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THE APPLICATION OF THE GEOTECHNICAL METHOD AND SATELLITE TRACKING DATA FOR LANDSLIDES STUDIES

(APLIKASI KAEDAH GEOTEKNIKAL DAN SATELIT PENGESAN DATA UNTUK KAJIAN TANAH RUNTUH)

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TAJUK PROJEK : THE APPLICATION OF THE GEOTECHNICAL METHODS AND SATELLITE TRACKING DATA FOR LANDSLIDE STUDIES

Saya OTHMAN BIN ZAINON

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ABSTRACT

The rapid development in Malaysia such as housing scheme at hilly terrain, construction of highways, mining activities and river bank instability especially in town areas such as Kuala Lumpur and Penang has triggered many landslide disasters. Since 1970 until 2002, more than 300 landslides have occurred throughout Malaysia and at least 30 landslides reported in Klang Valley alone. Most of the tragedies were largely triggered by incidences of heavy rainfall during the monsoon season. A landslide can be defined as the movement of the sand slope, rock and organic sources due to the gravity attraction. The landslides are caused by weather and external mechanism like heavy rain, human activities and slope erosion. Generally, there are various types of investigations and instrumentations used in monitoring the landslide's phenomena. The main investigations in landslide monitoring are geological structure, satellite tracking data (GPS) observation and geotechnical method. This report discusses the application of geotechnical method and satellite tracking data (GPS) using Rapid Static techniques in landslide studies. The study area of this project is located at the hillside area at Section 5, Wangsa Maju, Kuala Lumpur. The monitoring network consists of 16 monitoring points and two control points of the Coordinated Cadastral Survey and MASS station, namely W474 and KTPK, respectively. Observations were carried out at two epochs of observations independently using the single frequency GPS receivers. The GPS observation data had been processed and analysed using the Javad Pinnacle version 1.0 and STARNET-Pro software, respectively. For the geotechnical method, data was taken using the Mackintosh probe and laboratory tests was carried out on the disturbed soil samples at the study area. Results from the analysis showed that GPS technique with a standard specification by following a stipulated procedure can be used to detect horizontal and vertical deformation to centimeter level. However, it was found that there was no significance ground movement along the study area.

ABSTRAK

Pembangunan yang pesat di Malaysia seperti skim perumahan berhampiran kaki bukit, pembinaan lebuhraya, aktiviti perlombongan dan ketidakstabilan tebing sungai terutama di kawasan bandar seperti Kuala Lumpur dan Pulau Pinang telah menyebabkan berlakunya beberapa kejadian tanah runtuh. Sejak 1970 hingga 2002, lebih 300 kejadian tanah runtuh berlaku di seluruh Malaysia dan sekurang-kurangnya 30 tanah runtuh berlaku di Lembah Klang sahaja. Kebanyakan tragedi ini disebabkan oleh insiden hujan yang turun pada musim monsun. Tanah runtuh boleh didefinisikan sebagai pergerakan cerun tanah, batuan dan sumber organic disebabkan oleh tarikan graviti. Tanah runtuh adalah disebabkan oleh cuaca dan mekanisme luaran seperti hujan lebat, aktiviti manusia dan hakisan tanah. Umumnya, terdapat pelbagai jenis penyiasatan dan peralatan telah digunakan dalam pemantauan fenomena tanah runtuh. Penyiasatan utama dalam pemantauan tanah runtuh adalah struktur geologi, cerapan satelit pengesan data (GPS) dan kaedah geoteknik. Laporan ini membincangkan aplikasi kaedah geoteknik dan satelit pengesan data (GPS) menggunakan teknik Rapid Statik dalam kajian tanah runtuh. Kawasan kajian projek ini terletak di kawasan kaki bukit Seksyen 5, Wangsa Maju, Kuala Lumpur. Jaringan pemantauan mengandungi 16 titik pemantauan dan dua titik kawalan bagi Ukur Kadaster berkoordinat dan stesen MASS, iaitu W474 dan KTPK. Cerapan dibuat untuk dua epok berasingan menggunakan receiver GPS satu frekuensi. Data cerapan GPS telah diproses dan dianalisis menggunakan software Javad Pinnacle version 1.0 dan STARNET-Pro. Untuk kaedah geoteknik, data telah diambil menggunakan Mackintosh probe dan ujian makmal terhadap sampel tanah terganggu dari kawasan kajian dilakukan. Hasil daripada analisis menunjukkan teknik GPS dengan speksifikasi piawai dan prosidur yang ditetapkan boleh diaplikasikan untuk mengesan deformasi ufuk dan tegak hingga aras sentimeter. Bagaimanapun, didapati tiada pergerakan tanah berlaku di kawasan kajian.

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A	-	The design matrix
b	-	The misclosure vector
$\boldsymbol{C}_{\hat{x}}$	-	Covariance matrix
$Q_{\hat{d}}$	-	Cofactor matrix
I	-	identity matrix
l	-	The vector of observations
l_o	-	Vector of computed observation
n	-	Number of observations
u	-	Number of parameter
W	-	The weight matrix
x	-	The vector of unknown parameters
$\hat{x}_1, \hat{x}_2$	-	The vector of corrections to the approximate values
$\hat{v}$	-	The vector of residuals
$\hat{x}$	-	The vector of corrections
$\boldsymbol{\sigma}_o^2$	-	A priori variance factor
$\hat{d}$	-	Displacement vector

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CHAPTER I

BACKGROUND

1.1 Introduction

Landslides are diffused, complex natural phenomena that occur randomly. It occurs worldwide and is described as sudden, short-lived geomorphic events that involve the rapid-to-slow descent of soil or rock in sloping terrains. A landslide is a general term given to describe the various forms of mass movement such as down-slope movement of soils, rocks and organic materials under the influence of gravity force. Landslides can be triggered by gradual processes such as weather or external mechanisms, for examples, undercutting of slope erosion, intense rainfall and loading on upper slope. Landslides are not individual events. They often occur in conjunction with at least one of the many contributory factors.

There is countless number of factors involved in the landslide process. Some triggering factors of landslide are due to climatic, tectonic and human reasons. In the event of a rainstorm in tropical regions, heavy rainfall can induce slope failure and snowmelt in cold climates could cause landslides. Human activities are concerned in the nature of mining and quarrying, the surface material and underlying bedrock can be hampered by sudden shaking and vibrations. The soil type can further explain this impact on the crustal level.

Obviously, different soil or rock types respond in distinct ways to the amount of movement. The particle bonding dictates the overall strength or weakness of the rock and its ability to withstand external impacts. In a similar context, the water content of a rock or soil is important in identifying the cause of a landslide hazard. In addition to determining whether mass will move in landslide studies, it is necessary to determine the direction, velocity and acceleration of movement and even movement surface of study area. Due to these reasons, different disciplines should study and interpret the results together in landslides investigation – see Yelcinkaya and Bayrak (2002).

Generally, there are various types of investigation and instrumentation being used in monitoring the landslides phenomena. The main investigations are geological structure, surface deformation, ground water and geotechnical. According to Nakamura, et. al. (1996), the investigation of surface deformation is conducted to define the boundaries of the landslide, size, and directions of the movement. The instrumentation used for the surface deformation investigation includes extensometer, ground tiltmeter, movement determination by aerial photographs, and Global Positioning System.

The investigation of geological structure relies on exploratory borings, geophysical surveys and the evaluation of slide plane using the instruments such as pipe strain gauge, inclinometer and multi-layer movement meter depending on the requirements for surveying accuracy and magnitude of the movement. Investigation of ground water includes ground water level, pore water pressure, ground water logging, ground water tracing test, geothermal survey, and geophysical logging - see Nakamura, et. al. (1996).

In Malaysia, the rapid economic developments over the last decades have necessitated the cutting of many hill slopes in order to minimize land utilization – see Hui (1999). The development of highlands area such housing, highway and golf course construction, intensive forest logging have resulted in frequent occurrences of landslides. Roslan and Mazidah, has created the ‘ROM’ scale (after the name of researcher Roslan and Mazidah) to detect the occurrences of landslides in certain areas. Result from the soil sampling shows that from Tapah to Tanah Rata road at (km 47), Grik to Jeli (km 27) are the critical areas. Bukit Antarabangsa, Genting Highlands, Kuala Lumpur to Karak Highway (km 63.8), Paya Terubung are the highest landslide areas, while Tapah to Tanah Rata road (km 47) and Bentung to Raub road (km 27) are the lowest landslide areas.

Roslan also said that there are 149 areas along the North South Highway have the potential of landslides phenomena as reported by Marzita Abdullah (2000).

The above result shows that efficient and effective monitoring techniques should be established in order to detect the rate of movement, size and the direction of the landslides. Several monitoring techniques are available such as geologic, geotechnical and geodetic. The classical methods of land surveys, inclinometer, extensometer and peizometer are still the most appropriate one – see Popescu (1996). Although there are lots of classical instrumentation and monitoring techniques, Global Positioning System has the potential in monitoring the landslide phenomenon continuously. The Geographical Information System is the best tool to create the landslide maps and databases in 1D, 2D and 3D view with various analysis in the context of GIS.

1.2 Problem Statement

Landslide has become a very serious threat and problem in Malaysia in recent time. This phenomenon has been accelerated by the rapid development especially at hilly terrain, construction of highways, mining activities and river bank instability – see Tan (1996). Nevertheless, activities such as land clearing, reclamation and rehabilitation should be established only after a thorough study on the impacts of soil erosion has been performed. Instances of severe soil erosion occurrences are among the major cause of landslide, which have marked a black chapter in Malaysia. Since 1970 until 2002, more than 300 landslides have occurred throughout Malaysia and at least 30 landslides have been reported in Klang Valley.

Hillside development in the urban areas (Kuala Lumpur, Penang, etc.) is a topic of major concern in Malaysia. Most of the tragedies were largely triggered by incidences of heavy rain either a single heavy rainstorm event or successive days of moderate rain during the rainy season. The real time rainfall values in the hilly terrain could serve as a useful indicator of the risk level of landslides. Therefore, monitoring the landslides and solving their mechanism are very important to prevent and reduce their negative effects – see Kalkan, Baykal, Alkan,Yanalak, and Erden, (2002).

Landslides monitoring involves determining certain parameters and how they change with time. The most important parameters are groundwater levels in the slope, movement involves the depth of failure plane, direction of the landslide, magnitude and rate of the landslide movement – see Kane and Beck (1999). There are several types of measuring system being applied in monitoring the landslide tragedies worldwide such as geologic methods, geodetic methods, geotechnical methods such as piezometer, inclinometer, tiltmeter, extensometer and Total domain reflectometry (TDR), Global Positioning System, photogrammetry and remote sensing. All the investigations are carried out before and after any landslide tragedy. However, in Malaysia, the investigation is only carried out immediately after the incident occurs by the government sectors such as the Geological Survey of Malaysia, Department of Public Worker and other private sectors, using the geologic and geotechnical methods. The photogrammetry techniques are the most popular techniques in monitoring landslides in Malaysia. Nevertheless, GPS techniques are now being used widely in Malaysia to study and interpreted result of landslide behaviour.

Each of the monitoring methods provides certain information on the state of landslide body. As one of the latest surveying technology, it has been proven that the GPS method has substantial advantages but gives limited information on the surface movement - see Chrzanowski (1986). Generally, the geotechnical methods gave limited information of the subsurface of deformable body, which are capable of providing measurement in 1-dimension – see Hill and Sippel (2002). According to Kalkan, Baykal, Alkan, Yanalak, and Erden (2002), the cooperation of the Geotechnical, Geologic and GPS methods can give more satisfying result of the landslide behaviour. Beside that, Geo-spatial database (i.e. GIS approach) can be exploited as an efficient and cost-effective means of storing, processing and analysing the spatial geographical data to provide preventive measures Ashok (2001), Halounova, and Pavelka, (2000). The combination of geotechnical and GPS information will better define the mechanisms and the processes that generate and propagate slides as well as the interaction between different physical properties of soils and the stability of slopes.

1.3 Research Objective

The objective of this research is:

1. To employ the GPS techniques and geotechnical method for data acquisition in areas proned to landslide activities.
2. Analyse and determine the magnitude of the landslide behaviour.

1.4 Methodology and Scope of the Study

Landslide studies require high precision measurements and proper structural deformation networking and analysis technique. The geotechnical data and the satellite data system through the GPS technologies are capable of giving deformation conditions of the slope for safety purposes. The method of GPS employed for this research is the ‘Rapid Static Mode Positioning’ while for the geotechnical; the data were taken using the Mackintosh Probe method and laboratory test. These two methods are reliable, accurate and efficient for landslide monitoring deformation.

This research involves the following tasks:

1. Literature review on GPS technology and geotechnical and their applications in deformation surveys for slope.
2. Setting up and pre-analysis of various monitoring networks design with respect to the specified monitoring technique.
3. Testing and commissioning the hardware/software.
4. Field measurements for monitoring the landslide. This part will involve a technique of GPS rapid static and geotechnical method made at the control and monitoring stations within certain epochs (after completing the monuments at the test sites)
5. Data processing and validation, adjustment and analysis (pre and post analysis).
6. Deformation modeling and rating of object stability

1.5 The Study Area

The study area is located at latitude, between 3° 11’ and 3° 12’ and longitude between 101° 44’ and 101° 46’. The area are located approximately 10 km from Kuala Lumpur, see Figure 1.1.

