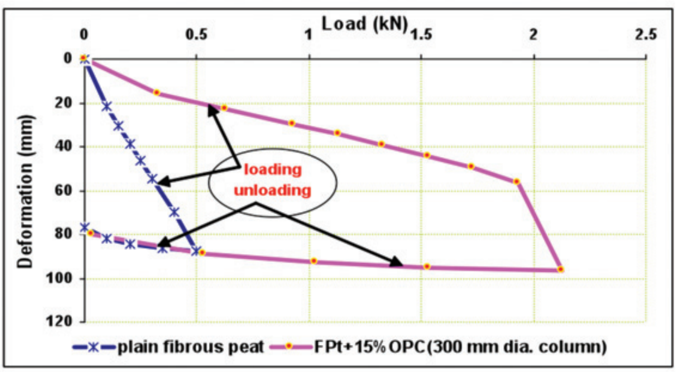
Instrumentation
1.Pressure unit, 2. Load cell, 3. Display unit, 4. Load jack,
5. Pore-pressure gauge, m6. LVDT, 7. Horizontal strain gauge, 8. Pressure cell.

Figure 24 Test tank

Precast cement column, 1.0 m long and 200 mm and 300 mm diameter were made of peat mixed with OPC and additives (polypropylene fiber, silica fume, polypropylene and steel fibers). The cement column was again shown to be beneficial in improving the load carrying capacity of peat (Figure 25).



(a) Load-deformation of untreated peat and peat treated
with precast cement columns



(b) loading-unloading

Figure 25 Load carrying capacity of fibrous peat with and without precast cement columns

A finite element model of precast cement column was carried out using a finite element software, Plaxis. An axisymmetric analysis was carried out using the hardening soil model for untreated peat and Mohr-Coulomb soil model for the cement stabilized columns. The parameters required for untreated peat and stabilized cement columns were unit weight (γ), Poisson ratio (ν), cohesion (c), friction angle (φ) and dilatancy angle (ψ). In addition, tangent oedometer stiffness (E_{oed}) was required for untreated peat and Young's modulus (E) for stabilized cement columns.

Drainage was permitted from the top. The typical finite element mesh consisted of 2001 nodes and 240 fifteen-node triangular elements. No interface elements had been used at the interface between the stabilized cement column and peat, as no significant shear is possible. The loads applied to the samples were 50, 100, 200 and 300 kPa. Each load in a stage was maintained for one day and then the next load was applied.

The parameters obtained from Rowe cell and triaxial tests were used to simulate the load carrying capacity of peat with full size cement stabilized columns. Analysis had been carried out for columns of 1.0 m diameter and 5.0 m long, arranged in a triangular pattern using unit cell concept at a spacing of 3d (d is the diameter). For the simulation of the ultimate bearing capacity of peat with prototype cement stabilized columns, the typical model consisted of 8589 nodes and 1050 fifteen-noded triangular elements.

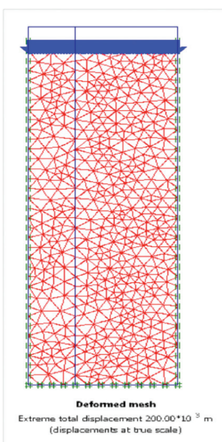


Figure 26 Deformed mesh of full size cement stabilized peat column

Figure 26 shows the typical deformed mesh. The displacement of 4.69 mm was very close to the actual recorded displacement of 4.76 mm at 50 kPa and with 10% cement. Similarly, very close agreements were observed between the actual and observed results from Plaxis for all the cases.

Figure 27 shows the result of simulated ultimate bearing capacity of the cement treated column in peat. The ultimate bearing capacity for all the cases increased with an increase in the cement content. The load at failure of sapric peat was apparently higher than that of hemic and fibrous peat. The ultimate load at failure of untreated fibrous peat was 44.17 kN and it increased to 78.04 kN for the column with 50% cement. Similarly, the loads at failure for untreated hemic and sapric peat were 49.63 and 51.33 kN respectively and increased to 84.83 and 96.14 kN for hemic and sapric peat respectively, with 50% cement.

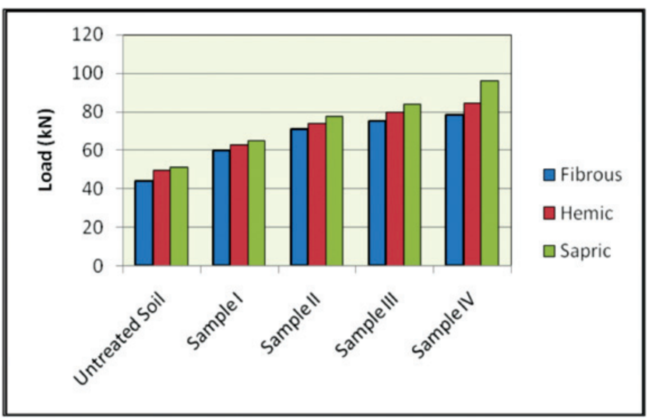


Figure 27 Ultimate load at failure of peat column

Stabilization by Injection and Vacuum

Cement columns made of ordinary Portland cement and additives such as polypropylene fibers and silica fumes have been shown to be beneficial in improving both the shear strength and the compressibility of peat. This technique involves pre-casting the column before putting it inside a pre-bored hole in the peat. Such techniques are however thought to have some limitations, such that it cannot really be used to improve very deep peat deposits. The technique may also be cumbersome and is believed to be feasible only for large scale projects, except for areas which only require limited treatment or in situations where specialized machinery are not available.

One alternative to overcome these limitations is to inject cement in the form of slurry into the peat. This technique can use conventional injector machines available for injection into other soft soils such as clay.

Grouting is generally used to fill voids in the ground (fissures and porous structures) with the aim to increase resistance against deformation, to supply cohesion, shear-strength and uniaxial compressive strength or finally (even more frequently) to reduce conductivity and interconnected porosity in an aquifer (Moseley &Kirsch, 2004). Figure 28 shows the sketch of grouting types for ground engineering.

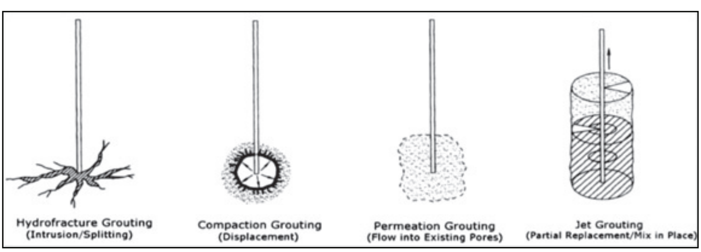


Figure 28 Types of grouting for grouting engineering (CIRIA, 2000)

The Deep Mixing Method (DMM) is today accepted world-wide as a soil improvement method and is based on mixing binders, such as cement, lime, fly ash and other additives, with the soil to form columns of a hardening material, since pozzolanic reactions are developed between the binder and the soil grains. The main advantage of this method is long term increase in strength especially for some of the binders used. Pozzolanic reaction can continue for months or even years after mixing, resulting in increase in the strength of cement stabilized soil with increase in curing time. As for all materials, the properties of stabilized soil depend on its microstructure.

DMM coupled with vacuum technique appears to be the solution for deep stabilization of peat. To do this, a laboratory scale DMM injector (injection-vacuum apparatus) was specially designed and developed as shown in Figure 29. In this study, the effect of cement and varieties of chemical grouts on peat properties in DMM was investigated. The injection-vacuum apparatus consisted of injection pump, vacuum pump and two kinds of auger for drilling vacuum and injection wells.

Stabilization of peat by injection-vacuum method is believed to have several up sides in comparison to other methods, such as:

1. The permeability of peat is not high and the application of vacuum would help speed up the stabilization process.
2. The low strength and loose matrix of peat means that usage of high power injection pump for stabilizing is not possible. In our method we can apply low power injection pumps.
3. In comparison with the conventional vacuum method and preloading, this method is believed to be more cost-effective.

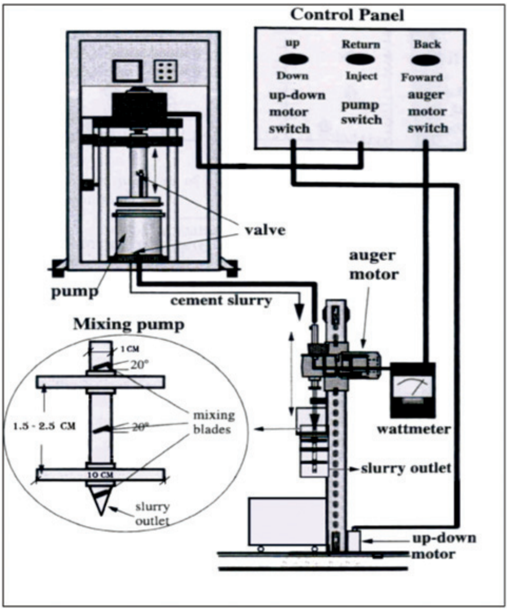


Figure 29 Schematic diagram of injection instruments in the lab

Figure 30 illustrates the schematic diagram of the injection-vacuum model.

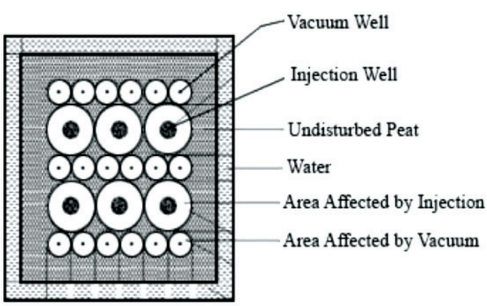


Figure 30 Injection-vacuum model

Electro-osmosising peat

In 2007 we began a study to look at the role of organic matter on electro-osmotic properties of peat; and to investigate the behavior of electro-osmotic flow in peat.

Engineering applications of electro-osmosis for stabilization and drainage of mineral (crystalline) soils have been used as far back as in the 1930s. Later, this technique was extended to improve stability of excavations, stabilization of fine grained soils, removal of metallic objects from soils, decreasing pile penetration resistance, stabilization of slopes and embankments, as well as strengthening sub grades and sub-bases under the pavement. Electro-osmosis is defined as a fluid movement with respect to a solid wall as a result of an applied electric potential gradient (direct DC current). In other words, electro-osmosis involves water transport through a continuous soil particle network, where the movement is primarily generated in the diffused double layer or moisture film where the cations dominate. When the direct electric potential is applied to a clay-water system, the surface or particle is fixed, whereas the mobile diffused layer moves carrying the solution with it.

While the electro-osmotic properties and electro-osmosis applications to mineral or crystalline soils have been quite readily understood, the electro-osmosis properties of peat, the feasibility of applying electro-osmosis, and the consequences of applying electro-osmosis to this soil mass are new. Unlike mineral or crystalline soil, the variations in peat arise from the variety of plants whose residues contribute to peat formation and from the environmental conditions in which humification takes place.

Colloids are the most chemically active fractions of soils. They are very small, less than 2 μm in diameter and are mineral (clays) or organic (humus). Further, they could be crystalline (definite structure) or amorphous. Clay and humus are dynamic and very active in charge in comparison with sand and silt which are static. The source of ions, source of electro-negativity (cation exchange capacity - CEC), buffering capacity and chemical cement agents are chemical properties, whereas the large surface area per unit of mass, and the plasticity are physical properties imparted to soils by colloids, respectively. The type of soil colloids are: (i) crystalline silicate clays, (ii) non-crystalline silicate clays, (iii) iron or aluminum oxide, and (iv) organic material (humus).

Electrical charges are carried by the surface of soils colloids, and these surface charges are the main reason that soil have a series of surface properties. The amount of ions which are adsorbed on the surface of soil colloids can be determined by the quantity of surface charges. In addition, the surface charge properties of the soil can affect the migration of ions in soil, the dispersion, flocculation and swelling. Consequently, the surface charge properties have a key role in soil structure. The sources of charges on colloids are: (i) permanent or constant charges due to isomorphous substitutions (montmorillonite, illite, zeolite, etc), and (ii) variable or pH dependent charges due to broken edge, OH, and COOH groups (kaolinite, humus, and Al or Fe oxides).

Humus is present in peat, and there is a large quantity of acid groups in humus. The origin of variable negative charges is from dissociation of these acid groups. A large quantity of negative charge is carried by humus which is in the range of 200 to 500 cmol kg^{-1}. The origin of the negative charges is mainly from carboxyl groups. Hydroxyl groups, including phenolic hydroxyl, quinonic hydroxyl, and enolic hydroxyl groups, can also produce negative charges. The role of phenolic hydroxyl groups under alkaline conditions is considerable. In soil humus, carboxyl groups and hydroxyl groups account for around 50 and 30% of the total functional groups, respectively. Furthermore, at high alkaline conditions, amino groups can also produce negative charge on the soil surface.

Several theories have been proposed for electro-osmosis, including the Helmholtz-Smoluchowski theory, Schmid theory, Spiegler friction model, Bucchingham ϖ theory and ion hydration theory. The Helmholtz-Smoluchowski theory is one of the earliest and still most widely used model to describe EK (electro-kinetic) processes. The Helmholtz-Smoluchowski theory assumes that the pore radii are relatively large compared to the thickness of the diffused double layer and all of the mobile ions are concentrated near the soil-water interface. Based on the Hemholtz-Smoluchowski, the zeta potential (ζ) and the charge distribution in the fluid adjacent to the soil surface play key roles in determining the electro-osmotic flow. The ζ is the electric potential developed at solid-liquid interface in response to movement of colloidal particles; i.e., ζ is the electrical potential at the junction between the fixed and mobile parts of the electrical double layer. The value of ζ is less than the surface potential of the particle and represents the value at the slip plane, which is located at a small unknown distance from the colloidal surface (Hunter, 1981).