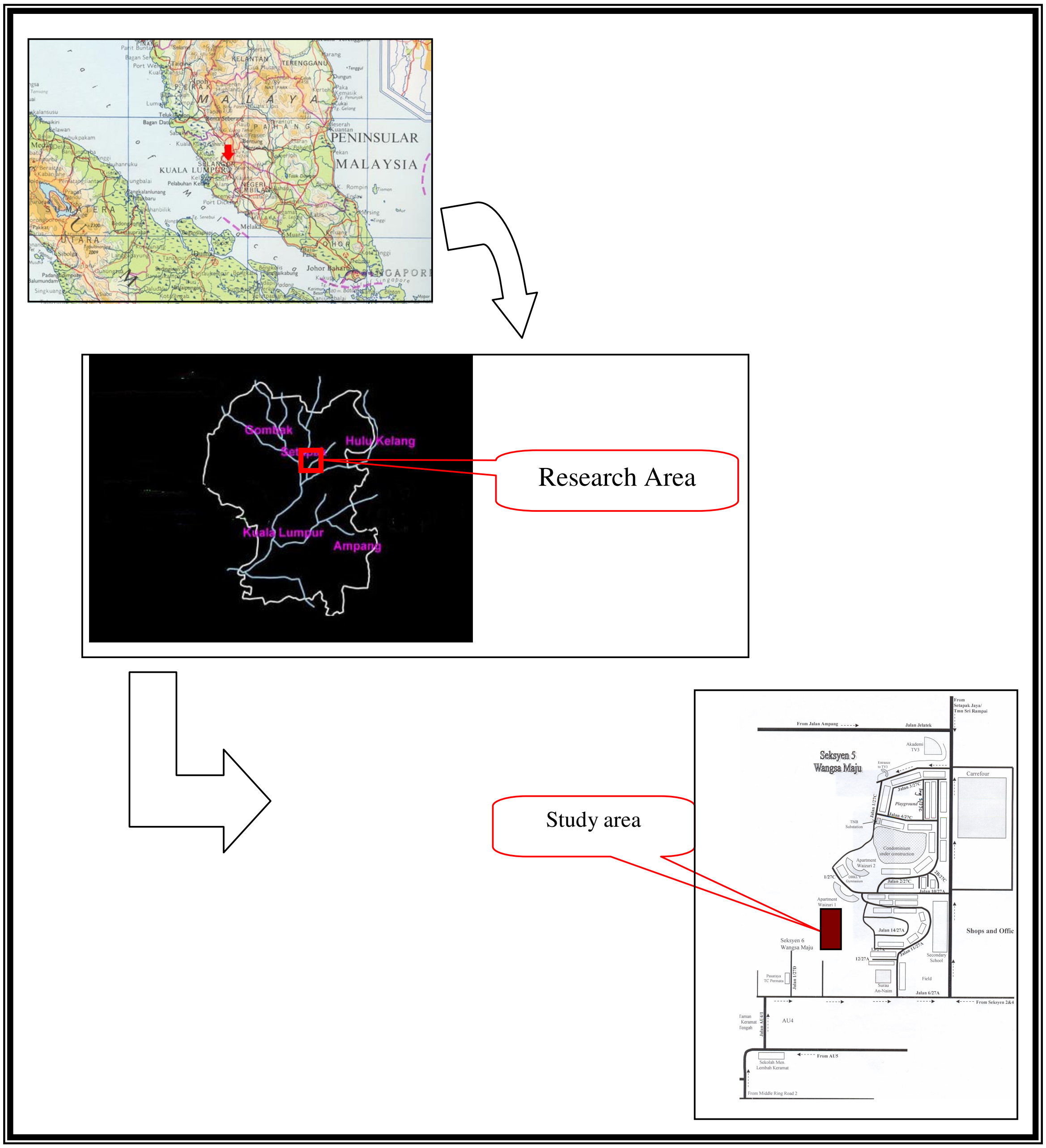


Figure 1.1: Study area

The published geological map indicates the site was underlined by graphitic schist and quartz-mica schist of Ordovician age – reported by Coffey (2001). This type of rock is a part of Hawthornden Schist, a low to medium grade metasedimentary rock formation in Kuala Lumpur area. Quartzite and phylitte members were also present within this formation. This formation has been noted to be strongly folded and contorted due to severe tectonic activities because of granitic intrusions. Quartz veining and quartz dykes associated with fault structures are common within this formation – see Rahayu, (1997); Zulkarnain (1997)

The most suitable area to be studied is Wangsa Maju. This is because a landslip has once occurred at the back row of terrace linked houses at Jalan 14/27A, Section 5, Wangsa Maju, Kuala Lumpur. It was noticed by the tenants of the properties concerned on 26th April 2001 after a very heavy rainfall. A short report had been made by the developer describing the observed surface condition, potential causes and mechanisms of slippage. The existing cut slope where the landslip had occurred was approximately 30 to 35 m high with 6 beams and the landslip happened at the foot of the existing cut slope at the bottom first and second beam slopes – Figures 1.2 and 1.3.

Figure 1.2: Landslide behind the terraced at a double storey houses in 26th April 2001.

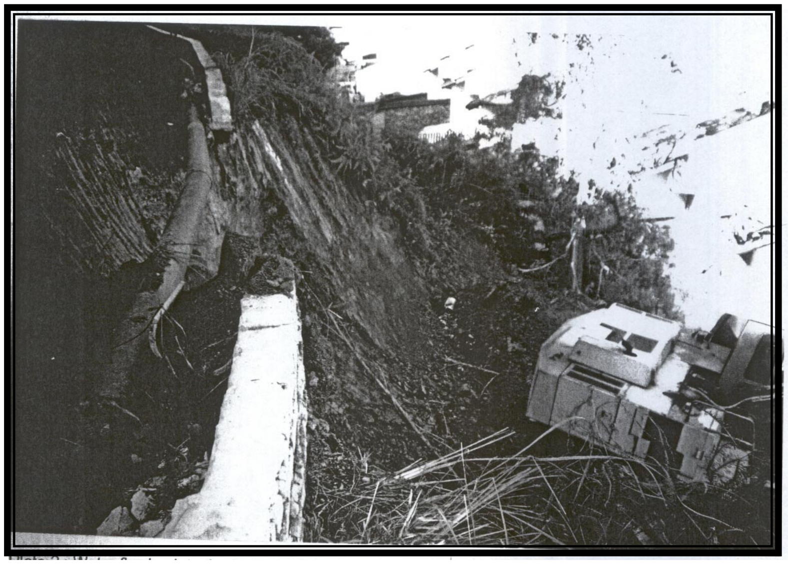


Figure 1.3: Water flowing into the landslip was diverted using PVC pipe

Figure 1.4 and 1.5 shows the picture of existing study area at Section 5, Wangsa Maju Kuala Lumpur, respectively.

Figure 1.4: The existing slope view

Figure 1.5: Picture of the research area.

Figure 1.6 shows the stream within the observation area. Some cracks can also be seen in the soil and at the concrete wall.

Figure 1.6: Picture of a stream at the study area.

CHAPTER II

LANDSLIDE MONITORING

2.1 Introduction

A landslide is a natural geologic phenomenon in which the soil and/or rock mass resting on top of sliding surface starts to slowly or rapidly move down the slope because of the pull of gravity. Many definitions have been applied to the term and they vary depending on the objective of the authors. Cruden, 1991 in Popescu (1996) define a landslide as a movement of a mass of rock, earth or debris down a slope. However, Hutchinson (1988) suggested that slope movements were categorized into individual groups based on the mechanism of failure. He classified them as fall, topple, rotational, translational slides, lateral spreading, flow and complex. The common features of landslides according to the above criteria are seen as slope failures, mudflows, rock falls and rockslides.

2.2 Causes of Landslide

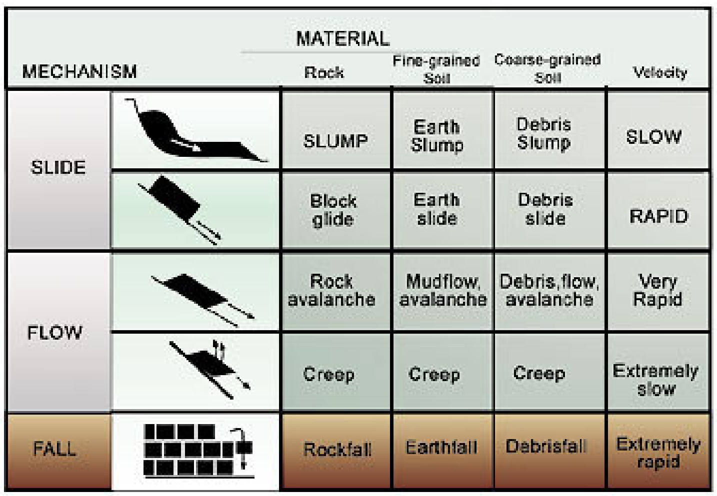
According to Ramakrishnan et. al. (2002), the causes of landslides can be divided into external and internal causes. External cause resulting in an increasing of the shearing stress such as geometrical change, unloading the slope toe, loading the slope crest, shocks and vibration, draw down and change in water regime. The internal cause is the steeping of the slope, water content of the stratum and mineralogical composition and structural features, which are tending to reduce the shearing strength of the rocks. Other causes of landslides include earthquakes and loud sounds. Many types of landslide move seasonally or periodically and may lie dormant for years.

2.3 Types of Landslides

Generally, landslides are classified into slides, falls, topples, lateral spreads and flows – see Varnes (1978). According to Leroueil, et. al. (1996), *Slides* move as large bodies by slipping along one or more failure surfaces. *Falls* of rock or soil originate on cliffs or steep slopes. Large rock falls can be catastrophic events. *Flows* are landslides that behave like fluids. Mudflows involve wet mud and debris – Hunt (1984) – sees Table 2.1 and Figure 2.1.

Table 2.1: Classification of landslides by mechanism, material and velocity

(Source: http://www.hbcumi.carl.edu/tqp/301-15/landslides.gif)



MECHANISM		MATERIAL: Rock	Fine-grained Soil	Coarse-grained Soil	Velocity
SLIDE		SLUMP	Earth Slump	Debris Slump	SLOW
		Block glide	Earth slide	Debris slide	RAPID
FLOW		Rock avalanche	Mudflow, avalanche	Debris,flow, avalanche	Very Rapid
		Creep	Creep	Creep	Extremely slow
FALL		Rockfall	Earthfall	Debrisfall	Extremely rapid

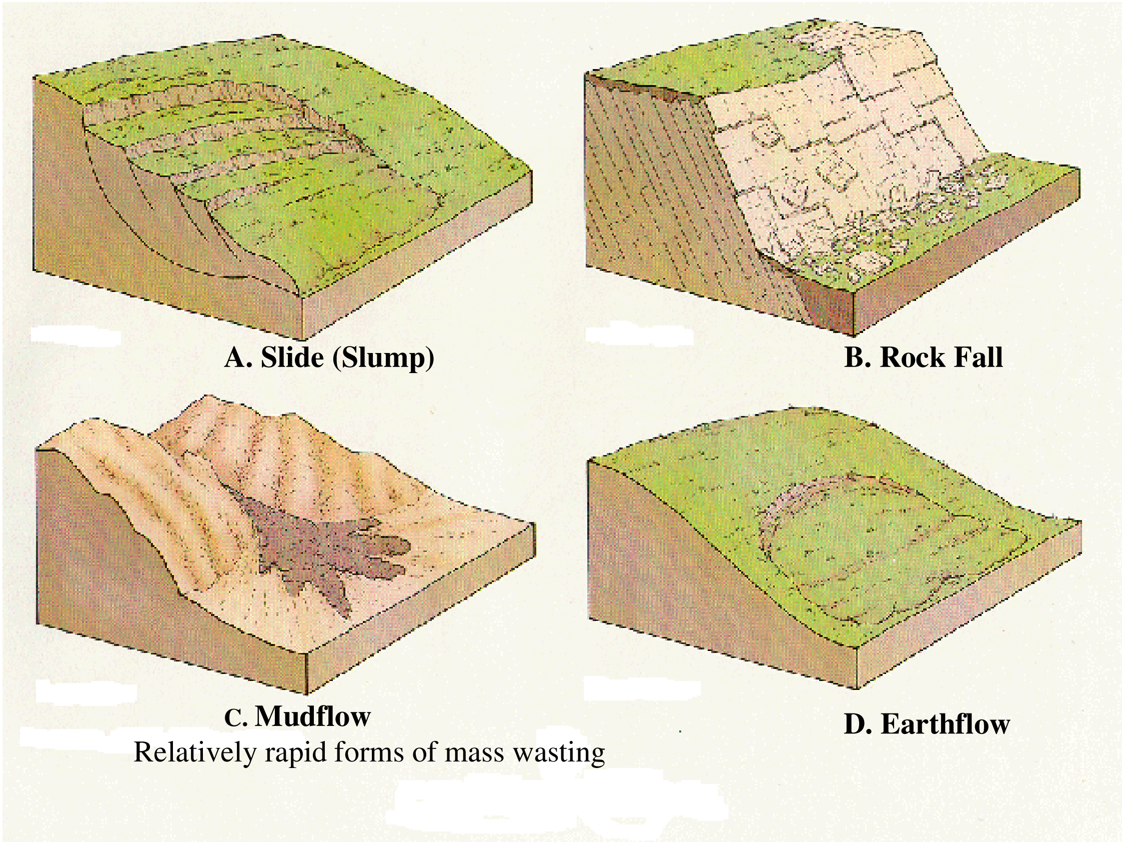


Figure 2.1: Type of landslides

(Source: http://www.hbcumi.carl.edu/tqp/301-15/mass%20wasting.gif)

Landslides occur when a portion of a hill slope becomes too weak to support its own weight. This weakness is initiated when rainfall or some other sources of water increases the water content of the slope, reducing the strength of the materials. The raised water table after a rainstorm would saturate the soil rendering it weak and therefore the slope failure would naturally occur due to gravitational forces. It occurs in every state of the nation and its island territories. They become a problem when they interfere with human activity and result in damage to properties and loss of life. Landslides also caused major socio-economic impacts on people, their homes and possession, industrial establishment, and lifelines as reported by the U. S. Geological Survey (2001). They have damaged or destroyed roads, rail roads, pipe lines, electrical and telephone transmission lines, mining facilities, petroleum wells and production facilities, houses, commercial buildings, canals, sewers, bridges, dams, reservoirs, port facilities, airports, forests, fisheries, parks, recreation areas, and farms – Maharaj (1996) – see Figure 2.2 and 2.3. Much landslides damage goes undocumented because it is considered instead with its triggering event, and thus is included in reports of floods, earthquakes, volcanic eruptions, hurricanes, or coastal storms, even though damages from the land sliding may exceed all other costs.

Figure 2.2: House damage after landslide
(Source: U. S. Geological Survey, 2001)

Figure 2.3: Road damage because of the landslide
(Source: U. S. Geological Survey, 2001)

Therefore, an efficient and effective monitoring technique should be established in order to detect the rate of movement, size and the direction of the landslides. Several monitoring techniques are available such as geologic, geotechnical and geodetic. The classical methods of land surveys, inclinometer, extensometer and peizometer are still the most appropriate one (Popescu, 1996). Although there are lots of classical instrumentation and monitoring techniques, Global Positioning System has the potential in monitoring the landslides phenomenon continuously. The Geographical Information System is the best tool to create the landslide maps and databases in 1D, 2D and 3D view with several analysis within the context of GIS.

2.4 Types of investigation

Generally, there are various types of investigation and instrumentation being used in monitoring the landslide phenomena. The main investigations are geologic structure, surface deformation, ground water and geotechnical. According to Nakamura, et. al. (1996), the investigation of surface deformation is conducted to define the boundaries of the landslides, size, and directions of the movement. The instrumentation used for the surface deformation investigation includes extensometer, ground tiltmeter, movement determination by aerial photographs, and Global Positioning System.

The investigation of geological structure relies on exploratory borings, geophysical surveys and the evaluation of slide plane with the instruments such as pipe strain gauge, inclinometer and multi-layer movement meter depending on the requirements for surveying accuracy and magnitude of movement. Investigation of ground water includes ground water level, pore water pressure, ground water logging, ground water tracing test, geothermal survey, and geophysical logging as being discuss by Nakamura, et. al. (1996). Brennan (1999) and Kovari (1988) have pointed out that landslide studies can be organized into three phases;

1. Detection and classification of landslides,
2. Monitoring activity of existing landslides,
3. Material testing in laboratory, and
4. Analysis and prediction of slope failures in space (spatial distribution) and time (temporal distribution).

In many cases, particularly when huge masses are involved, the most important information are those provided by the geological and hydrogeological factors. The most frequently observed physical quantities in relation to movements are displacement (relative and absolute), strains and inclination.

Popescu (1996) pointed out that monitoring of landslides plays an increasingly important role in the context of living with and adapting with landslides. The use of instruments in field monitoring has grown a great deal in checking on the validity of design assumption, obtaining an early warning of the impending failure in the investigation and control of landslides – see Bhandari (1988). Ground investigation involves a combination of topographic survey, sub-surface investigation, laboratory testing, monitoring of ground movement and groundwater studies, which are accomplished with the view to assessing stability conditions and suitability of remedial options. The GPS and geotechnical measurement in landslide studies have been reported and discussed by many researchers such as Coe, et. al (2000), Gili, et. al. (2000), Yalçınkaya, M and Bayrak,T (2002a), U. S. Geological Survey, 2001, Corbeanu, et. al. (2000), etc. Krakiwsky (1986) pointed out that the monitoring techniques including geodetic survey systems, photogrametric system, Global Positioning Systems and geotechnical instrumentation. Geodetic survey provides distances, azimuths and angles for determining horizontal coordinates, and spirit leveling for the height coordinates. According to Kalkan, et. al. (2002), geodetic measurement should be carried out every three months interval whereas geotechnical should be carried out every month. A summary of the main methods and their precision is shown in Table 2.2.

Table 2.2: Overview of methods used in measuring surface displacement and their precision (Source: Gili, et. al., 2000)

Method	Results	Typical range	Typical precision
Precision tape	Δ distance	< 30 m	0.5 mm / 30 m
Fixed wire extensometer	Δ distance	< 10 –80 m	0.3 mm / 30 m
EDM	Δ distance	Variable (1 – 4 km)	1 - 5 mm + 5 ppm
Surveying triangulation	Δ X, ΔY, ΔZ	< 300 – 1000m	5 – 10 mm
Surveying traverse	ΔX, ΔY, ΔZ	Variable	5 – 10 mm
Aerial photogrammetry	ΔX, ΔY, ΔZ	Hflight < 500m	10 cm
Clinometer	ΔX	+/- 10º	0.01 – 0.1 cm
Precise geometrical leveling	ΔZ	Variable	0.2 – 1 mm / km
GPS	ΔX, ΔY, ΔZ	Variable (usual < 20 km)	5 – 10 mm + 1 – 2 ppm

Various geotechnical methods can be used for monitoring landslides such as wire extensometer, inclinometer, tiltmeter, peizometer, and Time Domain Reflectometry (TDR) – see Dunnicliff (1993). Peizometers allow the determination of water levels. Inclinometers and tiltmeters allow determination of the direction and rate, and failure plane location. Extensometer provide indicator of magnitude. TDR can locate failure plane depth. A case study that has been carried out after the 1998 El Nino storms of January and February, which have caused a large number of landslides in California, showed that TDR, vibrating wire peizometer, electrolytic bubble inclinometers and tiltmeter were available to monitor landslides – see Kane and Beck (1999) and Kovari (1988).

Surveying methods are used to monitor the magnitude and rate of horizontal and vertical deformation of the ground surface and accessible parts of subsurface instruments in a wide variety of construction situations. In general whenever geotechnical instrument were used to monitor deformation, surveying methods are also used to relate measurement to a reference datum. Photogrametry plays a major role among the geometric methods of displacement monitoring as discuss by Armenakis and Faig (1988). It has been widely used in monitoring landslides. This technique is an effective tool for monitoring landslides and for analysing the velocity or strain-rate fields. The photogrametric work done by Smith (1999) provided an accuracy of standard deviation of 0.44 m in horizontal position. Photogrammetric method allows one to use photographs to determine displacements over long periods of time. However, field surveys are generally more precise than phothogrammetric measurement – see Smith (1999).

Aerial photogrammetry provides point coordinates contour maps and cross-sections of the landslides. Photogrammetric compilation enables a quantitative analysis of the change in slope morphology and also the determination of the movement vectors as discuss by Gili, et. al. (2000). Powers and Chiarle (1999) reported that, they have developed an alternative digital method for making measurement from aerial photographs by using GIS software to determine the movement of the Slumgullion landslide. Level, theodolite, electronic distance measurement (EDM) and total station measurements provide both the coordinates and changes of target, control points and landslides features – see Gili, et. al. (2000).

As one of the latest surveying technology, it has been proven that GPS has substantial advantages compared with conventional terrestrial surveying techniques for landslide monitoring schemes to determine the boundary of landslide block and rate of land movement. Coe, et. al, (2000) reported that the GPS technique is a useful tool for detecting first stage disaster and further mitigation. It can detect movement of cm/yr, and help in determining the landslide areas. Monitors can be placed anywhere you can access, and they are relatively easy to operate.

Corbeanu, et. al.(2000) pointed that GPS has the potential to monitor landslides. In 1992, the Romanian Oil Company has lost more than 60 to 70 tons of oil per day because of these phenomena. Research programme has been carried out using Stop and Go solution with 5 to 10 minutes observation time at all the monuments in a single day. According to Gili, et. al. (2000), GPS allows a larger coverage and productivity with similar accuracy in landslide monitoring practice. Furthermore, it can work in all kind of weather conditions and a direct line of sight between stations is not required.

Gili, et.al. (2000) reported the performance of the GPS equipment by using Rapid Static (5mm + 1ppm) and Real Time Kinematics modes in the landslide of Vallcebre, Spain, they can achieve accuracy between 16 mm in horizontal plane and 24 mm in elevation for Real Time Kinematics modes and 12 mm in horizontal plane and 18 mm in elevation for Rapid Static modes. The GPS measurement result also has been compared with the result obtained with the inclinometer and wire extensometer. The result showed that the GPS measurement results were fit well within the inclinometer and wire extensometer. From the experiment, it shows that Rapid Static and Real Time Kinematics methods are the most productive methods available for determining single points with precision of millimetre or centimetres. Ashok (2001) pointed that the combination of the GPS data and GIS allows for greater capabilities than what GPS and GIS can provide individually. With the combination of the two technologies one is able to display the "FIELD/ACTUAL SITE" on a PC. There is no need to make a specific site visits or review several documents/drawings. In addition, another benefit of the integration is the fact that unlimited users in various departments for their own specific needs and analysis can share the data.

The rapid development of computer technology has made a great impact on the field of information system such as GIS - Abd. Majid dan Ghazali (1995). GIS is a computer-based system designed for storing, analyzing, updating, manipulating and displaying spatial data. GIS can be viewed from three perspectives-the map, the data and the spatial analysis. The map view focuses on the ability to create and present information in a cartographic manner. This enables the presentation of information in a visual manner and assists in building user knowledge. Data is an important component of GIS. It provides users with a tool to capture, manage, query and analyze data from various sources – see Ashok (2001).

Sarkar and Kanungo (2000) pointed out that with the help of GIS, it is possible to integrate the spatial data of different layers to determine the influence of the parameters on landslides occurrence. They have generated a landslide risk map using Arc view GIS software for Darjeeling Himalaya. Remote Sensing and GIS techniques play a significant role in landslides zonation mapping (Cheng, 2000, Ramakrishnan, et.al., 2000, Sarkar and Kanungo, 2000). Babu and Mukesh (2000) has pointed that the ability of GIS to present the data and analysis results in map forms plays a key role in identifying the critical areas by its interactive visualization in a spatially optimized mode. Mongkolsawat, et.al. (1994) reported that the Huai Sua Ten watersheds soil erosion has made study using the Universal Soil Loss Equation and GIS. The study confirms that the use of GIS and remotely sensed data can greatly enhance classes of areas where land is used for field crops with no conservation practice.

2.5 Examples of landslide tragedies in Malaysia

The rapid economic development in Malaysia has resulted in the construction of many new roads and buildings. There are several places that has been recognized by the Public Authority that have the potential of landslides, such as the East West Highway at Kuala Kenderong (Perak) and Jeli (Kelantan), Gunung Berinchang (Cameron Highlands), Genting Sempah (Karak Highway), Bukit Peninjau (Fraser Hill), Kampung Sungai Liu (Langat/Kelawang Road), Gunung Gagau (Kelantan) and Penampang (Keningau, Sabah) – see Bernama (2002). Tan (1996) has reported that some case studies were carried out on landslides activities on various project involving mining and ex-mining land, highways, hill side development and river bank instability at various parts of Malaysia. For example, hillside development in urban areas (Kuala Lumpur, Penang, etc.) is a topic of major concern in Malaysia, especially after a recent disaster occurred involving the collapse of a condominium block Highlands Towers in Kuala Lumpur on 11 December 1993, killing 48 people. The collapse was attributed partly to a series of retrogressive slides of a cut-slope located behind the condominium – see Tan (1996).

Other examples of landslide disasters which have occurred in Malaysia are the natural landslide tragedies at Pos Dipang, Kampar, Perak on 29 August 1996, killing 39 peoples. On January 1998, one was killed, after a concrete wall collapse at Km 308.8 of North-South Highway near Gua Tempurung, Kampar, Perak, on 8 February 1999, 17 pupils were killed at Kampong Gelam, Sandakan, Sabah, when part of the hill collapsed. The next tragedies were on January 2000, where 70 hectares farms were destroyed by landslides at Kampong Baru Ringlet, Cameron Highlands. On 27 December 2001, 5 were killed by mudslides at Kampong Seri Gunung Pulai, Johor. The latest phenomenon occured on 20 November 2002, where 8 people were killed when landslides destroyed one of the bungalows near Hillview Garden (Bernama, 2002). Figure 2.4 to 2.6 show pictures of the landslides which have destroyed residential housing area in Malaysia.