The magnitude and sign of the ζ are dependent on the interfacial chemistry of both the liquid and solid phase. This potential is also influenced by ion exchange capacity, size of ion radius, and the thickness of the double layer.

Negative ζ causes electro-osmosis to occur from anode to cathode, while positive surface charge causes electro-osmosis to occur from cathode to anode. A number of investigators have studied variation of ζ of clays (for example, amorphous iron, gibbsite, and kaolinite) as a function of pH. There is however no study done on the variation of ζ in organic soils or peat. There is general agreement that ζ reduces in magnitude as acidity increases.

The electro-osmotic flow can virtually be eliminated at ζ equal to zero.

Peat or peaty soils have noticeable qualities to provide a suitable environment for utilization of electro-osmotic technique, i.e. (i) the saturated mass gives a good sense of the purpose, (ii) the high CEC and high specific surface area increase the presence of cations for water momentum in an electro-osmotic phenomena, (iii) the net negative surface charge may cause electro-osmosis to occur from anode to cathode, (iv) the decomposition processes enhance the surface charge resulting in better conditions for electro-kinetic experiments, and (v) the high resistivity may enable low energy expenditure in the processing. This predictive expectation has however to be proven by conducting laboratory electro-osmosis experiments.

Electro-osmotic Properties of Peat

Peat samples were prepared according to BS 1377-1 (1990) to evaluate the correlations between organic content (BS 1377-3: 1990), water content, liquid limit (BS 1377-2 1990), specific surface area (BET technique), ζ (ASTM D 4187), pH (BS 1377-3:1990) and cation exchange capacity (CEC).

For each sample, a solution of 0.1 g/L of the pretreated soil in 0.0001 M NaCl was prepared. The samples were shaken overnight at room temperature before measurements were taken. The room temperature was 21± 2.5ºC during the experiments. The ζ was measured with a zeta-meter as a function of pH values ranging from 1.91 to 11.5. Each sample was placed in an electrophoresis cell and then the electric field was activated. This field causes the particles to move with a velocity that is proportional to their zeta potential, and the direction indicates whether their charge is positive or negative.

All measurements were made in 0.0001 M NaCl solutions and pH adjustments were made using dilute HCl or NaOH solutions.

Zeta Meter Apparatus

After each measurement, the pH of the solution was measured again. If changes occurred in the pH solution, the last measured pH was recorded as the pH of the solution. The ζ of at least five particles for each sample was determined and their average taken. All solutions made use of deionized water.

The natural water content, liquid limit (LL), CEC, and specific surface area of the peat where found to increase with increase in organic matter (organic content), as shown in Figure 31.

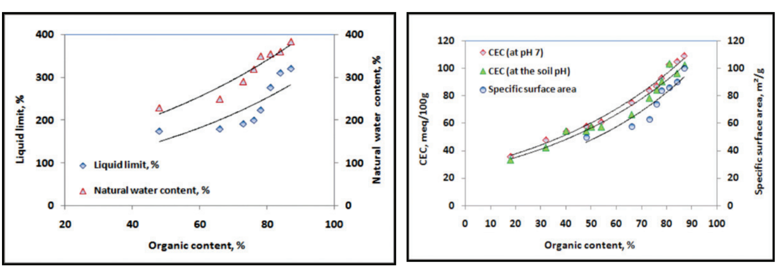


Figure 31 Organic content – liquid limit – natural water content and CEC – specific surface area of peat

Soil with high CEC has a high specific surface area, which provides the presence of more ions. Since a very wet soil mass and presence of hydrated ions are suitable environments for applying electrical potential across the mass, peat is a good candidate for the electro-osmosis technique.

Figure 32 shows the relationship between peat zeta potential (ζ) of peaty soils with soil pH. The variations in ζ with pH are probably related to the nature of the electrical energy field in peaty soils. The natural pHs of the peaty soils were 5.5 to 6.4. The sign of the natural ζ in peaty soils was negative. The charge was affected by pH. As the pH went up, net negative charge was produced. As the pH dropped, there was less and less negative charge. The ζ behavior of peaty soils in the presence of the NaOH was related to dissociation of H^+. The negative charge of humus is generally believed to be due to the dissociation of H^+ from carboxylic and phenolic functional groups. All the charges on the humus are strongly pH-dependent, with humic substances behaving like polyprotonated weak acids. Humic substances are a series of relatively high molecular weight, brown-to-black- colored substances formed by secondary synthesis reactions. Thus, the relationship between organic contents and zeta potential are not only under influence of organic contents, but also under influence of mineral portions and degree of humification.

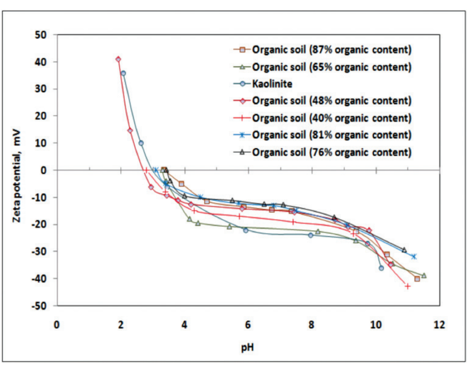


Figure 32 Zeta potential – pH relationship

The ζ of peat was found to increase with increase in degree of humification (more decomposition) as shown in Figure 33. Decomposition involves the loss of organic matter either in gas or in solution forms, through the disappearance of physical structure and change in chemical state. Breakdown of plant remains are brought about by the action of soil microflora, bacteria and fungi, which are responsible for aerobic decay. Decomposition tends to be most active in neutral to slightly alkaline conditions. The more acidic the peat, the better the plant remains are preserved. The degree of humification varies throughout peat since some plants and certain parts of the plants are more resistant than others. The negative charge of humus is generally believed to be due to the dissociation of H+ from functional groups. Since the charge in organic soils is strongly pH dependent, thus humified peat has a higher ζ with a negative mathematical sign and more charge than un-decomposed peat. The ζ is dependent mainly on organic content, degree of humification and the mineral fraction of the peaty soil.

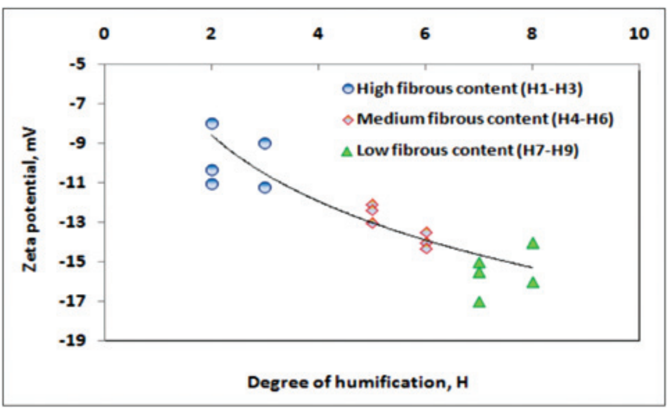


Figure 33 Zeta potential and peat degree humification

It is noteworthy that with increase in degree of decomposition, the pH at iso-electric point decreased as shown in Figure 34. The pH range at iso-electric points of the soils with high fibrous content was higher than the pH range at iso-electric points of the soils with low fibrous content. The soil surface charge drops to zero at the iso-electric point and the electro-osmotic flow can be eliminated. Since the pH at iso-electric point of the humified peaty soils was lower than in fibrous peat, thus, the elimination of the flow in humified organic soils may occur at a lower pH. The natural pH of humified organic soils was higher than that for fibrous peat, and the pH at the iso-electric point of the humified peat was lower, thus, in humified peat, the flow could be more consistent than in fibrous peat.

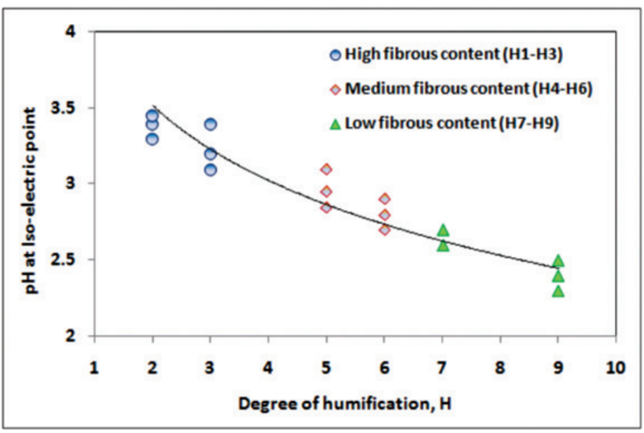


Figure 34 pH at iso-electric point – von Post degree of peat humification

Electro-osmotic Phenomena in Peat

It had been demonstrated that peaty soil or peat has all the characteristics required to provide a suitable environment for electro-osmosis. Our next task was to prove this experimentally.

However there was no standard equipment available for this purpose. As such we devised our own purpose-designed and built equipment which we named the '*electro-migration test system*'.

The test apparatus consisted of an acrylic unit with a central cylinder of 150 mm in length and 169 mm in internal diameter, as shown in Figure 35. The volume of both the cathode and the anode compartments was 2243 mL. Inert porous discs and filter papers were placed between the electrodes and specimen. Titanium disks were used as the electrodes and placed at each electrolytic compartment right behind the membranes. The electrodes were connected to a power supply. An oscilloscope and digital multimeter allowed the signal voltages and current to be viewed, respectively. Each electrolytic compartment was connected to a Mariotte bottle.

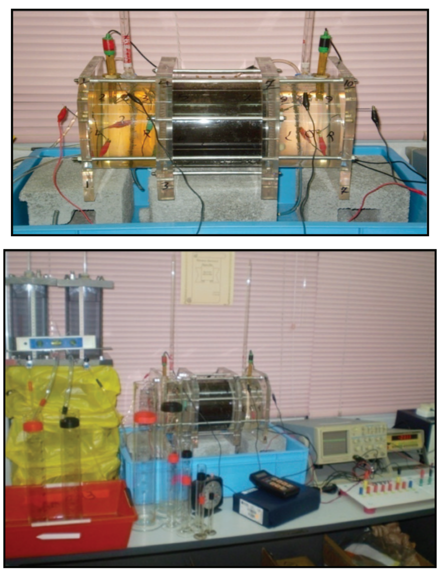
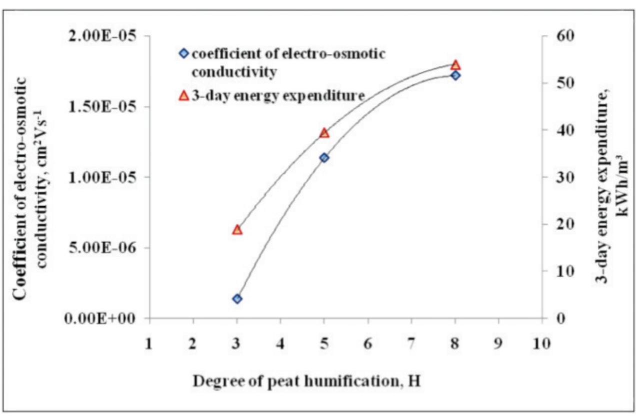
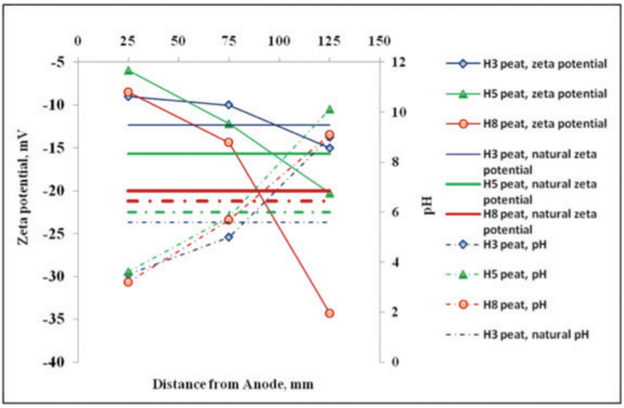


Figure 35 Electro-migration test system (awarded a 'gold medal' during the Research Invention Exhibition (PRPi09) held at Universiti Putra Malaysia on July 28, 2009)

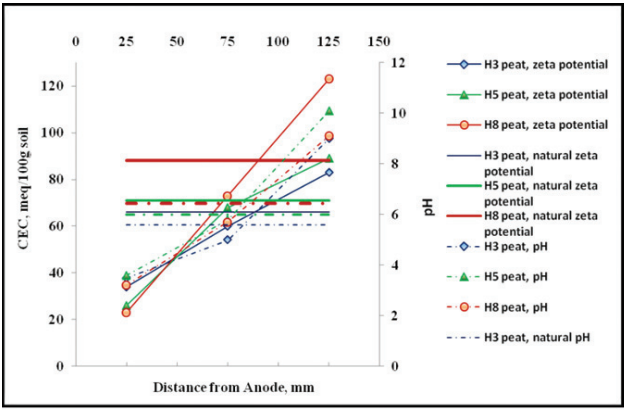
Undisturbed peat samples were gently inserted into the cell and fixed between the end porous discs and filter papers. The electrolytic compartments and Mariotte bottle were then filled with distilled water and the specimen was allowed to equilibrate in the test apparatus for 24 hours. The Mariotte bottle maintained a constant water level across the specimen, thereby preventing development of any external hydraulic gradient across the specimen. A constant electrical potential of 40 V was applied across the specimen. Peat samples were treated for a 3-day period. The effluent was collected to calculate the coefficient of electro-osmotic conductivity. Immediately after the test, the cell was drained of the remaining water in the electrolytic compartments and Mariotte bottles. The soil chamber was then detached from the electrolytic compartments and the soil sample extruded from the peat chamber and sliced into 3 sections. Each section was examined for basic properties including ζ and CEC and soil pH. The ζ of each treated sample was measured at the center point along the sample. The results obtained are shown in Figure 36.