(a)

(b)

Figure 2.4: Landslide in Kampung Gelam, Sandakan, Sabah

(Source: Berita Harian Online)

Figure 2.5: The collapse of Block 1 Highland Tower Condiminium in 1993.
(Source: Berita Harian Online)

Figure 2.6: The tragic landslide in Taman Hillview in 2002
(Source: Berita Harian Online)

According to Marzita (2000), there are 149 areas along the North South Highway that have the potential of landslides phenomena. The result from the soil sampling of the ROM scale shows that from Tapah to Tanah Rata road at (km 47) and Grik to Jeli (km 27) is the critical areas. Bukit Antarabangsa, Genting Highlands, Kuala Lumpur to Karak Highway (km 63.8), Paya Terubung are the highest landslide areas, while Tapah to Tanah Rata road (km 47) and Bentung to Raub road (km 27) are the lowest landslides areas.

They used the rainfall analysis to determine the erosion risk frequency potential along North South Expressway using rainfall data from the rainfall station nearby the expressway. They found that the month of October has the highest soil erosion risk frequency compared to January which has the lowest. Amir, et. al. (1999), Roslan and Hui, (1999a) has carried out a preliminary study on rainfall and erosion in Cameron Highlands using rainfall data from 13 rainfall stations. Their study was based on the Soil Erodibility Factor, K in the Universal Soil Loss Equation (USLE). They found that the relationship between rainfall and erosion risk is a useful factor for erosion prevention.

2.6 Landslides investigation in Malaysia.

In Malaysia, numerous research projects on landslides have been done over the last thirty years. Initially the investigations were carried out by Geological Survey Department of Malaysia mostly to solve the instability problems on site using the geological methods. Tan (1988) has reported that the studies of landslides were done on two highway projects of Kuala Lumpur – Karak Highway and Ipoh – Changkat Jering Highway and two housing schemes namely Taman Melawati and the Wangsa Ukay. From the investigations he found that the failure of slopes is related to geological factors. Tan (1988) has concluded that the geology and engineering geological factors which can cause landslide occurrences are:

1. The soil-rock interface with its associated poundings of groundwater
2. Natural valley uphill from slope cut.
3. Weathering grades.
4. Unfavourable orientation of joints.
5. Severe erosion and gullying of unprotected slope face.

Askury (1999a) has carried out a site investigation at Bukit Besar, Kuala Terengganu area and has detected 10 landslide occurrences along the road to Telekom transmitter station. He has classified the landslides into two groups such as rotational and translational. Rainfall, cutting of slopes and geological factor are the major causes of the landslide occurrences. He has also reported the landslides occurrence at km 69.54, East West Highway, Gerik, Perak and found that the rainfall is the major cause – see Askury (1999b, 1999c). Mohd Azmer and Hamzah (1998) have reported on the investigation at Jeli, Kelantan. He also found that the geological factor, rainfall and cutting of trees are the major causes of landslides along the highway. Abd. Majid, et. a.. (1999) has reported that a number of landslides have occurred at the Bukit Antarabangsa area in Hulu Kelang, Selangor near the Wangsa Heights Condominium and the Athenaeum Condominium, where these landslides threatened the stability of the condominium. They have used surveying techniques to observe the landslide area. Geological observation based on site investigation was carried out and using the Probe Mackintosh test and hand auger hole were used to get the geological profile of the area.

Jasmi, et.al. (2001) has produced hazard zonation landslides map of Selangor using remote sensing and GIS techniques. He found that most of the high landslide hazard area is elongated along the hilly terrain in the eastern part of the state that covers part of Hulu Langat, Cheras, Ampang and Sg. Buluh. Jasmi (2000) has used the statistical approach using the Information Value Method and Weight of Evidence, which delineated the most hazardous zones with GIS environment. Nawawi, et.al. (1998) reported that the landslide maps for Raub, Pahang were produced using USLE formula with Regis GIS software. Roslan and Hui (1999c, 1999d, 1999e), have carried out investigation using the remote sensing images with a resolution of 30 meters on the USLE factors for Cameron Highlands.

2.7 Global Positioning System

The Global Positioning System (GPS), sometimes also called NAVSTAR (NAVigation System using Time And Ranging) is a space-based navigation system created and developed by the US Department of Defense (DoD) for real time navigation since the end of the 70's. For the past ten years, the GPS has made a strong impact on the geodetic world. The main goal of the GPS is to provide worldwide, all weather, continuous radio navigation support to users in determinining position, velocity and time throughout the world – see Hofmann-Wellenhof (1986). The GPS can be used for absolute and relative geodetic point positioning. A navigation system is usually designed to provide the user with the information of determining 3D-user's position, expressed in geodetic coordinate system latitude, longitude and altitude (ϕ, λ, h) and in Cartesian coordinate system (X, Y, Z).

GPS relative positioning has several advantages as a monitoring tool. Presently achievable accuracies rival conventional survey methods for regional surveys, and advances in instrumentation and processing methodologies will further enhance the utility of GPS for local networks. Unlike terrestrial methods, GPS positioning does not require station neither intervisibility, nor favourable light or weather conditions. Moreover, GPS methods are inherently three-dimensional, and are relatively insensitive to ground station geometry. Finally, with the deployment of the full satellite constellations, continuous and automated monitoring using GPS will become increasingly practical and cost-effective in many surveying and mapping applications, including deformation surveys.

GPS has provided approximately fifteen years of increasing services to military and civilian users in a wide variety of applications. A variety of federal, state, country and public sector organizations and their counterparts in other countries are establishing or planning to use network of GPS reference stations, utilizing both differential (pseudo-range) and carrier phase tracking techniques, for either real time navigation or post processed positioning. The use of GPS networks for research in the Earth and oceanic sciences has been well established for a number of years. The technical and operational characteristics of the GPS are organized into three distinct segments: the space segment, the operational control segment (OCS), and the user segment. The GPS signal, which are broadcast and the ground control facilities, link the segments into one system. Figure 2.7 briefly characterizes the signals and segments of the GPS.

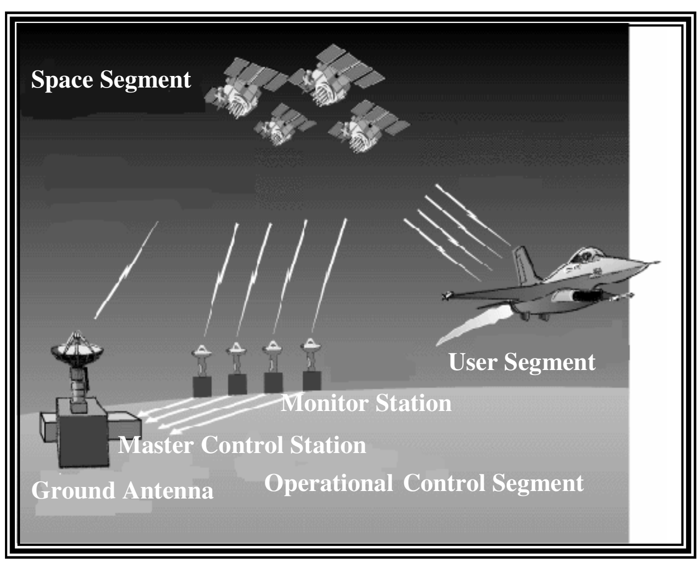


Figure 2.7: The GPS segments

The space segment consists of a complete constellation of 24 satellites (21 operational plus 3 active spares) arranged in six orbital planes where each plane is inclined by 55° relative to the equatorial plane at approximately 20,200 kilometers altitude above the Earth. Each satellite completes one orbit in one-half of a sidereal day (period of 12 hours) and, therefore, passes over the same location on earth once every sidereal day, approximately 23 hours and 56 minutes. With this orbital satellite configuration and the number of satellites, a user at any location on the Earth will have at least four satellites view at all time, i.e. 24 hours a day.

The ground control segment manages the performance of all satellites through clock monitoring and orbital tracking by using dual-frequency high power receiver with precise atomic clocks. The control parts are responsible for update and calculate satellites' ephemeris and satellite's clock correction and also, meteorological corrections. These corrections, together with other messages or information will then be transmitted or broadcast to all satellites and GPS users.

The user segment includes all equipment or elements to utilise the GPS satellites' signal in order to compute position. The most fundamental of this segment is the receivers which receive all the available satellites' signal at a moment to calculate and determine position, altitude and GPS time. Generally, the user segment can be divided into two parts: the military approaches and civilian users. Each GPS satellite transmits data on two L-band modulated frequencies which noted, as L1 (1575.42 MHz) and L2 (1227.60 MHz). These two L-band frequency signals are derived from the fundamental high accuracy atomic clock frequency of 10.23 MHz. Both L1 and L2 signals are used to determine distance between satellite and the receiver by measuring the radio travel time of the signals. The reason of transmitting two frequencies is to eliminate or determine the vector of refraction delays, caused by the earth ionosphere as well as atmospheric effects. The Course-Acquisition (C/A) code, sometimes called the Standard Positioning Service (SPS), is a pseudorandom noise code that is modulated onto the L1 carrier. The initial point positioning tests using the C/A code resulted better than the expected positions. The Precision (P) code, sometimes called the Precise Positioning Service (PPS), is modulated onto the L1 and L2 carriers allowing for the removal of the first order effects of the ionosphere.

A full exploitation of GPS relative positioning for monitoring applications will require a demonstrated capability for three dimensional 1 ppm or better relative positioning accuracies on local and regional scales, as well as practicable methodologies for the rigorous statistical testing and assessment of precision and reliability in GPS networks. Current GPS estimated accuracy ranges from 1-2 ppm for regional baseline vectors determined using commercial production software, to better than 0.1 ppm for transcontinental baseline vectors processed using sophisticated research packages. The packages employ orbital estimation techniques, dual frequency ionospheric correction, and water vapour radiometry – see Wells et al. (1986); Lachapelle et al. (1988). Antenna phase variation and other instrumental effects presently limit accuracy over short distances to 0.5-1.0 cm. Other limiting factors include the influence of the troposphere and ionosphere, orbit computation error, and the users ability to successfully exploit the integer nature of the carrier phase ambiguity, and to detect and repair cycle slips arising from loss of phase lock. Further improvements in receiver instrumentation, processing methods and orbital ephemerides are expected to greatly reduce these influences over the next five years, resulting in an ultimate estimated accuracy of the order of 2-3 mm + 0.01 ppm.

The complexity and number of observations involved hamper rigorous assessment of precision and reliability in GPS networks. Three limiting factors currently in evidence here:

1. The physical correlations arising among difference GPS observations due to orbital and atmospheric influences are not yet clearly understood.
2. Practicable methodologies for the detailed statistical testing and assessment of GPS observable, including outlier detection and internal and external reliability analyses are presently lacking, and
3. The large number of observations and parameters involved make the rigorous propagation of orbital uncertainty and mathematical and physical correlation difficult.

The resolution of the above problems will require a significant effort over the next several years to find mathematically tractable and computationally practical solutions.

2.8 Geotechnical Methods

Geotechnical methods are used extensively in the monitoring of landslide or structures. During the monitoring, geotechnical sensors of the desired type are carefully chosen and placed at strategic locations to ensure that adequate information is provided to verify design parameters, evaluate the performance of new technologies used in construction, verify and control the construction process for subsequent deformation monitoring. The main geotechnical sensors used for deformation monitoring include; extensometers, inclinometers, piezometers, strain gauges, pressure cells, tilt sensors and crack meters.

Geotechnical sensors can either store the measured data internally awaiting download, or the measurements can be automatically logged to a connected computer. Connection to a computer offers a number of advantages (e.g. data stored at a remote location; ability to change update rate of measurement data, when changes in measured values are detected; no need to visit site to download data) and disadvantages (e.g. transfer media required between sensor and computer, for example cable/radio/GSM; loss of data possible if transfer media is not operating and internal storage is not activated).

Geotechnical sensors provide measurements that are often essential in deformation monitoring. An additional sensor category that completes the portfolio of deformation monitoring sensors, that provide their own analysable measurements or measurements to calibrate additional sensors, is the meteorological sensors. Inclinometers have been used widely in the field of civil and geotechnical engineering to monitor soil movements and structural deformations. An inclinometer is basically an instrument, which measure the tilt in very high precision. Its main applications are, monitoring slope and landslide movements, monitoring subsurface lateral movements, inclinations and settlements of embankments, dams, open cut excavations, and structures. Most inclinometer systems have four major components:

1. A permanently installed guide casing, made of plastic, aluminum alloy fiberglass or steel. When horizontal deformation measurements are required, the casing is installed at a near vertical alignment. The guide casing usually has tracking grooves for controlling orientation of the probe.

2. A portable probe containing a gravity-sensing transducer.
3. A portable readout unit for power supply and indication of probe inclination.
4. A graduated electrical cable linking the probe to the readout unit.

The pipe may be installed either in the borehole or in fill, and in most applications is installed in a near-vertical alignment, so that the inclinometer provides data for defining subsurface horizontal deformation. Figure 2.8 shows the normal principle of inclinometer operation for the near-vertical guide casings. After installation of the casing, the probe is lowered to the bottom and inclination reading is made. Additional readings are made as the probe is raised incrementally to the top of the casing, providing data for the determination of initial casing alignment. The differences between these initial readings and a subsequent set define any change in alignment – see Dunnicliff (1988).

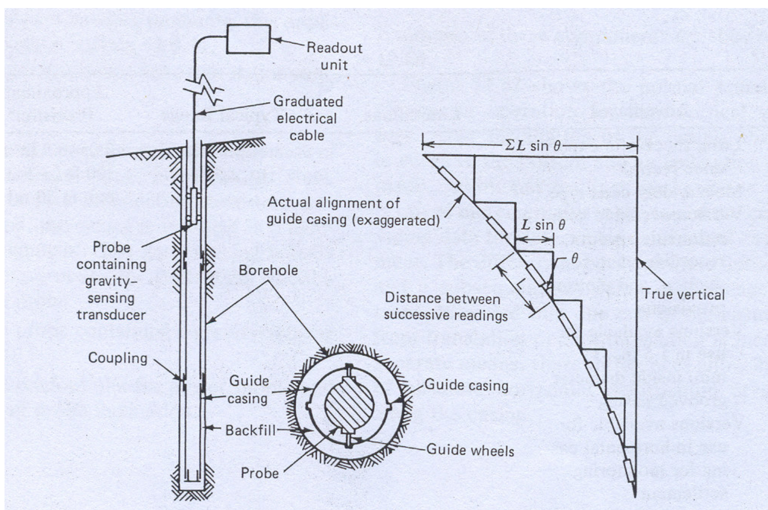


Figure 2.8: The principle of inclinometer
(Sources: Dunnicliff, 1988)

Inclinometer measurements are repeated every 50 cm through the vertical direction. As a result of this measurement, tilt changes can be determined at every 50 cm. These data are stored to the magnetic media during the measurement and than logged the computer for post-processing stages.

CHAPTER III

DEFORMATION ADJUSTMENT

3.1 Deformation Networks Design

The two remaining aspects of the monitoring scheme, which will be considered in detail, are the design and analysis stages. The network design is the first step towards establishing a deformation network. A network may be designed to fulfill some specific criteria, before any observations are actually made. In the specific case of a deformation monitoring network, the design may not only be required to meet precision (e.g. variances of point positions or derived quantities) and reliability criteria, but also has to be sensitive to the deformation pattern which is expected to take place. Since a postulated deformation model between two epochs of observations represented in effect a systematic difference between the two sets of measurements, the sensitivity assessment of a network can be regarded as being related to the detection of systematic errors – see Niemeier et al. (1982).

Many works can be done to design a network to ensure that it will achieve its desired aim, before any measurements are made. The basis on which the design can be carried out is seen from the form of the matrix equation, which gives the covariance matrix of the estimated parameters, namely;

$$C_{\hat{x}} = \sigma_o^2 (A^T WA)^{-1} \tag{3.1}$$

The only term in this equation which requires any observational data is σ_o^2, which can be assumed to be unity (1) for design purposes, since this will be its value once the weight matrix has been appropriately estimated. It therefore follows that all precision estimated based on $C_{\hat{x}}$ may be computed without any observation being made, provided the intended positions of the observation stations are approximately known and the measurements which are to be made have been identified. This situation represents the most usual design problems, that is, to decide where to position observation stations and which measurements to make in order to satisfy defined (precision) criteria. However, this is only one of four orders of design, which are commonly defined as follows.

1. Zero-order design (A, W, fixed; x, $C_{\hat{x}}$ free). The datum problem; the choice of an optimal reference system for the parameters and their covariance matrix. A common solution is to obtain the minimum trace of $C_{\hat{x}}$. If estimable quantities are used in the definition of the required precision criteria, then the solution to this order of the design problem is immaterial.
2. First-order design (W, $C_{\hat{x}}$ fixed; x, A free). The configuration problem; this is the situation whereby the station positions and observational scheme are designed to required a good accuracies and precision.
3. Second-order design (A, $C_{\hat{x}}$ and x fixed; W free). The weight problem; this is the determination of the observational accuracies required for a given scheme to meet defined precision criteria.
4. Third-order design ($C_{\hat{x}}$ fixed; *A, W* and x partly free). The improvement problem; really a combination of first and second-order design, required for example when strengthening an existing network to meet improved precision criteria.

Usually the design required not only needs to solve the problem of meeting precision criteria, but must also be the minimum-cost solution, often referred to as the optimum design. This introduced cost element can be very difficult to quantify, but possible designs are usually assessed subjectively taking regard of previous experiences. Once the design problem has been formulated, there are two basic approaches to the solution. Firstly, and most commonly, there is the computer simulation, or pre-analysis method, whereby proposed networks are analysed in turn to see whether they meet the required criteria, being subjectively modified by operator intervention, and using his experience, if the proposed scheme is either too strong or not strong enough. This method has been successfully used on many engineering projects, but it has the drawback of possibly (or even probably) missing the optimum solution. In contrast, the analytical approach attempts to mathematically formulate the design problem in terms of equations or inequalities and then to explicitly solve for the optimum solution. This latter method has so far been of only limited success, but is still being developed.

Finally, when considering the design for deformation analysis, it is important to take into account the sensitivity of the resulting network to the particular deformation expectation. This is because the purpose of such a network is usually not only to detect possible movements, but also to try and to establish the general mechanism of the motion taking place. In other words, it is required to test theoretical models of deformation against the result of the network analysis.

3.2 Network Adjustment

All points in the monitoring network are tied to each other by a combination of observable such as coordinates, elevation, etc. The numbers of observation usually exceeds the minimum number required to determine the unknown parameters. The method of least square estimation (LSE) is an important tool in estimating the unknown parameter from redundant data. Generally, the functional model relating to the measurements and parameters to be estimated can be expressed as:

$$l = f(x) \tag{3.2}$$

where l is the vector of observations and x is the vector of parameters to be estimated. In general, equation (3.2) is non-linear, and it needs to be linearized by using Taylor's theorem. After linearization the observation equation is written as:

$$\hat{v} = A\hat{x} + b \tag{3.3}$$

where $\hat{v}$ is the vector of the residuals, A is the design matrix, $\hat{x}$ is the vector of corrections to the approximate values (x_o) and b is the misclosure vector.

The normal equation with a full rank can be written as:

$$N\hat{x} + U = 0 \tag{3.4}$$

where:

$$\hat{x} = -N^{-1}U = -(A^T WA)^{-1} A^T Wb$$

$\hat{x}_a = \hat{x} + x_o$, the updated parameters

$Q_{\hat{x}a} = \sigma_o^2 (A^T WA)^{-1}$, cofactor matrix of $\hat{x}_a$

σ_o^2 = a priori variance factor

$b = l - l_o$

l = vector of actual observation

l_o = vector of computed observation

$W = \sigma_o^2 Q_l^{-1}$, the weight matrix

$\hat{\sigma}_o^2 = \dfrac{\hat{v}^T W \hat{v}}{n - u}$, a posteriori variance factor

n = number of observations

u = number of unknown parameter

Other important aspects that need to be considered are global test (Chi-square) and local test (TAU), precision, accuracy and reliability (internal and external) analysis. After the data for each were verified to be free of outliers and have a high degree of reliability, then the deformation analysis can be carried out.

3.3 Geometrical Analysis

In deformation surveys, one of the problems frequently encountered in practice is the stability of the reference geodetic points in the process of determining the absolute displacements of the object points. Any unstable reference points must be identified. Otherwise, the calculated displacement of the object points and the subsequent analysis and interpretation of the deformation bodies may be significantly distorted or wrongly interpreted. The "GPS-revolution" (Hofman-Wallenhof, 1986) affects surveying and analysis of deformation even more than other fields in geodesy. Measurement sessions lasting anywhere between a few days up to several weeks result in highly accurate and consistent coordinate differences between points of interest. The geometrical analysis of deformation surveys involve the reliable determination of changes in the geometrical status of a structure over time – see Chrzanowski et al. (1986). Such analysis is of particular importance when a structure is to satisfy certain geometrical conditions such as verticality, alignment or conformity to a design surface. They are also required in the physical interpretation of deformations for the confirmation and refinement of predictive models of the load-deformation response. In most cases, the analysis of deformations proceeds from repeated observations taken at distinct epochs. Two types of monitoring networks that can be distinguished are:

1. Absolute networks, in which a subset of points are assumed to remain stable between epochs, and
2. Relative networks, in which all points in the network, are subject to motion. Deformation between subsequent epochs can be inferred directly from a comparison of raw observable, or indirectly from changes in coordinates.

The principal advantage of the raw observation approach is that the solution is independent. A direct comparison of observations also assists in the reduction of systematic effects to both epochs, since the same observations and network geometry must be maintained throughout. A similar bias reduction effect can be advised through a simultaneous adjustment of the observations common to both epochs in a direct solution for displacements. Single epoch adjustment allow some leeway for variations in observation scheme between epochs, thus making it possible to utilize all available information in the solution, but introduces the problem of datum dependence. In either case, the use of adjustments has an important advantage that an evaluation of the quality of the observations can be undertaken, and an opportunity for the detection of random outliers and systematic effects are provided. However, it requires that the network is completed and free from configuration defects at each epoch, and that each epoch is referred to a common datum.

In basic approaches to geometrical analysis, the displacements at discrete points are directly compared with specified tolerances. In more advanced analysis, the point displacements are assessed for spatial trend, and a displacement field is determined by the fitting of a suitable spatial function. The displacement field may then be transformed into a strain field, which provides a unique description of the overall change in geometric status, by the selection of a suitable deformation model – see Chrzanowski et al. (1986).

Coordinate differences between two points obtained from GPS measurements depend on the orientation of the GPS datum specifically on the orientation of the coordinate system inherent in those coordinate differences. The effect of disturbances in orientation of the order of micro radians on measured GPS-height differences is significant and for points 10 to 20 kilometers apart it may reach intolerable magnitudes. In this paper we propose a model for the combined adjustment of GPS and spirit-leveling measurements which can filter out such biases is proposed. The results of employing such a model are unbiased orthometric height velocities which are the primary objective of any vertical deformation surveying.

The analysis of a network observed for the monitoring of deformations will initially consist of a conventional analysis with appropriate tests for the detection of outliers, and the computation of the estimated parameters and associated covariance matrix and unit variance. However, a monitoring network will be repeatedly measured at various epochs, and a comparison of successive network adjustments must be carried out in some way in order to detect any deformations, which have taken place. It is essential to realize that the straightforward difference in the two sets of coordinates does not in itself provide sufficient information to assess whether points have moved or not, since some consideration must be given to the accuracy with which the coordinates have been determined. The same magnitude of difference at two stations may represent a significant movement in one case and not in the other. Any analysis must therefore be carried out in conjunction with all the available accuracy estimates. There are many approaches to this problem, several of which are described and compared in Chrzanowski and Chen (1986). However, before proceeding to give details of techniques, it is necessary to differentiate between two-epoch and multi-epoch analyse. This may be done rigorously by using a multi-epoch analysis, or subjectively by inspecting the cumulative results of successive, rigorous, two-epoch analysis. Since the latter is less complex, it will be addressed first.

3.4 Two-epoch analysis.

The starting point for a two-epoch analysis will be the results of two single-epoch adjustments, namely

$$\hat{x}_1, \hat{x}_2; W_1, W_2; Q_{\hat{x}_1}, Q_{\hat{x}_2}; \hat{\sigma}^2_{o_1}, \hat{\sigma}^2_{o_2}$$

where;

$\hat{x}_1, \hat{x}_2$ is the vector of corrections to the approximate values, W_1, W_2 is the weight matrix, $\hat{\sigma}^2_{o_1}, \hat{\sigma}^2_{o_2}$ is a posteriori variance factor, and $Q_{\hat{x}_1}, Q_{\hat{x}_2}$ is the cofactor matrix of $\hat{x}$ for epoch 1 and epoch 2, respectively.

The aim of the analysis is to identify stable reference points in the network (if any), and to detect single-point displacements which will later be used to aid in the development of an appropriate deformation model. It is necessary to stress how crucial is the detection of outliers in each of the single epoch adjustments, since errors which escape detection are likely to be assessed as deformations later in the analysis. The reliability of deformation monitoring networks is therefore of most importance.

The first stage of a two-epoch analysis of an absolute network is to assess the stability of the reference points by assuming them to form a relative network and testing whether any points have moved. This may be achieved by carrying out a global congruency test. Similarly in analysing a relative network, the first step is usually to establish whether any group of points in the network has retained its shape between the two epochs, again by using the global congruency test. If such a group can be identified, then these points may be used as a datum, thus providing an absolute network for the analysis of the other stations. If in either case no group of stable points can be identified, then the resulting relative network must be assessed only in terms of datum invariant criteria.

3.5 Test on the Variance Ratio

The test on the variance ratio examines the compatibility of the independent variance factors of the two epochs. The test can either be one-tailed or two-tailed, with the former, as shown below:

$$H_0 : \hat{\sigma}^2_{oi} = \hat{\sigma}^2_{oj} \text{ at significance level } \alpha \tag{3.5}$$

$$H_a : \hat{\sigma}^2_{oi} > \hat{\sigma}^2_{oj} \text{ or } \hat{\sigma}^2_{oj} > \hat{\sigma}^2_{oi}$$

where $\hat{\sigma}^2_{oi}$ and $\hat{\sigma}^2_{oj}$ are the estimated variance factors of epochs i and j. Let their respective degrees of freedom become df_i and df_j. The test statistic is in the form of a ratio of the variance factors:

$$T = \hat{\sigma}^2_{0j} \Big/ \hat{\sigma}^2_{0i} \sim F_{df_j, df_i} \tag{3.6}$$

assuming j and i refer to the larger and smaller variance factor respectively.