(a) Coefficient of electro-osmotic conductivity-degree of humification-energy expenditure



(b) Zeta potential – pH



(c) CEC - pH

Figure 36 Electro-osmotic phenomena in peat

The coefficient of electro-osmotic conductivity and the 3-day energy expenditure were found to increase with increase in the degree of peat humification. The H8 peat had the highest electro-osmotic conductivity and energy expenditure. Peat had a net negative charge and the direction of electro-osmotic flow was from anode to cathode. The 3-day ζ and the 3-day CEC increased from a minimum value at the anode to a maximum value at the cathode. The CEC of the H3, H5 and H8 peat varied from 34 meq/100g soil at pH 3.5 to 83 meq/100g soil at pH 9, 26 meq/100g soil at pH 3.6 to 89 meq/100g soil at pH 10.1 and 23 meq/100g soil at pH 3.2 to 123 meq/100g soil at pH 9.1. The study showed that all charges on the soil surfaces were strongly pH-dependent. The ζ and CEC changes of the H3 and H5 peat were not of such intensities as that observed in the H8 peat, meaning the H8 had the highest sensitivity to pH gradients. The very highly decomposed (H8) peat had significant differences in electro-osmotic properties in comparison with the H5 and H3 peat.

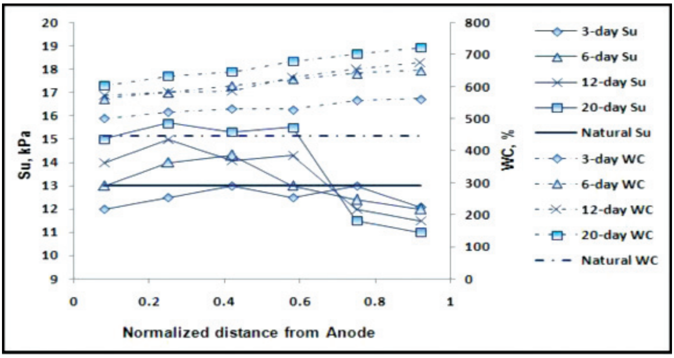
A good understanding of the humus as the most chemically active fraction of the peat colloids could make clear the underlying reasons for the significant differences. The specific surface area and the portion of the sample passing the No. 100 sieve could be the indicators of the quantity of the humus. Since the H8 peat had a larger surface area per unit mass (i.e. smaller particles) and had a higher portion of the sample passing the No. 100 sieve, thus, the quantity of the humus portion in H8 peat was higher than that in H3 and H5 peat. Since the humus is dynamic and very active in charge (Stevenson, 1994), the H8 peat had a higher electro-osmotic conductivity. The ζ and CEC of the H8 peat were higher than for H3 and H5, which resulted in the current changes and consequently increased the energy expenditure value. Despite the quantities of the colloidal fractions (i.e. humus) in the H8 peat, the quality of the surface charge could increase with an increase in the degree of peat humification. Hence, there was a direct relation between the degree of peat humification and quantity and quality of the colloidal fractions of the peat.

The water ionization caused H^+ ions to be released near the anode, which resulted in the decreased pH values observed. The ζ was then shifted to less negative values and consequently the CEC values decreased, while the water decomposition in the vicinity of the cathode had a contrary effect.

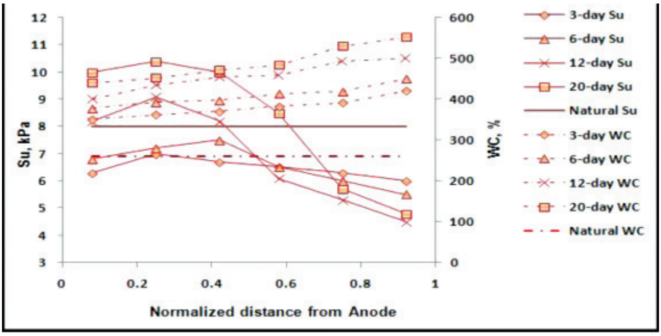
The study was important in that it confirmed that the electro-osmotic properties of the peat were hinged to the ion exchange capacity, specific surface areas, and the degree of peat humification.

Effect of Electro-osmosis on Shear Strength of Peat

Figure 37 shows the effect of electro-osmosis on peat undrained shear strength, S_u. The S_u was found to increase at the cathode because of the acidic conditions in this vicinity, while alkaline conditions at the cathode apparently had an opposite effect. Amorphous peat was apparently more sensitive to the EK environment compared with the fibric peat because of the larger quantities of the colloids and quality of the charges. The initially lowered values of the S_u in the vicinity of the anode could be attributed to constant peat water supply. The S_u decreased in the vicinity of the cathode by reason of highly negative ζ and dispersion of the peat particles. However, with a larger quantity of mineral portion of the peat soils in tropical areas, the ions migration mechanisms due to the CEC increase and precipitation of aluminum hydroxide at the cathode are expected to improve the S_u. The peat water flow also caused soil erosion in the vicinity of the cathode, through electrophoresis, affecting peat particles packing and thereby contributing towards the noticeably decreased S_u values observed. The fibrous peat particles packing were more resistant to the peat water flow than the amorphous peat because the fibers acted as reinforcement. The colloidal fractions of the amorphous peat were higher than the fibrous peat and furthermore they were more active in charge in comparison with the fibric peat. As a result, the sensitivity of the amorphous peat to the EK environment was apparently higher than that in the fibric peat. The result was important in that it confirmed the relationship between the peat's degree of humification and the sensitivity of peat soils to the EK environment.



(a) Shear strength of fibrous (fibric) peat



(b) Shear strength of amorphous peat

Figure 37 Effect of electro-osmosis on shear strength of peat

Acknowledgement

The works described in this section (4.2) were funded through the following research grants:

1. Deformation and shear strength characteristics and index properties of peat and organic soils with variable fiber and organic content. Intensification of Research Priority Areas, Ministry of Science, Technology and Environment. Ref. 54225
2. Deep stabilization methods for improvement of peaty soils. Intensification of Research Priority Areas, Ministry of Science, Technology and Environment. Ref. 54407
3. Tropical peaty soil stabilization using cement columns. Research University Grant Scheme. Ref. No. 91151
4. Cement columns for peaty soil stabilization. Science Fund. Ministry of Science, Technology and Innovation. Ref. No. 5450411
5. Electrokinetic improvement of soft clay & organic soils and modeling of their mechanical properties. Research University Grant Scheme. Ref. No. 91152

Note: Further details on materials summarized above can be found in the following publications:

1. Stabilizing Peat Soil with Cement and Silica Fume, *Geotechnical Engineering, Proceedings ICE*. (to be published)
2. Cement and Silica Treated Columns to Improve Peat Ground. *Ground Improvement. Proceedings ICE*. (to be published)
3. Use of Cement, Polypropylene Fibers, and Optimum Moisture Content Values to Strengthen Peat Soil. *Korean Journal of Civil Engineering* (to be published).
4. Physicochemical Sensitivities of Tropical Peat to Electrokinetic Environment. *Geosciences Journal* (to be published)
5. (2010). Effect of Polypropylene Fibers on the California Bearing Ratio of Air Cured Stabilized Tropical Peat Soil. *American Journal of Engineering & Applied Science* 3(1) 817-822

6. (2009). Effect of Fly Ash on the Strength Values of Air Cured Stabilized Tropical Peat with Cement. *Electronic Journal of Geotechnical Engineering*. 14N
7. (2009). Precast Stabilized Peat Columns to Reinforce Peat Soil Deposit. *Electronic Journal of Geotechnical Engineering*. 14B
8. (2009). Experimental Investigations on the Geomechanical Properties of Tropical Organic Soils and Peat. *American Journal of Engineering and Applied Science*. 2(1) 184-188
9. (2009). Compressibility Behavior of Fibrous Peat Reinforced with Cement Columns. *Geotechnical and Geological Engineering*. Springer. 27(5) 619-629
10. (2009). Compressibility Characteristics of Fibrous Tropical Peat Reinforced with Cement Column. *Electronic Journal of Geotechnical Engineering*. Volume 14C
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13. (2009). Electrical Resistivity of Tropical Peat. *Electronic Journal of Geotechnical Engineering* 14J
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Eliminating the Slide out of Landslides – A Tall Order?

Literally, landslides can be defined as massive mass of soil and rock debris that move downhill because of the action of gravity. The massive mass of material involved, and lightning speed at which it occurs, make it a possible disaster, because of the extensive damage it can cause to properties and even the loss of lives. In fact landslides can be classified as calamities that terrify man, together with other natural calamities like volcano eruptions, earthquakes, typhoons and tsunamis. In Malaysia around 30 major landslides have been reported since 1975, with loss of more than 200 lives and billions of Ringgit in damages to properties.

Landslide is a subject of interest not only to the geomorphologist but also to geotechnical engineers. A geomorphologist will view landslides as a process in the evolution of earth surface formation whereas, a geotechnical engineer will look at landslides as a form of slope instability, albeit, an extreme one that must be considered and prevented in an engineering design. This is perhaps where we should draw a difference. Natural evolution of hill-slopes may be of little interest to most of us, for example where landslides occur in undisturbed and unpopulated hills. However, landslides that are caused directly by human activity includes chopping up hill land for development, clearing hill slopes for agriculture, etc. Through our activities we will affect the geomorphology as well as drainage pattern of the hill slope, which may lead to its failure.

The 1990s can be considered as the beginning of a period when hill slope problems/failures became imminent in the country through a number of major, and in some cases catastrophic, landslides. The main reason for this was that the country was then experiencing rapid economic growth. Hill-site construction had increased tremendously, partly due to depleting flat land, and partly due to other influencing factors like beautiful scenery, fresh air, exclusiveness, etc. With this came major infrastructure developments.

What Causes Landslides?

We must appreciate that in engineering, there is a concept of short and long term problems or failures. Short term failures are failures that if they were to occur will occur during the construction period itself, for example, building a road embankment on soft clay ground. If there is to be a slope failure, it will happen as the embankment is being built, meaning the short term. With time, the ground under the embankment will compress and it will become stronger, although we may have other problems such as sinking of the road or embankment because of this compression. Chopping or cutting hill slopes is actually the reverse of the above. As we actually remove material from the slope, the ground is temporarily 'stronger', but with time, through action of water etc. the ground will become weaker. Therefore we call this a long term problem, meaning that if the slope is to fail, and a landslide was to occur, it will occur long after the initial construction is completed. It is therefore not surprising that landslides will occur, say, 10-15 or even more years after the construction has been completed.

Can Landslides Be Prevented?

In theory we can prevent if not minimize the risk of landslides. Theoretically we can use existing analytical tools to do this, but we need very careful and thorough planning and study. First we must do extensive site investigation to study the strength and geology of the site concerned in order to have meaningful data for our analysis. Then we must cut the slope at an angle at which it will remain stable, take necessary steps to prevent erosion like vegetating the slope and building retaining structures like retaining walls to improve the stability of the slope. Equally important, the slope must be provided with adequate drainage systems to ensure that the water does not ingress into the slope.

This is a short to medium term solution. In the long term, the slope must be continually maintained. We have to ensure that the drains are in working order. This is very important, for if water gets into the slope, it will erode the slope, and in the long run it will weaken the slope, and eventually lead to a landslide. This is perhaps where major weaknesses lie – long term maintenance, and who is actually responsible for this, the property owner, property developer or local authority?

First and foremost every party involved, i.e. the property owner, property developer and local authority must be aware of their individual specific roles in mitigating the risk of landslides. The property owner must be aware that any property built on or next to a hill slope is under some degree of risk from landslides. The Institution of Engineers Malaysia (IEM) for example, in their position paper published in year 2000, classified slopes of greater than 15m high, and with angles of greater than 30 degrees, in the high risk category. The owner must also be aware that landslides are a long term problem and disaster may strike years after the construction of the property.

Perhaps even more important, the property developer and local authority must be aware of their responsibilities in safeguarding the public from the hazard of landslides. Presently we do not seem to have a systematic regulatory measure to address safety problems related to hill-side developments in our country. The existing legislation and guidelines on slope failure mitigation are apparently inadequate to produce satisfactory solutions. Despite the series of landslide catastrophes resulting in more than 200 lives lost, we have yet to produce a solution to overcome this problem.

In 1999 IEM set up a task force to review several case histories of landslides in Malaysia, and summarized the causes as follows:

1. Design – inadequate investigation and lack of understanding of analysis and design.
2. Construction – lack of quality assurance and quality control by contractors.
3. Site supervision and maintenance – lack of proper site supervision during construction and lack of maintenance after construction.
4. Communication – lack of communication amongst the various parties involved in construction.

The task force also recommended the following measures to mitigate the risk of landslides in hill side developments:

1. To appoint qualified consultants to audit design of major developments in high risk areas.
2. To have full time professional engineers.
3. To have full time professional engineers to supervise construction.

Developers, contractors and supervisors should be made more accountable to the authorities for construction safety. IEM also proposes a specific agency or department to be formed to regulate and monitor hill side developments. This new agency would monitor hill-side developments and assist the local authorities to regulate and approve all hill-side developments. The Agency could engage or outsource, whenever necessary, a panel of consultants to assist and expedite implementation. For existing hill-side developments, the Agency should advise the local government to issue 'Dangerous Hill-Side Order' to owners of doubtful and unstable slopes so that proper remedial and maintenance works can be carried out to stabilize the slopes and to prevent loss of lives and properties.

In 2006, a National Slope Master Plan Study was commissioned by the Government of Malaysia. UPM was part of the team carrying out the study. The overall objective of the National Slope Master Plan Study was to provide a comprehensive and effective national policy, strategy and action plan to reduce losses from landslides. The Master Plan included the following aspects of slope engineering and management: 1. Policies and Institutional Framework, 2. Hazard Mapping and Assessments, 3. Early Warning and Real-Time Monitoring System, 4. Loss Assessment, 5. Information Collection, Interpretation, Dissemination and Archiving, 6. Training, 7. Public Awareness and Education, 8. Loss Reduction Measures, 9. Emergency Preparedness, Response and Recovery, as well as 10. Research and Development. The Master Plan was completed in 2008 but is yet to be implemented at the time of this writing.

As landslides are a long term problem, continued and proper maintenance of the slope and slope structure with the likes of retaining walls and slope drains is also very important. It is well known that failure to maintain the slope, particularly after periods of heavy rain and prolonged erosion, may propagate and trigger landslides. Figure 38 demonstrates the influence of rain on incidences of landslide.

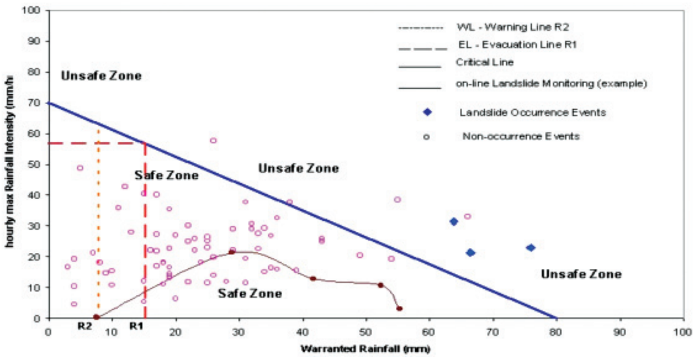


Figure 38 Effect of rainfall on landslides (Raj, 2004)

Globally it is recognized that rainfall is the main trigger for landslides (Figure 39).

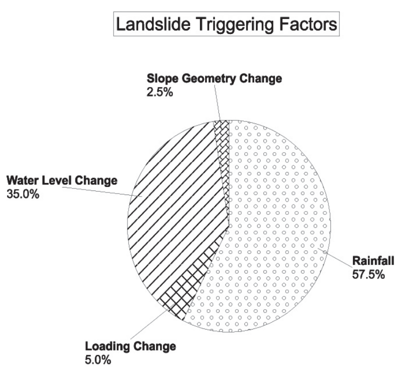


Figure 39 Landslide triggering factors based on worldwide literature review (PWD, 2007)

However, awareness alone is not sufficient. All slope infrastructures must be properly maintained. Personnel involved must be properly trained; they need a set of standards of good practice in slope maintenance to follow. A good guideline from

Geotechnical Engineering Office of Hong Kong, like 'Geoguide 5 – Guide to Slope Maintenance' (1995) for engineers and 'Layman's Guide to Slope Maintenance' which is suitable for the layman can be referred to. The Guide recommends maintenance inspections to be sub-divided into three categories, namely:

1. Routine Maintenance Inspections (RMI) - which can be carried out adequately by any responsible lay person.
2. Engineer Inspections for Maintenance (EIM) - which should be carried out by a professionally qualified and experienced engineer.
3. Regular Monitoring of Special Measures (RMSM) – which should be carried out by a firm with special expertise in the particular type of monitoring service required.

Such monitoring is only necessary where the long term stability of the slope or slope structures such as retaining walls rely on specific measures which are liable to become less effective or deteriorate with time. This measure is seldom carried out in Malaysia. For Malaysia, which has at least two monsoon seasons, the RMI by 'layman' should be carried out a minimum of twice a year for slopes with negligible or low risk-to-life. For slopes with high risk-to-life, more frequent RMI is required (once a month frequency). In addition, it is good practice to inspect all the drainage channels to clear any blockage by siltation or vegetation growth and repair all cracked drains before the monsoon. Inspection should also be carried out after every heavy rainstorm. The EMI should be taken to prevent slope failure when the RMI by 'layman' observes something unusual or abnormal, such as occurrence of cracks, settling ground, bulging or distorting of wall or settlement of the crest platform. The Guide recommends as an absolute minimum, an EIM be conducted once every five years or more as requested by those who carry out the RMI. More frequent inspections may be desirable for slopes and retaining walls in the high risk-to-life category.

It is also recognized that climate change or global warming is one of the most serious environmental threats of the 21st century. Extreme or severe weather is simply very bad weather or weather on larger, more serious and devastating scales which create natural disasters. In Malaysia, forecasts of climate change have been made using 14 GCM's (Global Climate Models) which show that Malaysia could experience temperature increases ranging from +0.7 to 2.6 °C and precipitation variations ranging from -30 to 30%. In other words, Malaysia too is expected to experience extreme weather variations such as prolonged storms and droughts, in the future. Figure 40 shows the simulated annual rainfall for Peninsular Malaysia. Some 'wet' years are expected in some areas in the years ahead. More rain can only mean more problems for more Malaysian slopes unless mitigation measures are taken.

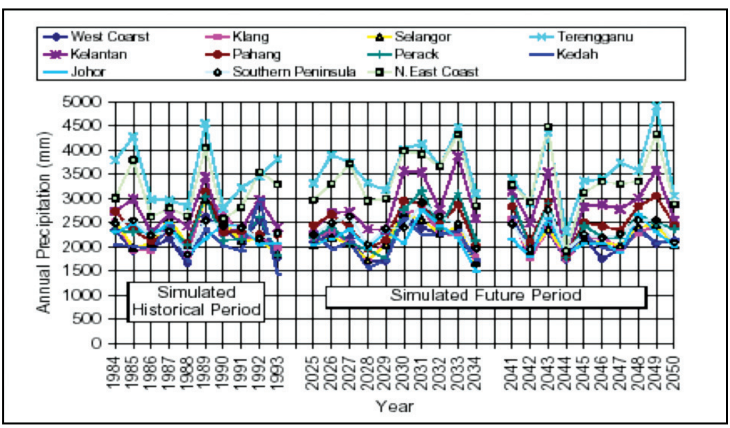


Figure 40 Simulated annual rainfall in sub regions of Peninsular Malaysia during historical (1984-1993) and future (2025-2034 and 2041-2050) periods (Zakaria *et al,* 2007)

There are many factors which have been identified as causing and influencing climate change. These factors can either be global, national or local. Global warming and open burning (haze) are some good examples of global factors that are trans-border in nature by definition whilst industrialization, clearing of land for agriculture and encroachment of fragile ecosystems are examples of national and local causal factors.

(The above write-up was based on articles with similar titles – 'Eliminating the slide out of landslide' which was published in the Sarawak Tribune as a 2 part series on the 9th and 16th of February 2003)

Work Carried Out at UPM

While issues like policy making is not something we have looked at, we have tried to contribute in other areas like 'engineering solutions' to mitigate the landslide problem. Our work focuses on 3 areas namely:

1. Slope assessment system (SAS) for predicting landslides, for the purpose of monitoring, management and prioritizing maintenance (2003-2006)
2. Use of bio-engineering for stabilizing slopes (since 2007), and
3. Low cost repair for failed slopes using waste materials (scrap car tires) (since 2003).

Slope Assessment System for Predicting Landslide

The probability of occurrence and likely severity of landslides can be predicted by assessing related significant slope parameters. Statistical method is one of the ways to estimate the probability of occurrence and likely severity of the landslides. Examples can be found in the work of Carrara et al., 1995, Guzzetti et al., 1999, and Péloquin & Gwyn, 2000. The technique includes developing a conceptual model consisting of mapping the landslides, mapping a set of environmental factors which are supposed to be directly or indirectly correlated with slope instability, estimating the relationships between these factors and the instability phenomena, and classifying the land surface into domains of different landslide susceptibility degrees, on the basis of the detected relationships. Results of these assessments can be presented in the form of a landslide hazard map, which is useful in planning development, as well as in slope maintenance and management. They can also be combined with landslide consequences analysis to produce a landslide risk map that can be used in prioritizing maintenance works, and in emergency and rescue preparedness.

In our work, we developed a slope assessment system (SAS) for cut slopes underlain by granitic formation using discriminant analysis.

Four major trunk roads traverse through the Main Range granite formation of West Malaysia, namely the East-West highway (Gerik-Jeli), the Tapah – Cameron Highland road, the Kuala Kubu Baru – Gap road and the Kuala Lumpur – Bentong Old road, as shown in Figure 41, were chosen as the field study sites. An additional fifth site, the Gunung Raya road on the Island of Langkawi, Malaysia, was also selected. These mountainous roads had experienced landslides in the past, which had caused disruption to traffic, injuries and loss of lives.

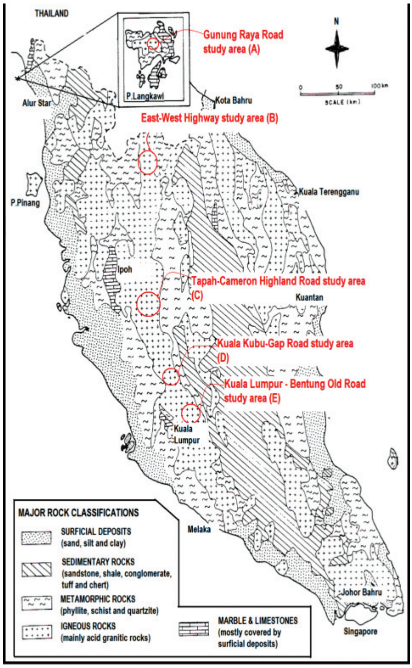
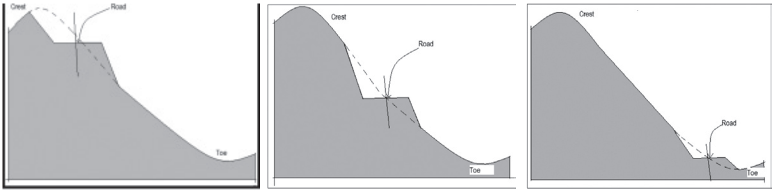


Figure 41 Locations of field sites and general geology of Peninsular Malaysia

The SAS was developed based on the principle drawn by Varnes (1984) that landslides in the future would be most likely to occur under the same geomorphic, geologic, and topographic conditions that had produced past landslides. Data from 139 cut slopes along Site A, B and C, were used in the development of the SAS. These data were divided into two groups i.e. 86 failed slopes and 53 not failed (without sign of failure) slopes, and were analyzed using stepwise discriminant analysis. Statistical analysis was used because there were ample slope inventories and landslides data collected over the past ten years. Furthermore, the statistical analysis could easily be conducted using available computer software to determine the significant slope parameters and their coefficients for a linear model which could be easily applied and verified by others. The data that was compiled for 10-years (1994-2004) included slope height, slope angle, soil type, weathering grades, erosion, drainage conditions, rainfall etc. These data were obtained from previous records as well as through site visits (walkthrough survey). Landslide occurrences were determined either from written historical records, differences seen in multi-date aerial photos, or difference between older sketches on the data collection forms and current site conditions. Data prior to the occurrence of the landslides were used as input for the SAS. Both past and recent landslides were summed up together. Past landslides refer to landslides that occurred before the initial data was collected and it was recorded or sketched in the initial data collection forms. Recent landslides refer to landslides that occurred after the initial data was collected.

From the available data, 25 physical parameters related to landslides were selected for every slope for use in the development of the SAS. These data, in form of continuous variables/parameters, were transformed into various classes and were used in the statistical analysis and regression equation for the computation of instability score (individual discriminant scores). The parameters of the failed and not failed slopes were analyzed using the stepwise discriminant analysis. Description, reasons behind its selection and distribution based on classes of the said 25 parameters used in the model development are described briefly below;

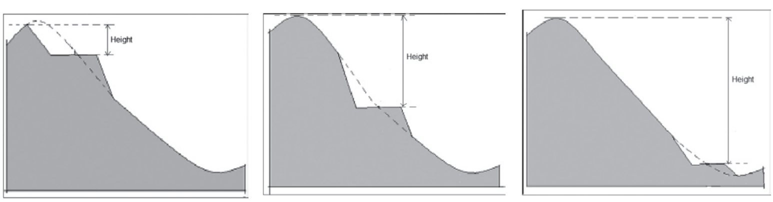
(1) *Slope feature location/position*: This parameter indicates the location of the assessed slope, either near to the crest, at mid-slope or near to the toe of the hill or mountain. The slope located near to the toe is expected to have a high probability to fail compared to a slope located near to the crest.



(a) (b) (c)

Slope feature location / position. (a) at the crest, (b) at the middle, and (c) at the toe

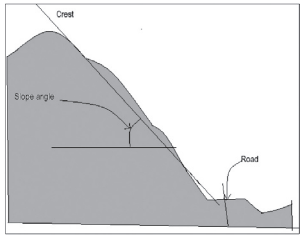
(2) *Height of slope*: This is the approximate vertical distance of slope in meters from the assessed point to the highest point on the slope. Based on common understanding of slope stability, slope height is one of the significant parameters influencing the stability of any slope. Slopes with higher elevation have a higher probability to fail compared to lower slopes.