Their relevant degrees of freedom become df_i and df_j. The outcome of the one-tailed test on variance ratio is:

if $T < F_{df_j, df_i, \alpha}$, test passes, accept H_0

if $T \geq F_{df_j, df_i, \alpha}$, test failed, reject H_0

If H_0 accepted, indicating the two variance factors are statistically equivalent, the variance ratio test is passed and the pooled variance factor $\hat{\sigma}^2_o$ may be computed as:

$$\hat{\sigma}^2_o = [(\hat{\sigma}^2_{oi})(df_i) + (\hat{\sigma}^2_{oj})(df_j)] \Big/ df \tag{3.7}$$

where $df = df_i + df_j$

If H_0 reject, it indicates improper weighting of observations, and requires the examinations of observational data or the adjustment results.

3.6 Stability Determination by Congruency Test

From the results of the two single-epoch adjustments it is possible to calculate the displacement $\hat{d}$ and the associated cofactor matrix $Q_{\hat{d}}$ from

$$\hat{d} = \hat{x}_2 - \hat{x}_1 \tag{3.8}$$

$$Q_{\hat{d}} = Q_{\hat{x}_2} + Q_{\hat{x}_1} \tag{3.9}$$

(assuming $\hat{x}_1$ and $\hat{x}_2$ are uncorrelated), and in addition the quadratic form given by

$$\Omega = \hat{d}^T Q_{\hat{d}}^{-1} \hat{d} \tag{3.10}$$

The displacement vectors (equation 3.8) and its cofactor matrix (equation 3.9) need to be transformed from minimum constraint datum to other datum definitions (i.e. the partial minimum trace of minimum trace datum):

$$d_1 = [I - G(G^T WG)^{-1} G^T W\}d = Sd \tag{3.11}$$

$$Q_{d_1} = SQ_d S^T \tag{3.12}$$

where: I = identity matrix

d = displacement vector (equation 3.9)

S = S-transformation matrix

W= weight matrix (with diagonal value of the one for datum points and zero elsewhere)

The full components of matrix G^T for a 3-D network is given as:

$$G^T = \begin{bmatrix} 1 & 0 & 0 & 1 & 0 & 0 & \cdots & 1 & 0 & 0 \\ 0 & 1 & 0 & 0 & 1 & 0 & \cdots & 0 & 1 & 0 \\ 0 & 0 & 1 & 0 & 0 & 1 & \cdots & 0 & 0 & 1 \\ 0 & z_1 & -y_1 & 0 & z_2 & -y_2 & \cdots & 0 & z_n & -y_n \\ -z_1 & 0 & x_1 & -z_2 & 0 & x_2 & \cdots & -z_n & 0 & x_n \\ y_1 & -x_1 & 0 & y_2 & -x_2 & 0 & \cdots & y_n & -x_n & 0 \\ x_1 & y_1 & z_1 & x_2 & y_2 & z_2 & \cdots & x_n & y_n & z_n \end{bmatrix}$$

where x_i, y_i, z_i are the coordinates of point P_i that is reduced to the center of gravity of the network. The first three rows of the inner constraints matrix (G^T) takes care of the translations in the x , y and z direction, while the third row defines the rotation of the x, y and z axis and the last row defines the scale of the network. It is readily shown in Caspary (1987) that a suitable test of the hypothesis that the points under consideration have remained stable, i.e. F(d) = 0, is

$$T = \frac{\Omega}{h\hat{\sigma}_o^2} \tag{3.13}$$

where h is the rank of $Q_{\hat{d}}$ *(3n-d)* for a 3D network

$\hat{\sigma}_o^2 = (r_1\hat{\sigma}_{o_1}^2 + r_2\hat{\sigma}_{o_2}^2)/r$

$r = r_1 + r_2$

r_i = degrees of freedom in the adjustment of the i^{th} epoch.

T is tested against the Fisher distribution (F-test) with $F_{h,r}$ at an appropriately chosen level of significance. If the test is successful (the hypothesis is not rejected), then the two epochs are assumed congruent, i.e. the points involved have remained stable. If the test is unsuccessful, at least one point has moved, and must be removed from the group of reference points. Several methods exist for identifying which point (or points) should be removed. The simplest of these methods is to identify the point, which has the greatest contribution to Ω. This point is then eliminated from the reference group and the global congruency test repeated. The process is repeated until a stable group of points is identified.

Having determined, by means of the global congruency test, a group of points, which have remained stable, it is now necessary to calculate coordinates for these stations, as well as for the other unstable points. There are different solutions to this problem. Firstly, it would be possible to adopt the first epoch estimates for the stable group and use these in a computation of the second epoch observations. However, this is not sensible since the measurements between the stable points in the second set are being ignored. It is also not entirely reasonable to adopt the separate estimates x_1 and x_2, since this would result in stable points having changing coordinates. The most preferable solution is to carry out a *combined adjustment* of the observations from both epochs, with only one set of unknown coordinates being estimated for the stable group of points, and two (one for each epoch) being estimated for the moving points. In fact, the required solution may be obtained without actually carrying out the combined solution since the displacements and covariance matrix of the unstable points can be obtained directly from the information available from the single epoch solutions (Caspary, 1987). The difference in the resulting coordinates for the moving points, (namely the displacements), together with the associated covariance matrix, can then be used in an assessment of the significance of the detected movements.

The movement $\hat{d}$ calculated for an unstable point can be tested for significance by comparing it with the appropriate elements ($Q_{\hat{d}_i}$) of the associated covariance matrix. The test statistic, which is most commonly used, is;

$$T = \frac{\hat{d}_i^T Q_{\hat{d}_i}^{-1} \hat{d}_j}{\hat{\sigma}_o^2} \tag{3.14}$$

and it is tested against the Fisher distribution $2F_{2,r}$ at the chosen level of significance. In a similar fashion to the computation of absolute point error ellipses it is possible to compute a point displacement ellipse by using the appropriate sub-matrix of $Q_{\hat{d}}$ in place of the sub-matrix of $Q_{\hat{x}}$. This ellipse may then be plotted along with the displacement vector for a graphical representation of the significance of the movement. Figure 3.1 below shows a 90% confidence ellipse. In this example, the movement is significant at the 10% level, since the displacement vector extends outside the ellipse. The size of a given (1 - α) percentage confidence ellipse is obtained by multiplying the axes of the standard ellipse (obtained by the procedure described above) by $\sqrt{2}$ $(F_{2,r})_{\alpha}$.

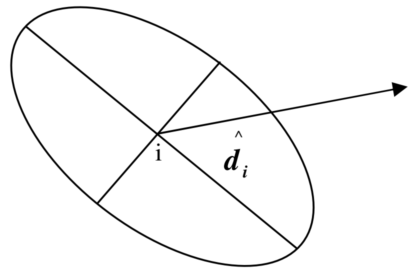


Figure 3.1: Point displacement ellipse

CHAPTER IV

RESEARCH METHODOLOGY

4.1 Introduction

The methodology of this research has been divided in four phase. The first phase includes the preliminary investigation such as field reconnaissance, topographic investigation and collection of existing data. Second phase is the network design and monumentation of the landslide area. Third phase will include the strategies in spatial data acquisition and processing. Fourth phase is to modeling techniques for landslide assessment. The flow chart shown in Figure 4.1 describes the general investigation procedures in an attempt to understand the mechanism of origination of disasters associated with slope movement.

4.2 Phase one

Preliminary study of is the first step and most important work to be done, to identified the areas that prone to landslide activities. Information data such as collection of existing data (history of the problem, records of restoration work, and data review) have been done, in order to understand the topography, geology, and properties of similar landslides. It is also important to understand their relationship with meteorological factors, period of activity, existence of any warning sign, ground water conditions, chronology of topographic change or erosion, and other factors which may have a relationship with the slope deformation surrounding the investigation site prior to the detailed investigation.

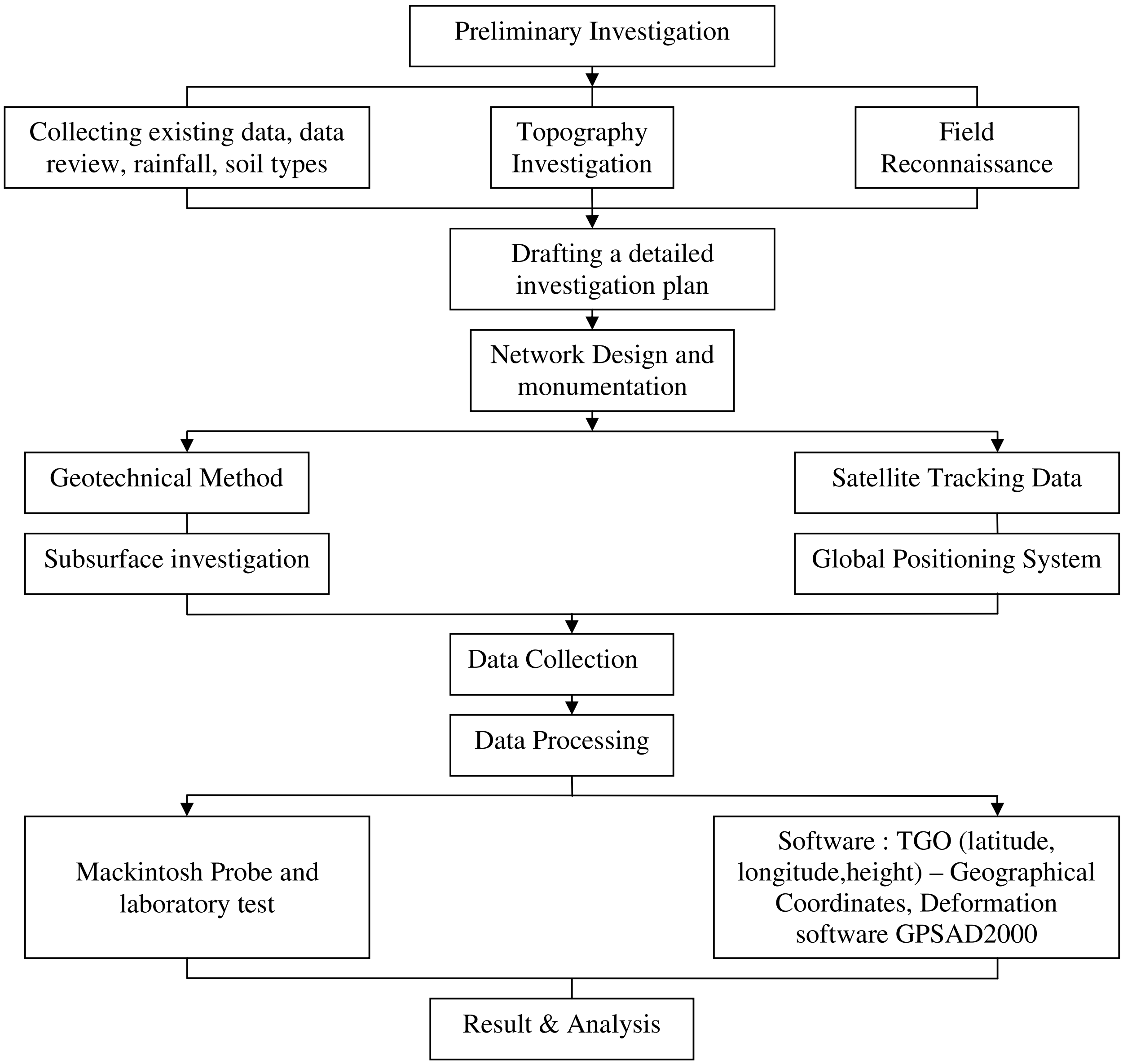


Figure 4.1: Flow chart of the operational framework

The acquisitions of data for the preliminary investigation have been divided into three major sections as follows:

1. Desk Study and Site Reconnaissance
2. Existing Subsurface Investigation
3. Selection of Types of field Tests and Sampling Methods.

4.2.1 Desk Study

The desk study includes review of the following information:

1. Geological Maps and memoirs

   Reviewing geological maps and memoirs together with an understanding of the associated depositional process can enable a preliminary assessment of the ground condition to be made.

2. Topographical Map

   Use the topographic map to examine the terrain, access and site condition. The topographic map should be checked through site reconnaissance.

3. Site History and Details of Adjacent Development

   The knowledge of the site history like land use before the current development, tunnels, stream, the number of landslide cases and others information. All of this information is useful to design the monitoring network and data acquisition planning.

The purpose of site reconnaissance is to confirm information obtained in desk study and also to obtain additional information from the site. The information includes examining adjacent and nearby development for tell-tale signs of problems and as part of the pre-dilapidation survey. Site reconnaissance allows us to study the sky view of the site for designing the monitoring network. It is also important to choose a suitable location for planting the monitoring monument at the site.

4.2.2 Existing Subsurface Investigation

In this study, an existing borehole at the site was found. From the borehole data, the type of soils at the location can be recognized together with the geological formation and features at the site.