(a) (b) (c)

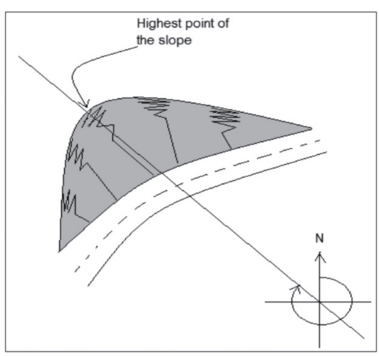
Slope height

(3) *Slope angle*: The slope angle is the average angle from the horizontal line to the line connecting the toe to the crest of the slope. This line is obtained from the cross section of the steepest part of the slope. Slopes with higher angles are expected to have a high tendency to fail compared to slopes with lower angles.



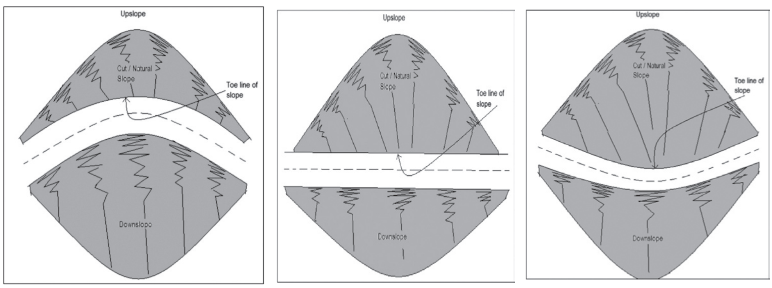
Slope angle

(4) *Feature aspect*: The feature aspect is the horizontal bearing of the slope to the north of the line from the slope to looking up to the highest point of the slope crest as illustrated in the diagram below. The aspect of a slope can influence landslide initiation. Moisture retention and vegetation is reflected by the slope aspect, which in turn may affect soil strength and susceptibility to landslides. The slope facing a certain direction where the wind pattern carries heavy rain is expected to have a high probability of landslide occurrence, for example in respect to Peninsular Malaysia, it is the slope facing North-East (facing the South China Sea).



Slope feature aspect

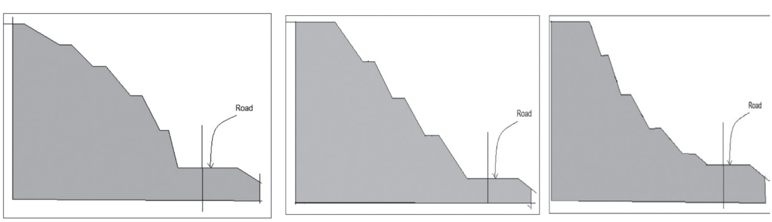
(5) *Plan profile*: The plan profile is the shape of the curvature of the slope toe line in plan, recorded as concave or straight or convex. It is expected that the slope with concave plan profile is more prone to landslides compared to a straight and convex profile.



(a) (b) (c)

Slope plan profile. (a) Concave, (b) Straight, and (c) Convex

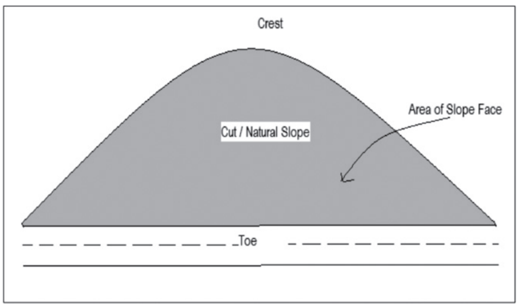
(6) *Cross profile shape*: The cross profile shape is a measure of the curvature of the slope profile obtained from cross-section of the steepest part of the slope, recorded as concave or straight or convex. It is expected that the slope with a concave cross profile shape is more prone to landslides as compared to straight and convex cross profile shapes.



(a) (b) (c)

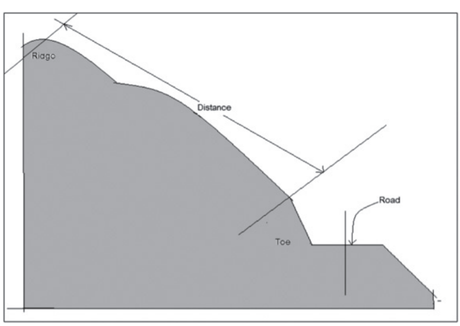
Cross profile shape (a) Convex, (b) Straight, and (c) Concave

(7) *Feature area*: The feature area is an approximate area of the slope face as shown below. It can be calculated based on the slope length, height, angle and their shapes. It is expected that the higher the slope face area the higher the probability for landslide to occur.



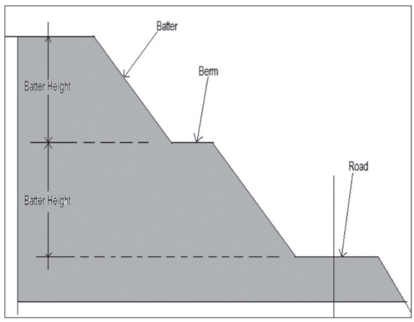
Slope feature area

(8) *Distance to ridge*: Distance to ridge is measured approximately from the highest point of cutting of the assessed slope to the ridge of the hill or mountain. The longer the distance from the highest point of cutting of the assessed slope to the ridge of the hill or mountain, the bigger the catchment areas of the slope, and thus the higher the probability for landslides to occur.



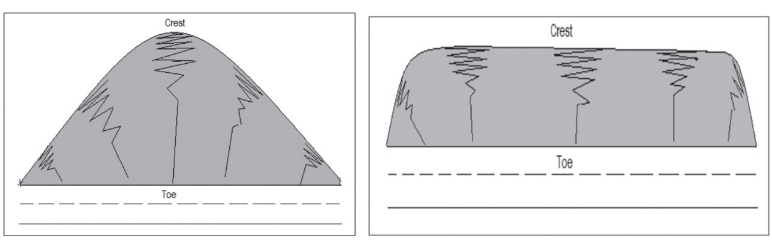
Distance to ridge

(9) *Batter/bench height*: Batter or bench height is measured vertically, and most of the cut slopes in Malaysia have a height of about five to six meters. For slopes without berms, we can assume it as a slope with one batter.

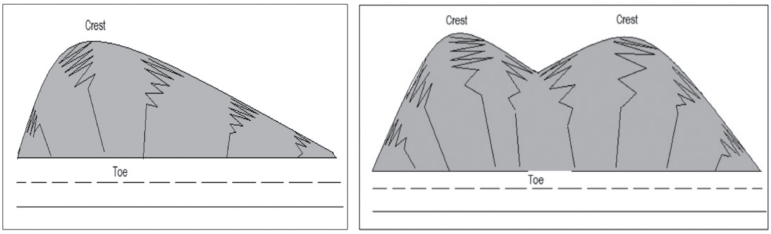


Batter/bench height

(10) *Slope shape*: The slope shape is the front elevation view of the slope, and can be recorded as simple, planar, asymmetrical or compound. The probability of landslides to occur on the slope with compound shapes is higher compared to the slope with a simple shape.



(a) (b)



Slope shape. (a) Simple, (b) Planar, (c) Asymmetrical, and (d) Compound

(11) *Main cover type*: This is recorded as the main or dominant type of slope cover, either artificial (gunite, shotcrete etc.), trees, shrubs or grass. It is expected that slopes covered with trees are more stable than those covered by grass. As reported by Dai and Lee (2002), extensive investigations have shown that vegetation cover, especially of a woody type with strong and large root systems, help to improve stability of slopes (Greenway, 1987). Vegetation provides both hydrological and mechanical effects that are generally beneficial to the stability of slopes.

(12) *Percentage of feature uncovered*: This is an approximate percentage of slope face that was exposed or uncovered, expressed as a percentage of the whole slope face area. The slope with a high percentage of features uncovered is exposed to erosion, thus it can trigger landslides.

(13) *Soil type*: The type of soil can be determined by squeezing it between fingers, and can be described as clayey, silty or sandy. Slope forming materials consisting of clayey types is more unstable as compared to silty and sandy types.

(14) *Soil strength*: The strength of soil used in this model development is based on standard definitions of strength for soil materials used in the BS5930.

(15) *Presence of rock exposure*: Any presence or absence of exposed rock or outcrops will be recorded. The presence of rock exposure is the sign of shallow formation of rock underneath the slope face. A rock slope is more stable than a soil slope.

(16) *Percentage of rock exposure*: This is an approximate percentage of rock exposure from the whole slope face area. The higher the percentage of rock exposure, the more stable the slope.

(17) *Weathering grade*: The weathering grade of the slope feature materials refers to the weathering profile drawn by Little (1969), as shown in Figure 42, and is divided into three classes; Class 1 – Grade I to II, Class 2 – Grade III to IV, and Class 3 – Grade V to VI. A slope with a lower weathering grade is more stable than a highly weathered slope.

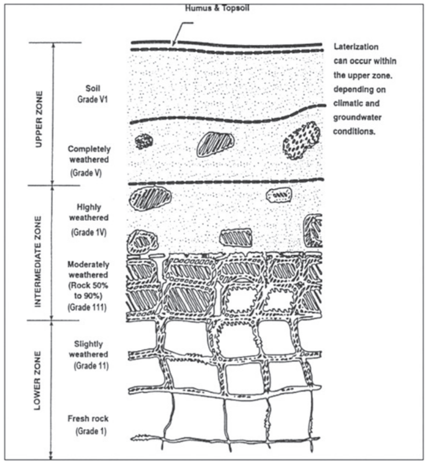
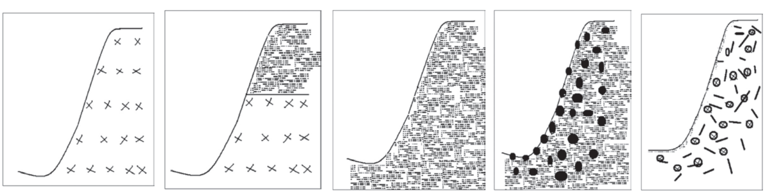


Figure 42 Typical weathering profile (Little, 1969)

(18) *Rock condition profile*: A rock condition profile represents overall or slope regolith forming materials. The slope formed by rocks of Grade III or less is more stable than the slope formed by colluvium.



(a) (b) (c) (d) (e)

Rock condition profile: (a) Majority of slope Grade III or less (Rock), (b) Slope consists partly of materials of Grade III or less and partly of materials greater or equal to Grade IV, (c) Slope consists predominantly of grade IV to Grade VI, (d) Slope consists of predominantly Grade IV to VI but with core-stone boulders and (e) Slope consists predominantly of colluvium

(19) *Bench drain*: Any presence or absence of bench or berm drain should be recorded. The slope constructed complete with bench drain is expected to be more stable than a slope without bench drain.

(20) *Horizontal drain*: Any presence or absence of horizontal drain should be recorded. The slope constructed complete with horizontal drain is expected to be more stable than a slope without horizontal drain.

(21, 22) *Roadside drain /toe drain*: Any presence or absence of roadside drains or toe drains should be recorded. Roadside drains or toe drains are important to channel out the water from the site. The slope constructed complete with toe drain is expected to be more stable than a slope without a toe drain.

(23) *Number of water courses within features*: The number of water courses within the slope features should be recorded. Water courses within the slope features such as natural surface runoff, seepage flow or any ponding area will contribute to slope instability. During rainy seasons, pore water pressure in the slope mass will be increased on slopes with water courses within it.

(24) *Presence of erosion*: Any sign of past and recent erosion should be noted. Erosion is an early indication of slope instability, and the probability that a landslide will occur in this area is high.

(25, 26) *Daily and monthly highest rainfall*: In Malaysia, rainfall is considered as triggering factors of landslide occurrences, and it should be considered in any slope assessment system. Furthermore, due to only one or two rain gauges covering each study area, statistically the effects of rainfall will be uniform throughout the study areas. Due to this fact rainfall is expected not to appear as a significant parameter in this assessment. The reason why this parameter is included in this study is to confirm this fact (Table 2).

Table 2 Parameters of slope feature used in the model development

No.	Parameters	Ranges (Classes)	No.	Parameters	Ranges (Classes)
1	Slope feature location / position	Near crest (1)			Artificial cover (4)
		Mid-slope (2)	12	% of feature uncovered	< 10 (1)
		Near toe (3)			10 to 30 (2)
2	Height of slope (m)	<10 (1)			> 30 (3)
		10 to 20 (2)	13	Soil type	Sandy (1)
		20 to 30 (3)			Silty (2)
		> 30 (4)			Clayey (3)
3	Slope angle (in degrees)	< 15 (1)	14	Soil strength	Hard (1)
		15 to 30 (2)			Very stiff (2)
		30 to 45 (3)			Stiff (3)
		45 to 60 (4)			Firm (4)
		60 to 75 (5)			Soft (5)
		> 75 (6)			Very soft (6)
4	Feature aspect in degrees	0 to 90 (1)	15	Presence of rock exposure	Yes (0)
		90 to 180 (2)			No (2)
		180 to 270 (3)	16	% rock exposure	0 to 25 (1)

No.	Parameters	Ranges (Classes)	No.	Parameters	Ranges (Classes)
		270 to 360 (4)			26 to 50 (2)
5	Plan profile	Convex (1)			51 to 75 (3)
		Concave (2)			76 to 100 (4)
		Straight (3)	17	Weathering grade	I to II (1)
6	Cross profile shape	Convex (1)			III to IV (2)
		Concave (2)			V to VI (3)
		Straight (3)	18	Rock condition profile	Grade III or less (1)
7	Feature area (m^2)	< 2,500 (1)			Grade III and Grade VI (2)
		5,000 to 7,500 (2)			Grade IV to Grade VI (3)
		7,500 to 10,000 (3)			Grade IV to Grade VI with corestone boulders (4)
		>10,000 (4)			Colluvium (5)
8	Distance to ridge (m)	< 50 (1)	19	Bench drain	Yes (0)
		50 – 149 (2)			No (2)
		150 – 249 (3)	20	Horizontal drain	Yes (0)
		> 250 (4)			No (2)
9	Batter/bench height (m)	< 5 (1)	21	Roadside drain/toe drain	Yes (0)
		5 – 9.9 (2)			No (2)
		10 – 14.9 (3)	22	Number of water courses within features	0 (0)
		15 – 19.9 (4)			1 (1)
		> 20 (5)			2 (2)
10	Slope shape	Simple (1)	23	Erosion	No (0)

No.	Parameters	Ranges (Classes)	No.	Parameters	Ranges (Classes)
		Planar (2)			Yes (2)
		Asymmetrical (3)	24	Highest monthly rainfall (mm)	Any value
		Compound (4)	25	Highest daily rainfall (mm)	Any value
11	Main cover type	Trees (1)			
		Shrub (2)			
		Grass (3)			

Engineering properties such as shear strength parameters, permeability functions, etc. of the slope forming materials were not included in the assessment because they were not readily available. These properties were normally used for stability analyses, to assess the level of risk posed by the slopes.

The stepwise discriminant analysis was carried out using various levels of significant value of F to add and delete the parameters in the analysis. From the results, it was seen that the higher the significant value of F to add new and delete variables, the higher the Eigen-values and χ^2 produced. The Wilk's λ however became smaller. Generally the number of significant parameters would increase with increase of the significant value of F to add and to delete.

Based on the highest percentage of correct classification and optimum number of significant parameters, significant value of F at 0.20 to add and 0.25 to delete was selected for model development. The ten optimum significant parameters that could discriminate the failed and not failed slopes were namely: the slope angle, feature area, distance to ridge, slope shape, percentage of feature uncovered, presence of rock exposure, rock condition profile, bench drain, horizontal drain and erosion. Discriminant function was then calculated using general regression formulae and canonical discriminant function coefficients.

Discriminant function of the 86 failed and 53 not failed slopes was computed. The boundary of the discriminant function separating these two groups (failed and not failed slope) was calculated using the average of the two group's means. Group mean for not failed and failed slopes were -0.91 and 0.58 respectively. The value of discriminant function separating these two groups (noted as *g*) could be calculated as:

$$g = (Y_f + Y_s) / 2$$

where, Y_f= Mean of failed group; and Y_s= Mean of not failed group

Value of *g* for the model is:

$$g = (0.58 - 0.91)/2$$
$$= -0.165$$

Using this *g* value, the boundary condition separating failed and not failed slopes is as follows:Not failed if $Y < -0.165$, otherwise failed.

The hazard rating was designed using the maximum and minimum value of discriminant function. The maximum value of discriminant function was 5.906 and minimum value was -7.083. Table3 below shows the hazard rating designed for the SAS.

Table 3 Hazard rating designed for the new SAS

Range	Rating
2.871 to 5.906	Very high
-0.165 to 2.871	High
-3.459 to -0.165	Low
-7.083 to -3.459	Very low

Evaluation of the SAS in predicting landslides was carried out on two new sites (Site C & E). The database was compiled between 1994 and 2003 for a total of 36 slopes, 21 slopes from Site C (along the Tapah – Cameron Highland road), and 15 slopes from Site E (along Kuala Lumpur – Bentung old road). A total of 25 landslides had occurred along these roads, 13 landslides along the Tapah – Cameron Highland road (Site C) between 1994 and 2000, and 12 landslides along the Kuala Lumpur – Bentung old road (Site E) in November 2003. The results of the comparative study are as shown in Table 4.

Table 4 Evaluation SAS in predicting landslides

(1)	Number of assessed slopes	36
(2)	Number of actual landslides or failed slopes	25
(3)	Number of Slopes classified as high hazard	28
(4)	Number of slopes classified as high hazard that actually failed	24
(5)	% of correctly classified failed slopes	96%

This SAS appears to be satisfactory.

Use of Bio-engineering for Stabilizing Slope

Bio-engineering in our context refers to the use of plants to enhance the stability of hill slopes primarily against shallow-seated landslide features. Grass and trees do grow well on slopes. Their roots reinforce the soil and prevent erosion. However grass roots tend to be shallow, while large trees may add undesirable weight to the slope and may in-turn induce instability. An alternative is to plant selected hardy species (short trees or shrubs) and to plant them spaced closely and deep into the slope. This is the basic principle of what is termed as the *live pole* technique.

The live pole technique which essentially involves planting woody stems (50-80mm diameter) of selected hardy species in close centre (0.5 - 1.0 m) arrays up to lengths of up to 2 - 3 into pre-drilled holes can provide immediate shear reinforcement down to 1.5 - 2 m depth, as illustrated in Figure 43. Theoretically the woody stems will initially act as short piles in reinforcing the slope. In the medium to long term roots will start to grow all along the woody stems, thus further reinforcing the soil in addition to the increase in the woody mass and increase in soil suction due to water uptake. The live pole would benefit the slope in the following manner:

Environment: Through increase of carbon sequestration to counter rising carbon dioxide levels in the atmosphere which is generally believed to bring about global warming.

Mechanical: Through reinforcement of rooted soil thus preventing soil surface erosion and slope failure, and,

Hydrological: Through reduction in run off by intercepting rain water during rain thus minimizing water from entering the slope which would otherwise weaken the slope. By keeping the slope relatively dry, the soil suction is maintained for longer periods thus keeping the slope stronger.

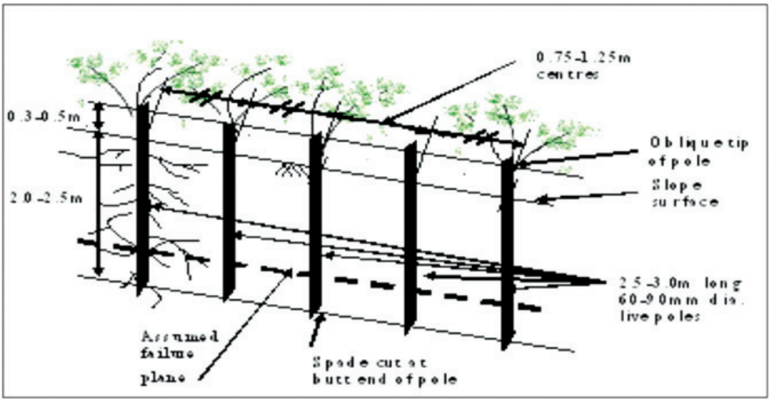


Figure 43 Principle of the live pole technique

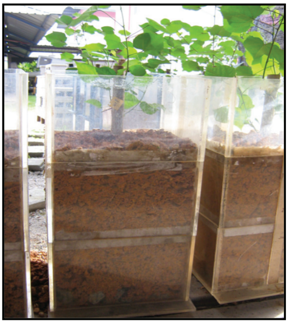
Our first challenge was to identify suitable plant species for such an application. No such attempt has ever been made before. Our study led us to 10 potential tropical plant species as listed in Table 5. These species were then planted in four replicates in a standard medium consisting of crushed well-graded sand and 10% organic matter for about 12 weeks in a shade-house. Based on screening trials and observation of distribution/location and shape of growing roots, three species were initially identified as the best candidates for the live pole technique, namely the *Hibiscus tiliaceus* (*Ht*), *Dillenia indica* (*Di*) and *Dillenia suffruticosa* (*Ds*). The shade house experiment was replicated but with mineral soils (aerisols, ferralsols, histosols, luvisols, regosols, gleysols, fluvisols) that commonly form Malaysian slopes. From this experiment, only 2 species i.e. *Ht* and *Ds* were were found to be the most suitable. Figure 44 shows the rooting trial.

Table 5 Potential tropical plant species for live pole

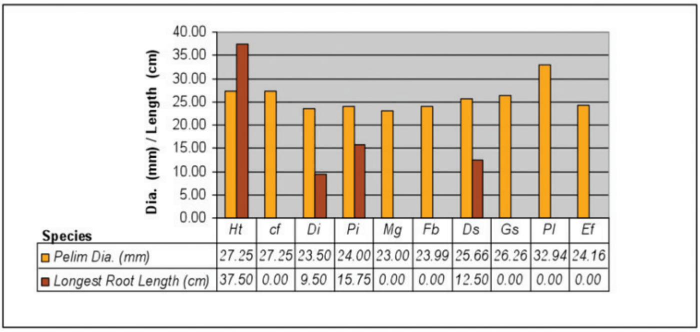
No.	Species
1	Hibiscus tiliaceus
2	Cassia fistula
3	Dillenia indica
4	Pterocarpus indicus
5	Macaranga
6	Ficus benjamina
7	Dillenia suffruticosa
8	Glyricidia sepium
9	Pajanella longifolia
10	Erythrina fusca

(a) *Hibiscus tiliaceus*

(b) *Dillenia suffruticosa*

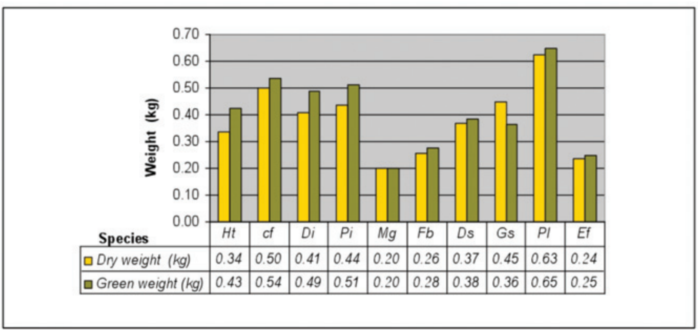


(c) Shade house sample box of acrylic



Species	Ht	cf	Di	Pi	Mg	Fb	Ds	Gs	Pl	Ef
Pelim Dia. (mm)	27.25	27.25	23.50	24.00	23.00	23.99	25.66	26.26	32.94	24.16
Longest Root Length (cm)	37.50	0.00	9.50	15.75	0.00	0.00	12.50	0.00	0.00	0.00

(d) Length of roots



Species	Ht	cf	Di	Pi	Mg	Fb	Ds	Gs	Pl	Ef
Dry weight (kg)	0.34	0.50	0.41	0.44	0.20	0.26	0.37	0.45	0.63	0.24
Green weight (kg)	0.43	0.54	0.49	0.51	0.20	0.28	0.38	0.36	0.65	0.25

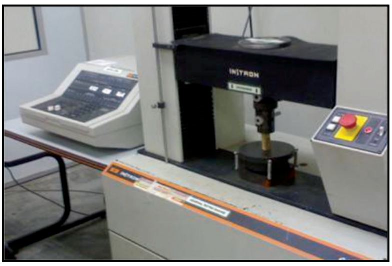
(e) Dry/Green weight

Hibiscus tiliaceus *Dillenia indica* *Dillenia suffruticosa*

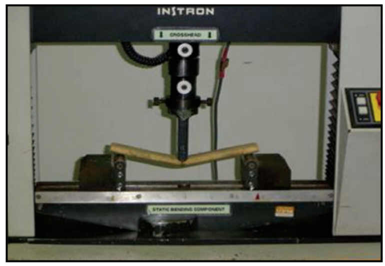
(f) Root growth along stem

Figure 44 Rooting trial

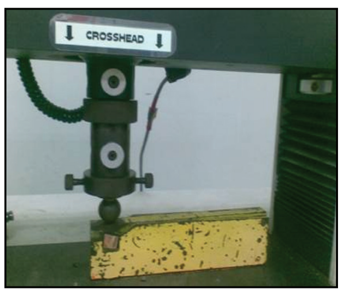
The mechanical properties of the selected species were determined based on BS 373 testing methods, for compressive, bending and shear, for design purposes. These are shown in Figure 45.



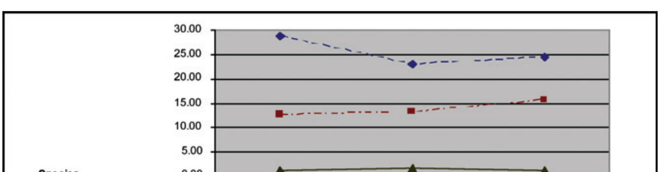
(a) Compression test



(b) Bending test



(c) Shear test

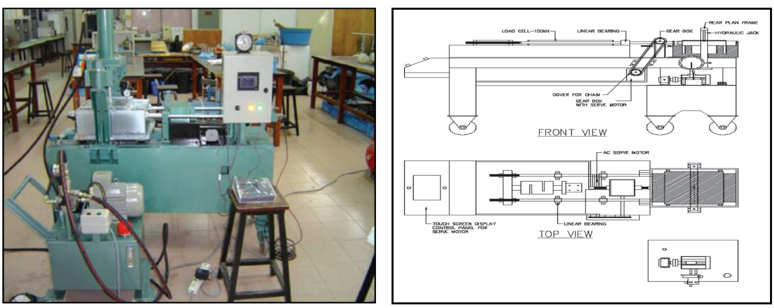


Species	Ht	Di	Ds
Bend. Strength (MPa)	28.925	22.951	24.598
Comp. Strength (MPa)	12.599	13.128	15.696
Shear Strength (MPa)	1.038	1.549	1.179

(d) Mechanical properties of the live pole

Figure 45 Test equipment and mechanical properties of the live pole

We also carried out shear test to measure the effect of the roots on soil strength. Unfortunately the conventional shear box was not good enough to model represent the effects of roots in a soil mass. For this, we had to design and fabricate our own large shear box. The equipment is shown in Figure 46. This box had a sample size of 300mm x 300mm x 200mm, compared with conventional equipment whose sample size is only 50x50mm. The shear load was applied using a serve motor and a gear assembly to control the strain rate. The load was measured through a load cell of 100 kN capacity. The normal load was applied through a hydraulic jack. LVDT were used to measure the horizontal and vertical displacements. All the equipment and the shear box assembly were mounted on a rigid steel frame and all the instruments on board were connected to a data logger.