4.2.3 Selection of Types of field tests and sampling methods

The selection of types of field tests and sampling methods should be based on the information gathered from the desk study and site reconnaissance. In this research we choose the Light Dynamic Penetrometer (Mackintosh Probes) was choosen. This Mackintosh probe is usually used in preliminary site investigation and to identify subsoil variation between boreholes particularly in the area of very soft soils. This method is also effective in identifying localized soft or weak material or slip plane. Figure 4.2 show the set of Mackintosh probe.

Figure 4.2: Mackintosh Probe

(Source: http://www.intec.com.my/product/soilequip/soil.html)

Beside the above investigations, we also have done the laboratory test to determine soil classification, chemical and mechanical properties was also done – see Table 4.1.

Table 4.1: Laboratory Testing

Soil Classification Test	
1	Particles size distribution – sieve analysis (for content of sand and gravels) and Hydrometer tests (for content of silt and clay)
2	Atterberg limit:- Liquid limit, plastic limit and plastic index (to be used in Plasticity Chart for soil classification)
3	Moisture content
4	Unit weight
5	Specific Gravity

Mackintosh probe log holes have been done at the study area at 5 selected monitoring stations: WM3, WM10, WM13, WM21 and WM29 stations, respectively. Figure 4.3 and 4.4 shows our team do the Mackintosh probe job.

Figure 4.3: The Mackintosh probe at WM 29 station

Figure 4.4: The Mackintosh probe at WM 3 station

At the same time, some disturb soil sample at all the 5 points was collected. Figure 4.5 show the team collected the sample of soil. As a comparison, the laboratory test for the entire soil sample was also done. The laboratory tests have been done in two methods. The laboratory test equipments are show in Figure 4.6.

Figure 4.5: The collection of disturb soil sample at WM 3 station

Figure 4.6: The instruments for laboratory soil test.

The soil sample was first placed in a tray (Figure 4.7) and later put them in the oven for 24 hour to dry the soil (Figure 4.8). After one day, the soil was taken out from the oven (Figure 4.9) and Atterberg limit test was carried out. The dried soil was then seived (Figure 4.10). As shown in Figure 4.11, the dry soil that has been sieved weighed on an electronic weighting machine. The weight of the soil was recorded. Usually about 200 gm of soil sample was taken for the liquid limit test. After that, the soil was mixed with water (Figure 4.12) and the liquid limit test was then carried out using the penetrometer equipment (Figure 4.13).

Figure 4.7: Putting the sample on the tray

Figure 4.8: Putting the tray into the oven to dry the soil

Figure 4.9: Taking out the dry soil from the oven

Figure 4.10: Sieving the dry soil

Figure 4.11: Weighing the soil before the test

Figure 4.12: Mixing the soil with water

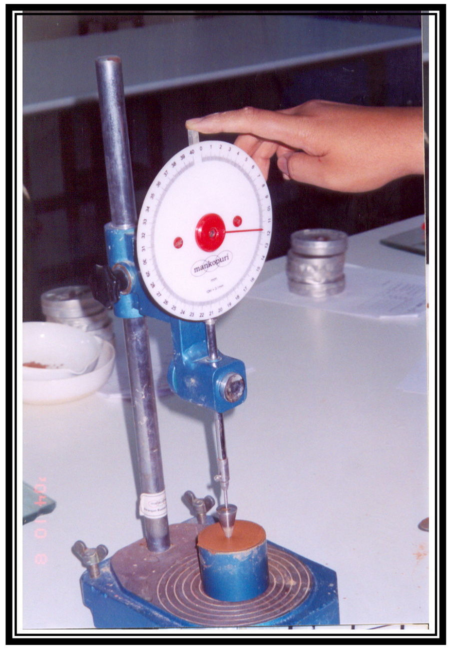


Figure 4.13: Cone panetrometer

Then the plastic limit test was carried out (Figure 4.14, 4.15, 4.16 and 4.17). In this test, about 20 gm soil sample was taken and mixed with water and mixed thoroughly. The sample was formed into a ball and the ball was cut into four parts. Each part was rolled on the glass plate until it reaches a diameter of 3 mm and later broken up into segments. The segments then placed in a small container and weighed and the weights were recorded. The container and the sample was put in the oven to dry. After one day, the container was taken out from the oven and put them on the electronic weighing machine and the weight recorded.

Figure 4.14: Mixing the soil with water

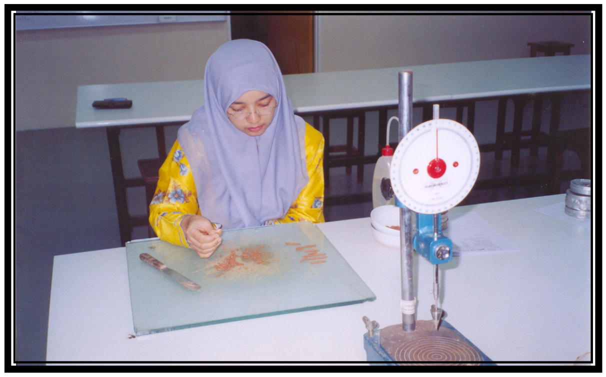


Figure 4.15: Cutting the soil into parts

Figure 4.16: Placing the soil in a small container

Figure 4.17: Weighing the container

4.3 Phase two

Carry out a network design is the second step towards establishing a geodetic network. A geodetic network is defined as a geometric configuration of three or more control survey points that are connected either by geodetic measurements or by astronomical or space-based techniques. In order to prevent the whole operation from failure, the surveyors/engineers should know the results at the work according to the pre-set objectives before any observation process is started. Here, the network design will answer the essential questions of where the network points should be placed, how many control points should be established, and how does a network should be measured in order to achieve the required accuracy with optimum cost. Figure 4.18 shows the monitoring network for the site area. In order to detect the stability of the slope, a series of GPS and geotechnical observation have been carried out in two epochs. Before such a observations could begin, it was necessary to set up a deformation network consisted of selected reference stations (datum) and the monitoring (object) points with respect to the corresponding engineering slope design, which suited to GPS and geotechnical investigation.

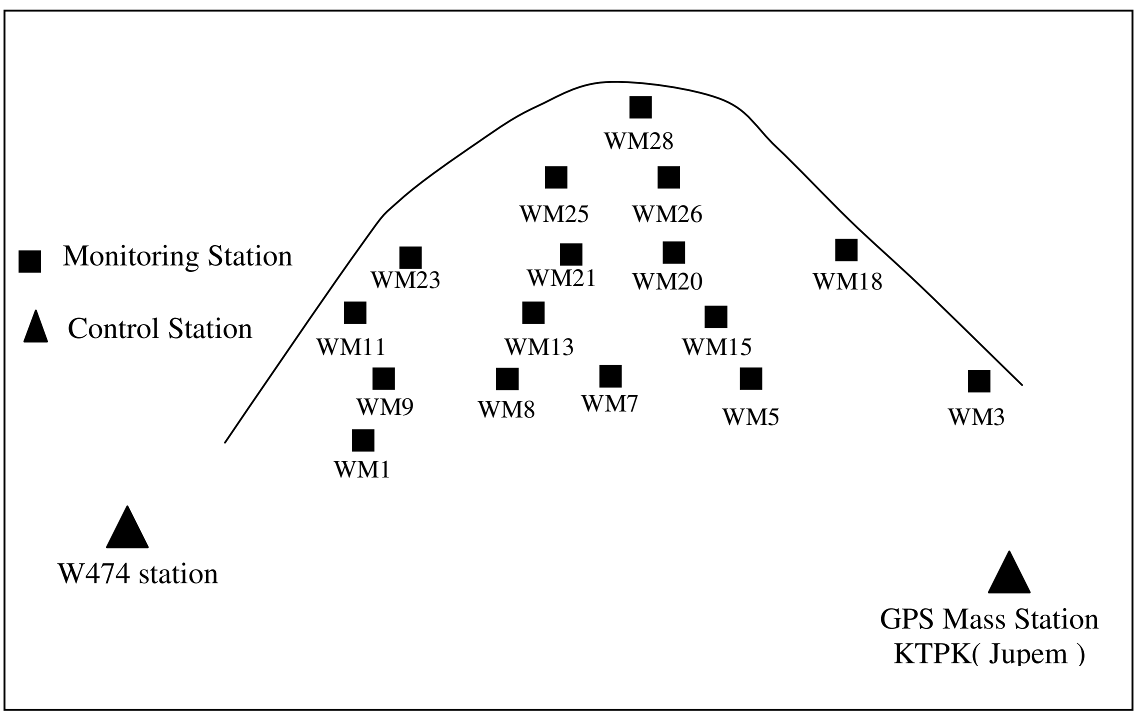


Figure 4.18: The configuration of monitoring network

The investigation of surface deformation is conducted to define the boundaries of the landslide, size, level of activity and direction(s) of the movement, and to determine individual moving blocks of the main slide. The presence of scarps and transverse cracks are useful in determining whether the potential for future activity exists. After the site reconnaissance, the monitoring network is designed with two control points and 16 monitoring points. The two control points are KTPK, on top of JUPEM building and W474, Taman Melawati, Hulu Klang, Selangor (Figure 4.19 and Table 4.2). The control points are located about 5 to 8 kilometers from the site area.

Table 4.2: Coordinate for the GPS control stations

Station	Latitude	Longitude	Ellipsoidal Height (m)
KTPK	3° 10’ 15.441”	101° 43’ 3.35363	99.267
W474	3° 13’ 9.497”	101° 44’ 20.848”	69.030

Figure 4.19: Station W474

Next process is the designing of the monument at the site area that minimizes the cost, man power and the strength of the monument. From the literature review, it was found that the suitable monument for the landslide is a stand pipe with 120.00 cm in length and 2.25 mm diameter. The design of the monument can be seen in Figure 3.20. After the monument design stage, 11 locations were chosen to plant the monument with sky-view clearance for GPS observation. The monument was planted on December, 2003. The process of planting can be seen in Figure 4.21 to 4.24.

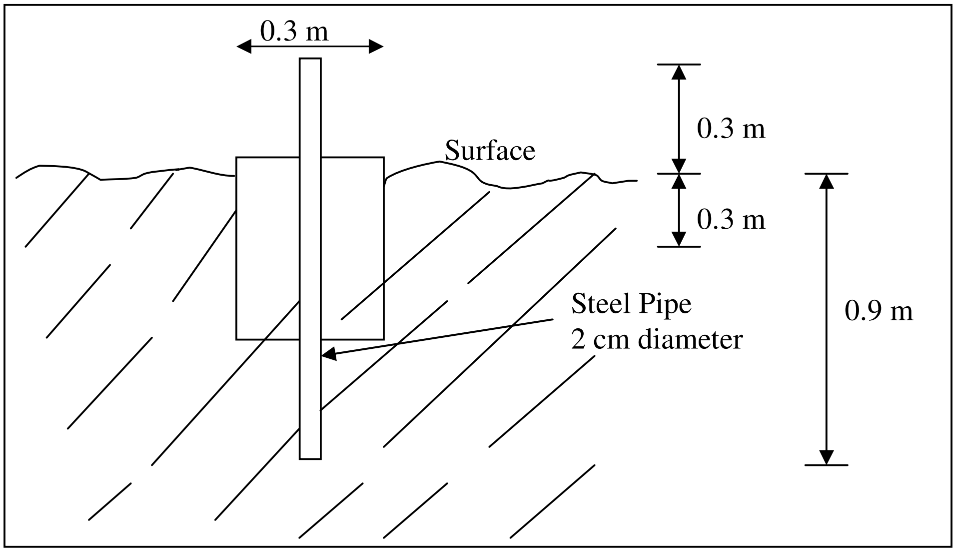


Figure 4.20: The monument design

Figure 4.21: Marking of WM3

Figure 4.22: Marking of WM11

Figure 3.23: Plastering the monument with concrete

Figure 4.24: Monuments ready for observation.

4.4 Phase three

In this research two units of *Topcon GPS Legant_E* single frequency receiver have been used (Figure 4.25). This Topcon GPS equipment gives an accuracy of 0.5 mm + 1 ppm for the horizontal component and 0.6 mm + 1 ppm for the vertical component.

Figure 4.25: *Topcon GPS Legant_E* Single Frequency Receiver

The observation for single frequency receivers was made in January 2004 – known as 1st epoch and the other measurement was performed in February 2004 – known as 2nd epoch. The monitoring network is consisted of selected reference stations and the object (target) points surrounding the area and object of interest. In this study, observation at 16 monitoring stations and two control stations, namely KTPK and W474 were carried out at two epochs with an interval of three months for both GPS and geotechnical methods. The GPS observation data were processed using the Pinnacle version 1.0 software for the Geographical coordinate (latitude, longitude and ellipsoidal height), the Cartesian coordinates (X, Y and Z) and the vector of Cartesian coordinate (ΔX, ΔY and ΔZ).

Before the observation, the plan for the observation strategy was made to obtain an efficient process, with optimum cost. First of all, it has to know the satellite geometry, numbers of satellite and the DOP (*Dilution of Position*) of every time observations were carried out. The good DOP is below 8.0. For example, Figure 4.26 shows the DOP of satellite on the 22 January 2004 and the satellite *Availability* at the location of latitude 3° 12' and longitude 101° 44' (Figure 4.27). The observations started with the single frequency receiver using the Rapid Static mode. Figure 4.28 shows the single frequency receiver at W474 station. To make sure that the GPS data is a very high quality data we have to do the job consistent and systematic. GPS observation using the single frequency receiver is shown in Figure 4.29, 4.30, and 4.31 respectively.