(a) Photo (b) Schematic drawing

Figure 46 Large shear box

Table 6 shows the large shear box results for *Ht* roots on yellow well graded sand of Jalan Alumni after 13 months. It appeared that the presence of roots had significantly improved the shear strength of the soil and it also showed that the effect was mainly on the cohesion. Root of *Ht* had enhanced the cohesion component of shear strength by 593% (at 30 cm depth) and 722% (at 50 cm depth) as compared to the unplanted soil.

Table 6 Result of large shear box test for Jalan Alumni Soil

No.	Sample Description	Direct Shear Test				Eng. Parameters		Remarks
		σ_n (kPa)	τ(kPa)	φ(°)	c (kPa)	γ_n (kN/m^3)	w_n(%)	
1	*Jln Alumni SW (depth 30-50 cm)*	306.59	136	20.1	14.7	17.0	27.03	Unplanted. Compacted to as in situ
		459.88	166			16.9	28.60	
		613.17	248			16.4	29.76	
2	*Jln Alumni SW + Ht roots (depth 30 cm)*	306.59	176	17.5	87.2	15.9	27.85	Planted in Feb 1, 2008 Tested in March 7, 2009
		459.88	249			15.8	26.58	
		613.17	272			15.7	26.23	
3	*Jln Alumni SW + Ht roots (depth 50 cm)*	306.59	189	16.7	106.1	15.6	27.80	Planted in Feb 1, 2008 Tested in March 10, 2009
		459.88	263			16.0	27.02	
		613.17	281			15.6	28.39	

Table 7 Result of large shear box test of Jalan MARDI Soil

No.	Sample Description	Direct Shear Test				Eng. Parameters		Remarks
		σ_n (kPa)	τ(kPa)	φ(°)	c (kPa)	γ_n (kN/m³)	w_n (%)	
1	*Jln MARDI SP (Depth 30-50 cm)*	306.59	121	17.6	11.2	18.5	32.36	Unplanted. Compacted to as in situ
		459.88	131			18.3	33.62	
		613.17	218			18.1	36.99	
2	*Jln MARDI SP + Ds roots (Depth 30-50 cm)*	306.59	139	16.0	56.5	17.7	28.83	Planted in Feb 1, 2008 Tested in March 13-14, 2009.
		459.88	199			17.6	26.77	
		613.17	227			17.6	25.62	
3	*Jln MARDI SP + Ht roots (Depth 30-50 cm)*	306.59	146	17.1	60.2	17.7	30.90	Planted in Apr 2, 2008 Tested in March 12-13, 2009.
		459.88	220			17.7	29.26	
		613.17	240			17.7	27.69	

Table 7 shows the large shear box results for *Ht* and *Ds* roots in white poorly graded sand of Jalan MARDI after 12 months. As was expected, the roots of *Ht* and *Ds* enhanced the cohesion component of shear strength by 538% and 505% respectively as compared to the unrooted soil. *Ht* exhibited about 6% better cohesion than *Ds*.

To assess the suitability of the plant species and live pole in real situation, a field trial was carried out. A meta-stable slope in the vicinity of UPM campus was selected. The slope had a general angle of inclination of about 23-28^{o}, with shallow failures (0.95-1.5m deep) at several locations. Most of the failures were rotational failures, but translational failure was also evident on this slope. Figure 47 shows a general view of the trial site as it was in July 2007, with *rotational type* of failure.

Figure 47 The trial site as in July 2007

Two trial strips, each of about 2.0 m wide and 7.5 m long, along and down the slope were installed with a grid of live poles at 0.5 m and 0.75 m staggered centers across and down the slope. *Hibiscus tiliaceus* and *Dillenia suffruticosa* were selected for the live poles. They were planted as stem cuttings (50 – 80 mm in diameter) (Figure 48) in alternate rows, and to depths of 1.5 m below the ground surface.

(a) Stem cutting batches (b) Trial strip

Figure 48 Live pole stock and trial site as in April 2008

The planting operation was done in two days. All fresh cuttings of *Ht* and *Ds* had initial lengths of 2.10 to 2.30, diameters between 50 to 70 mm at upper end and 50 to 80 mm at butt end; and almost straight, smoothly tapered and with no bends or branch points forming large bifurcations. Asthe poles were to be installed for most of their length into the slope, and they should be free of bumps or angularity which prevent or hinder entry into the prepared holes. All live poles were freshly harvested, directly transported to the site in first day and kept cool and moist by wet gunny sack cover during planting days. The live poles were removed and installed by batch when the holes (about 100 mm diameter) had been pre-bored for installation using a 2 man augur as shown in Figure 49. Prior to installation, positions of 150 cm and 180 cm from butt ends were marked along the poles, the extreme 200 to 250 mm length of butt ends of the live poles were shaped into a point; also the upper end of each pole was protected from splitting by wrapping it, about 25mm from the end, with at least 2 turns of 1mm diameter galvanized wire.

Figure 49 Pre-formed hole by two- man powered augur

Each pole was placed butt end first into the base of a pre-formed hole and driven into the slope using a 14 lb hammer until about 1.5 m of length. After installation the annulus between the pole and its pre-formed hole was backfilled with fine dry sand to within 250mm of the surface of the slope. This final 250mm was then backfilled with the removed soil and topsoil. On completion of driving, the exposed end of each live pole was trimmed cleanly at an angle of about 60° to the longitudinal axis of the pole. Any splintered portion at the end of the pole was cut off cleanly prior to trimming, and the end re-wrapped with wire as described earlier.

After completing the planting stage, the following details were recorded for each installed live pole:

- identification number of the pole
- harvesting date
- installation date
- species
- live pole dimensions: length and diameter at both ends
- installation details: depth of hole, driven length.

Each live pole was labeled with a durable label detailing its identification number and date of installation. In addition, an irrigation system and temporary shading cover in the form of plastic netting was installed in each of the trial sites. Figure 50 shows the progress of the live pole after several months of field planting.

(a) Irrigation system

(b) Temporary shade

(c) 4 weeks later

(d) 3 months later

(e) 6 months later

(f) 18 months later (and after 2 rainy seasons)

Figure 50 Progress of the live poles as in field trials

As shown in Figure 50 (f), the surrounding slope had turned into a moon-like surface upon failure, whereas the live pole section still remained intact.

At the end of the 12 month monitoring period, 2 *Ht* and 2 *Ds* live poles were exhumed from the Jalan MARDI trial in order to study the root growth and properties of the live poles in a field condition as shown in Figure 51.

Figure 51 Exhumed live poles from the Jalan MARDI trial

A visual comparative study of the *Ds* and *Ht* live poles showed that about 25% of the embedded length of *Ht* was rooted, for *Ds* it was about 31% Figure 52). However, the roots of *Ht* seemed longer (230to 1230mm long) and thicker (0.5 to 5.7mm diameter) compared with roots of *Ds* (length: 350 to 1060mm; diameter: 0.5 to 2.5mm).

(a) *Ds* pole

(b) *Ht* pole

Figure 52 Roots of exhumed *Ht* and *Ds* pole

A finite element analysis was carried out to illustrate the benefit of the live pole in stabilizing slopes against shallow failures using 2D Plaxis (Figure 53). As expected, the factor of safety of the slope increased markedly. The live pole reinforcement was apparently more effective if poles were installed in closer array and soil had higher original cohesion.

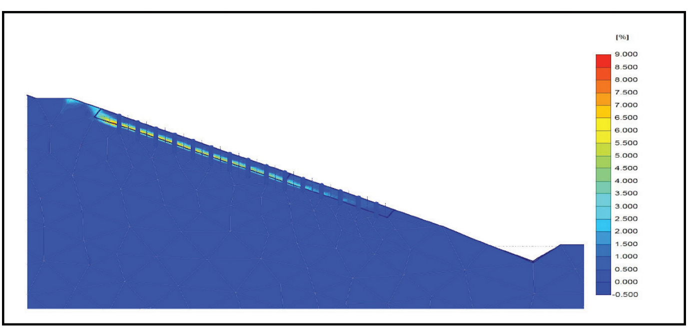


Figure 53 FE model of slope with live pole

Low Cost Repair for Failed Slopes

Small slope failures or shallow landslide, as shown in Figure 54, is a very common scene on Malaysian slopes. Their numbers are probably in several hundreds though nobody is really counting. These failures usually occur during or immediately after intense rain. Although shallow slides, unlike the major failures or big landslides, seldom cause fatalities, immediate repair works must be carried out to prevent it from triggering bigger failure. There are few slope repair methods being used to reinstate this type of failure but most of them need imported materials for construction purpose. It is an advantage to have a new slope repair method where locally available soil can be reused with recyclable material like scrap car tires. With this type of repair method, there will be a great savings for authorities who carry out the maintenance work. Apart from that, repair work can be carried out immediately without having to import new materials and waiting for big budgets.

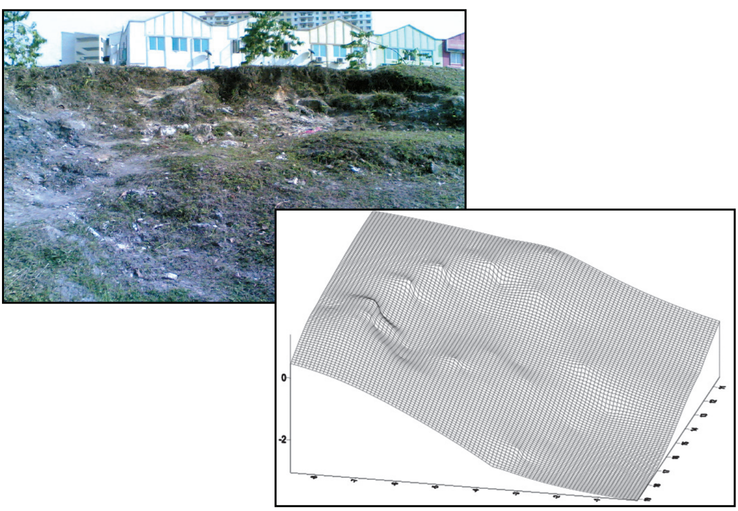
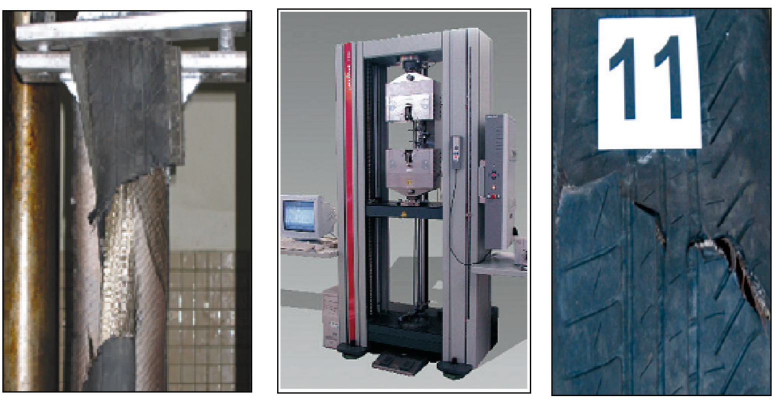


Figure 54 Shallow slope failure (location Equine Park, Selangor)

Huge quantities of unwanted scrap tires are being generated every year and result in major environmental hazards worldwide. The present recycling techniques of the scrap tires may only consume a very small amount of the unwanted tires. The percentage of scrap tires recycled is not keeping pace with the growth of scrap tires. This has become a serious problem in many countries including in Malaysia. The Rubber Manufacturers Association estimates that the United States produced 290 million scrap tires in 2003 alone. Historically, these scrap tires took up space in landfills or provided breeding ground for mosquitoes and rodents when stockpiled or dumped illegally. Fortunately, markets now exist for some of these scrap tires. About 44% of scrap tire in the US are used as fuel, 19% are recycled or used in civil engineering projects, 8% are converted into ground rubber and recycled into products, 4% are converted into ground rubber and used in rubber-modified asphalt, 3% are exported, and 2% are recycled into cut/stamped/punched products, used in agricultural and miscellaneous other uses. The remaining 10% (about 27 million scrap tire annually) are however still stockpiled or land-filled, In Malaysia, there were over 6 million passenger cars registered between year 1980 to 2005. This is partly due to the booming economy in recent years, as well as the national car industry. Assuming that each passenger car changes 4 tires every five years, there will be 4.8 million scrap tires generated annually. This figure is not inclusive of commercial vehicles, 4 x 4 vehicles and motorcycles.

In order to apply scrap car tires for slope repair, a good understanding of the tyre's physical and mechanical strength properties, and durability are required. There was however very little information available and this information was not provided or even studied. There were also no testing standards or guidelines that were suitable to test for suitable mechanical properties of the tyre for such an application.

We began by designing suitable tests and then tested the locally produced (scrap) car tires. We carried out tensile tests on commonly available sizes, i.e. R12 to R15. Two types of tensile tests were carried out. In the first test, the whole tyre (with side walls removed) was pulled using a universal tensile machine (UTM) until it broke. Its maximum tensile strength was recorded. In the second test, samples were cut into 100 mm width x 300mm length and tested in accordance to ASTM D4595 standards. Figure 55 shows the tyre tests described above.



(a) Tensile test with UTM   (b) Zwick tensile for ASTM D4594 test

Figure 55 Tensile tests on tires

Mean tensile strength of 55.81 kN, with standard deviation 15.19 kN was obtained from the pull-out test, with the statistical probability of tensile strength greater than 20 kN being 99%. The values obtained from the ASTM D4595 tensile test, which was originally developed for determination of the tensile properties of geotextiles and related products, using wide-width strip was slightly lower. Statistical probability of tensile strength greater than 20 kN was 89%. This was expected as the cutting process of the tyre samples induced major disturbances to the design structure of the tyre. The ring structure of a tyre was a continuous element that was designed to distribute stress evenly to the whole structure.

Our next task was to design suitable attachments to attach the tires together so as to act as an integral unit. This was an equally important aspect of the system design, and probably was the most delicate. Two attachment systems were considered and tested in the laboratory for tension, the wire rope and U-clip, and polymer rope of different numbers of wrap and knots. For the wire rope and U-clip system, a 12 mm diameter wire rope with its compatible sized U-clip was selected for testing purposes. Whilst for the polymer rope, 12 mm diameter polypropylene was chosen for ease of handling and economy. The rope was tightened on the cross beam of the tensile machine and pulled until the rope ruptured. The wire rope had working load of 25 kN but the attachment (wire rope and U-clip) could only carry 15 kN, and was therefore a suitable attachment for the scrap tire wall. Furthermore the cost of this attachment was merely RM 1.50 per unit.

Next we needed to design and study the performance of the proposed tire system. A full scale field trial was carried. The trial site was a previously failed slope in IKRAM Park. The proposed wall comprised of whole scrap tires tied together to make a mat configuration, backfilled with in situ cohesive soil, and then placed in successive layers so that the resulting structure could function as a retaining wall. This concept was similar to that employed in reinforced soil. This arrangement would maximize the number of scrap tires used in a single structure. The failed slope was first excavated as shown in Figure 56. Scrap tires with one side wall (top side) removed for better soil compaction were tied together with polypropylene rope into mat configuration and staggered layer by layer to form the 7 m wide by 5 m high slope, with slope angle of about 35°. Lengths of the reinforcement (tire mats) were 5 m at the base and 3m at the top of the wall.

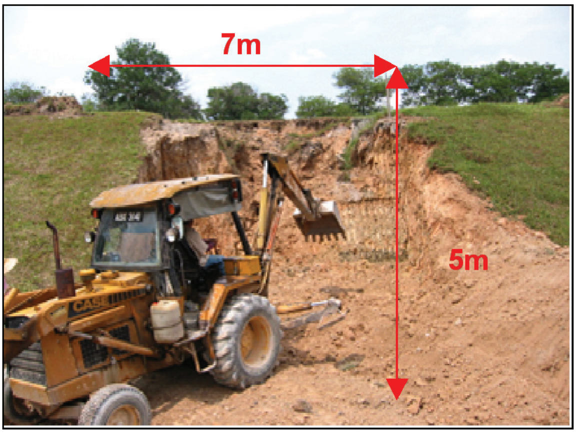


Figure 56 Trial site in Ikram Park, Selangor

In situ cohesive soil was used as backfill, and compacted with 1 ton smooth-wheel roller compactor, each layer being compacted to a thickness of about 200 mm, i.e. the thickness of the tire layer. Figure 57 illustrates the construction sequence. A total of 2100 number of scrap car tires in 25 layers was used. It only took 5 unskilled workers 20 working days to complete the structure. Instrumentation like settlement plates and pressure cells were installed to monitor the performance of the trial wall.

(a) (b) (c)

Figure 57 Constructing the trial scrap tire wall: (a) tires tied with polypropylene rope, (b) Laying first tires in mat configuration and compacting backfill, and (c) the completed structure – 2 years after construction

The in situ (excavated) slope material would be deemed as unsuitable fill for conventional reinforced slope, but was used in this case to minimize repair cost. For example, FHWA-HI-95-038 recommends backfill material for reinforced slopes to have percentage passing 0.075 mm sieve less than 50%, and PI less than 20%. Field density tests using sand replacement method on compacted fill gave relative compaction of 78% to 89%. The in-situ moisture contents were between 24% and 26%, which was on the wet of optimum due to construction being done during the rainy season. Instrumentation (settlement plate and vibrating wire cell) installed showed satisfactory performance of the system with regards to settlement and earth pressures. Our structure was still standing several years after construction.

Besides technical feasibility, construction cost plays an important role during selection of appropriate systems for slope rehabilitation work. Construction cost of a particular system will vary depending on factors such as: (1) location and size of slip (slope failure), (2) weather conditions, (3) availability of plant and operator skills, (4) haulage distances and availability of materials, (5) motivation of workforce and site operations, and (5) type of treatment used.

A study on the construction cost for five types of commonly used retaining wall systems in Malaysia (gabion wall, stone pitching (rubble) wall, RC wall, Terramesh wall and reinforced earth wall) was carried out. These costs were then compared with the cost of constructing the proposed scrap tire system. The following software (GawaWin, Prokon and MacStars) were used for the preliminary design. Some basic design assumptions were (1) maximum bearing capacity of base was 100 kPa, (2) effective internal friction was 28°, (3) no water level, (4) surcharge was 10 kPa, and (5) factors of safety against sliding, overturning, bearing failure and overall slope stability were 1.5, 1.5, 3.0 & 1.5 respectively. As shown in Figure 58, the proposed scrap tire system was apparently the most cost effective for walls of 2m to 6m. The significant advantage of the proposed scrap tire system was the reduced cost of wall materials and construction. Most of the wall components (scrap tires and back fill soils) were recycled materials. Further, the construction of the wall was relatively simple. It did not require skilled workers and heavy machinery.

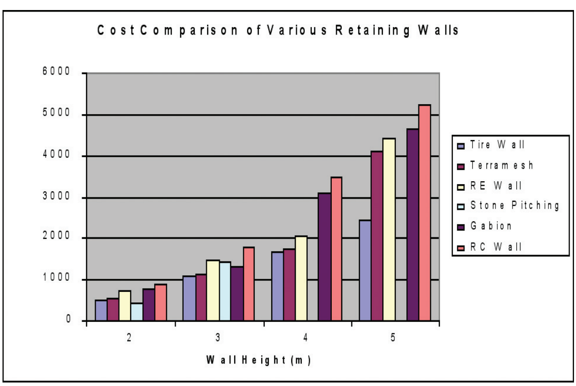


Figure 58 Cost per linear meter of various retaining wall systems versus height

In 2009, we published a book entitled '*Landslides in Malaysia: Occurrences, Assessment, Analysis and Remediation/Preventive Solutions*' with Universiti Putra Malaysia Press, ISBN 978-967-5026-39-3. This is the first book of its kind in the country. It was given an Excellence Award by Universiti Malaya in the same year.

An interesting case of utilizing scrap tires for a retaining wall was reported by an Alaskan Authority, City of Kodiak Parks & Recreation, Alaska, USA (Figure 59). They had utilized scrap tires to make a part of the retaining wall at Baranof Park (in August 2009) taking cues from our published work (*www.ejge.com/2008/Ppr0825/Abs0825.htm)*.

(a) (b)

Figure 59 Scrap tires retaining wall: (a) tires with one side wall removed tied with polypropylene ropes, and (b) wall construction in progress

Acknowledgement

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Note: Further details on materials summarized in this Section (4.3) can be found in the following publications:

Slope Assessment System

1. (2006). Slope Assessment System for Predicting Landslide. *Geo-Singapore*. December 11-13, 2006. Singapore. 13-26. *(keynote address)*
2. (2006).Evaluation and Development of Cut-Slope Assessment Systems for Peninsular Malaysia in Predicting Landslides in Granitic Formations. *Journal Technology B*. University Technology Malaysia. 44B: 31-46
3. (2006). Evaluation of Slope Assessment Systems for Predicting Landslides of Cut Slopes in Granitic and Meta-sediment Formations. *American Journal of Environmental Sciences.* Special Issues on Recent Advances in Landslide Engineering and Technology. NY, USA. 2(4): 135-141
4. (2007). Slope Instability and Climate Change for Malaysia. *Expert Symposium on Climate Change: Modeling, Impacts and Adaptations*. December 17-19, 2007. Singapore. *(invited lecture)*
5. (2008). Slope Assessment System for Landslide Prediction Using Discriminant Analyses. *5th International Conference on Slope Stability & The Safety of Infra-structures*. Kuala Lumpur. July 24-26, 2008. pp. 7-22. *(keynote address)*
6. (2008). Slope Assessment Systems: A Review and Evaluation of Current Techniques Used for Cut Slopes in the Mountainous Terrain of West Malaysia. *Electronic Journal of Geotechnical Engineering*. OK:USA. 13E

Bio-Engineering

1. (2008). Use of Bioengineering (*Live Pole*) and Scrap Tires for Mitigating and Repairing of Slope Failures. *Geo-Chiangmai*. December 10-12, 2008. Chiangmai, Thailand. *(keynote address)*

2. (2009). Live Poles for Slope Stabilization in the Tropical Environment. *Electronic Journal of Geotechnical Engineering*. 14G
3. Stability of Tropical Soil Slope Reinforced by Live Pole: Experimental and Numerical Investigation. *Journal of Soil and Water Conservation (accepted for publication)*.
4. Green Slope Engineering Using Live Pole Technique on Tropical Slopes. *Int. Journal of Geotechnics and Environment (accepted - to be published in 2010)*.

Scrap Tire Wall

1. (2008). Application of Scrap Tires as Earth Reinforcement for Repair of Tropical Residual Soil Slope. Electronic Journal of Geotechnical Engineering. USA:OK. 13 B
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BIOGRAPHY

Dr. Bujang Bin Kim Huat (Huat, B.B.K.) was born on December 13, 1958 in Kuching, Sarawak. He received his Diploma in Civil Engineering from Universiti Teknologi Malaysia, Kuala Lumpur in 1979. He continued his education in the United Kingdom and graduated with a first class degree with honors in Civil Engineering from the Polytechnic of Central London in 1983. Upon graduation he returned to Kuala Lumpur to work as a consultant engineer for 2 years before joining UPM as a tutor in 1984. He continued his education at the Imperial College London where he obtained his MSc and DIC in Soil Mechanics in 1986.

He was promoted to the post of Associate Professor in 1993, and Professor in 2006.

From 1994 to 2000, he was seconded to Universiti Malaysia Sarawak (Unimas) to help set up the new university, where he held various administrative positions from Head of Department to Deputy Dean and Dean of the Faculty of Engineering. Currently he is the Deputy Dean of UPM School of Graduate Studies, in charge of thesis and academic records.

He is a member of several professional bodies, namely the Commission 18 on problematic soils, TC6 of the International Society of Soil Mechanics and Foundation Engineering and member of the World Association of Soil and Water Conservation.

He serves on the editorial board of several international journals such as the Electronic Journal of Geotechnical Engineering (USA) since 2006, the American Journal of Applied Sciences (USA) since 2005, the American Journal of Environmental Sciences (USA) since 2005, and the American Journal of Engineering and Applied Scienc,e since 2009. He also serves as a reviewer for several Citation Index journals namely the Computer and Concrete Journal, Arabian Journal of Geosciences, Geotechnical Testing Journal, Engineering

Geology, Scientific Research and Essays, Canadian Geotechnical Journal, Advances in Engineering Software, the International Journal of Water and Waste Management and the Journal of Hydrology.

In research, his interests span from basic to applied research in the fields of Soil Mechanics, Geotechnical and Foundation Engineering which include Non Destructive Testing Techniques, Geo-environment (waste disposal – concerns, facilities & legislation, and use of waste materials for geotechnical engineering applications), Slope Engineering, Bio-Engineering and Bridge Scour. His interests are indicated by the list of his publications which include characterization, behavior and properties of soft mineral, organic, peat and tropical residual soils; soil stabilizations with vertical drains, stone columns, chemical admixtures, deep stabilization and piled embankment; laboratory and numerical models (Finite Element simulations), centrifuge and field testing, design methods and appropriate construction techniques; slope assessment systems, suction-rainfall relation, infiltration characteristics and permeability functions, design/modification of laboratory equipment for testing residual soils in an unsaturated state, materials for pavement, scouring of bridge piers, precast shell footings and applications of bio-engineering for slope stabilization.

He has headed 14 research projects since 2000. The funding for his projects came from several sources which include the IRPA and Science Fund (Ministry of Science, Technology and Environment/ Innovation), the Ministry of Education, university internal funding and collaboration with industries, local as well as foreign institutions such as the Norwegian Geotechnical Institute, MARDI, the Public Works Department, IKRAM, Pilecon Engineering Berhad, CIDB and the JSPS (Japan Society for Promotion of Science). His research supervision includes 4 PhDs, 10 Masters and more than 100 undergraduate final year project students since 1991.

He has authored and co-authored 20 books (2 of which were published by Taylor and Francis, United Kingdom), 13 book chapters, edited 15 conference proceedings, and published more than 100 journal articles and 100 conference proceeding papers in the field of soil mechanics and foundation engineering.

LIST OF INAUGURAL LECTURES

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