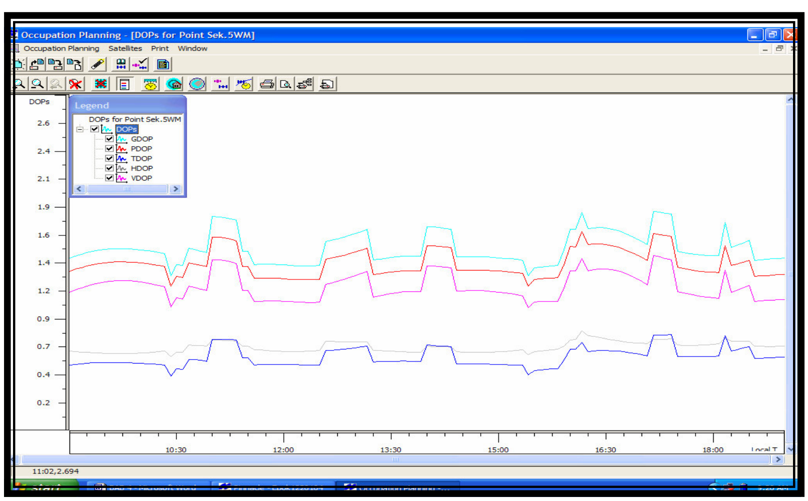


Figure 4.26: Satellite *Dilution of Position* on 22 January 2004

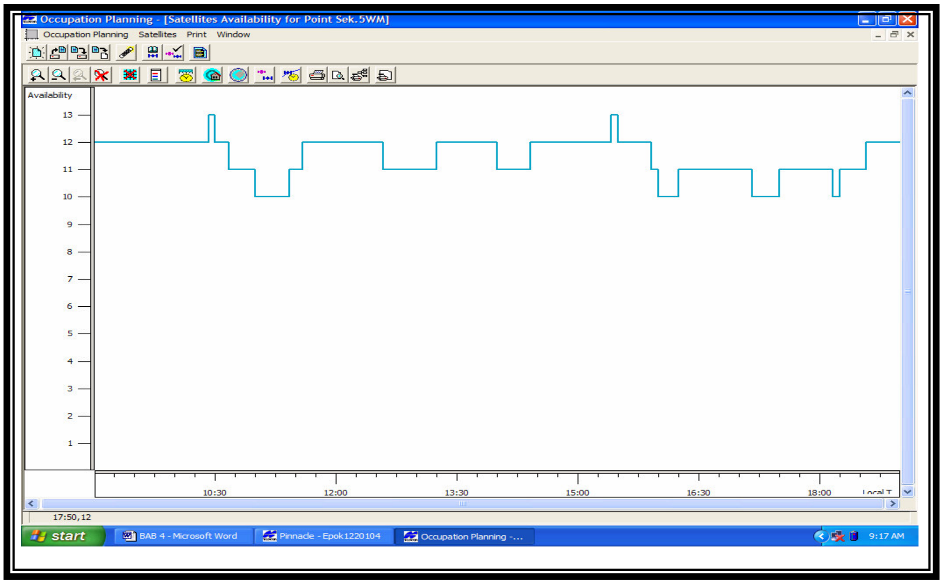


Figure 4.27: Satellite *Availability* on the 22 January 2004

Figure 4.28: Single frequency GPS receiver at W0474 station

Figure 4.29: Observation at WM2 station

Figure 4.30: Observation at WM3 station

Figure 4.31: Observation at WM5 station

4.5 GPS Network Adjustment

The GPS network adjustment of data from both epochs is accomplished using the Pinnacle version 1.0 and Starnet Pro. The displacement analysis data from the GPS constraint network adjustments are computed and summarized with respect to the GPSAD2000. In GPS survey, sets of two GPS receiver are usually used to give a coordinate for a large number of points. Basically, in order to estimate the geodetic parameters of interest, the appropriate modeling of the GPS observable and the development of processing strategies are done in post processing mode, session by session procedures. The entire network adjustment is carried out by the outcome of post processing as an observation (input parameters).

There are different types of adjustments that can be performed in order to obtain the most probable values of estimated parameters. Typically, the type of adjustment used for GPS survey is a minimal constraint and constraint adjustment. A minimal constraint adjustment is performed by holding the minimum required stations fixed to acquire a solution. Through this adjustment procedure, one station will be fixed horizontally and another vertically. A minimally constraint three-dimensional network adjustment is performed in order to investigate the internal quality of the GPS survey (data) without the contamination from external factor such as unreliable control coordinates. Furthermore, analysis of control ties because this will help to determine how well they tie in relation to each other. There are many minimally constraint solutions possible but the adjusted observation, a posteriori variances of unit weight and covariance matrices for the residuals are the same (Leick, 1990).

The constraint adjustment approach gives the best fit of the GPS network to local control. Thus, for constraint adjustment with minimum of 3 stations held fixed, the translations, scale and rotations of the local system can be determined with respect to GPS coordinates system. This would be appropriate if the aim of the GPS survey was in fact for geodetic control densification. The danger is that any errors in the control stations will lead to a distortion of the relatively high quality GPS results to fit the framework defined by the control system – see Rizos (1993).

CHAPTER V

DATA, RESULT AND DISCUSSION

5.1 Introduction

The first stage of data collection is from the desk study and the existing subsurface investigation. From the desk study, it was found that the geological structure of this area can be divided into two categories which are the metamorphic and plutonic stones. The metamorphic stones consist of the Wall schist and Hawthornden, whereby the plutonic stones is Kuala Lumpur granite – see Zulkarnain, (1997) and Jasmi, (2000). Figure 5.1 shows the geological map of Kuala Lumpur area and Figure 5.2 shows the lithology of the Wangsa Maju area.

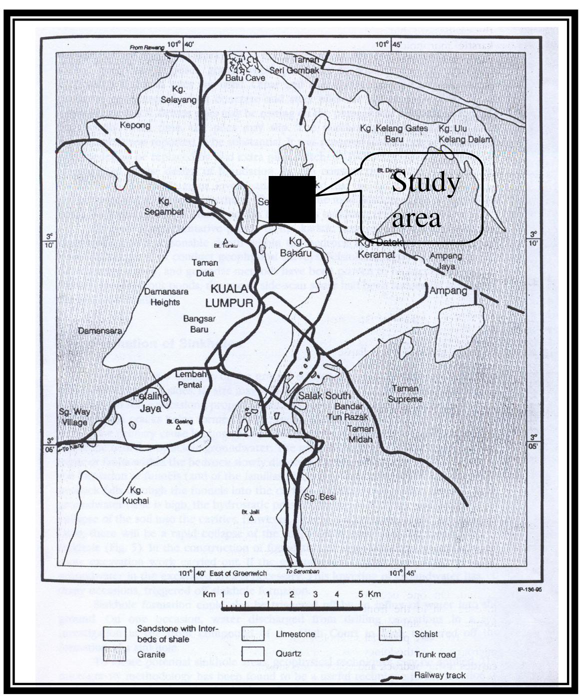


Figure 5.1: Geological Map of Kuala Lumpur (Source: Chow, 1995)

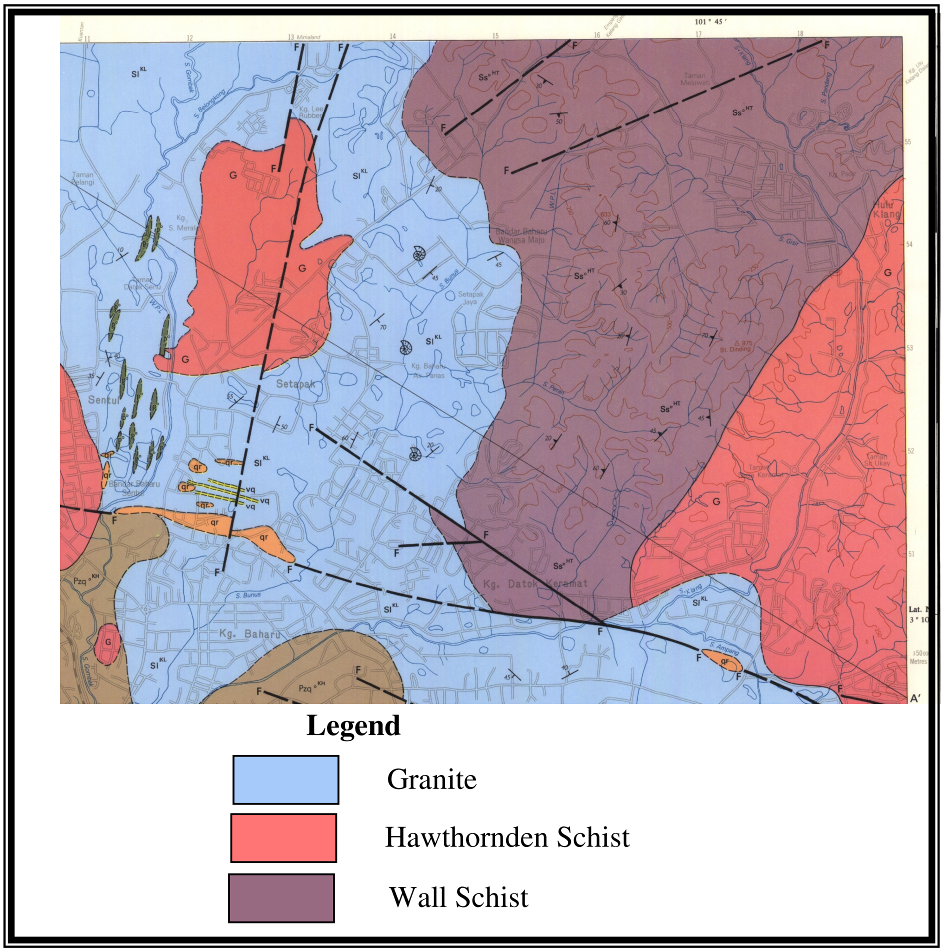


Figure 5.2: The Lithology of Wangsa Maju area.

(Source: Jabatan Penyiasatan Kajibumi, 1975)

According to Gobbett (1964), the Wall Schist is borne since pre-Silurian age, whereby, the Hawthornden Schist is borne in Silurian age and Kuala Lumpur granite is from Triassic age. Mohd Fauzi (1992) said that the Wall Schist consists of mica quartz and metavolcan at the middle of Wangsa Maju. The East of Wangsa Maju consist a lot of granite, which is at medium size ranging from 0.5 to 5 cm.

5.2 The Distribution of Rainfall Data

The daily rainfall data may show the landslide. Therefore, the rainfall data needs to be analyzed to the distribution of rain trend daily, monthly and yearly. According to Md. Kamsan Hamdan (Utusan Malaysia, 2002) and Mohd Izranuddin (2004), rain is one of the major factors that can change the structure of the sand. If the total rainfall is 2,000 mm per year, it can cause a landslide.

Therefore in this research, a four years record of rainfall from 2000 to 2003 has been analyzed to see the trend of rain at the study area. All of these data have been collected from three rainfall observation stations from the Department of Hydrology, Jabatan Pengairan dan Saliran (JPS), Cawangan Ampang. The stations are JPS Ampang station, JPS Wilayah Persekutuan Kuala Lumpur dan Empangan Kelang Gate station. Figure 5.3 show the location of three rain observation stations which are located around 5 to 10 km from the study area.

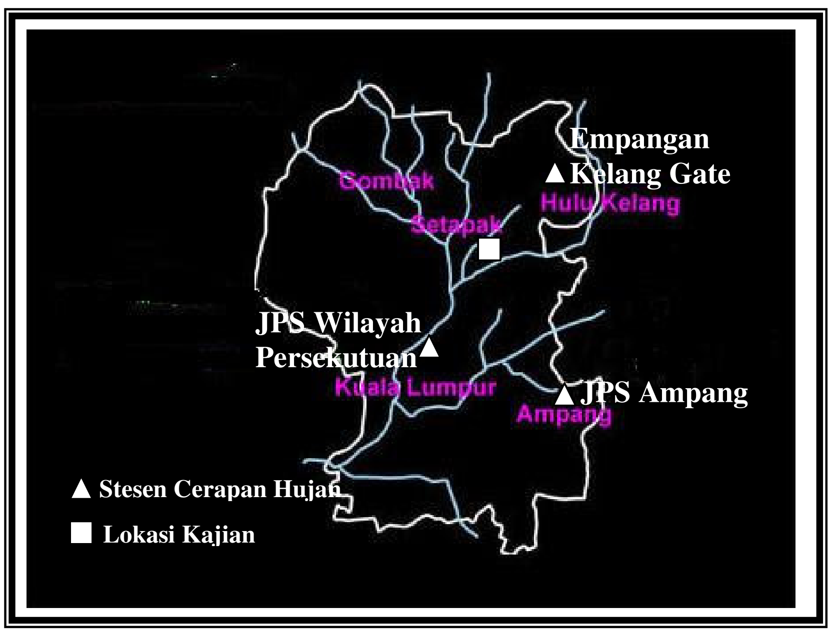


Figure 5.3: The location of the rainfall observation stations in Kuala Lumpur

The distribution of rainfall for the four years from year 2000 to 2003 at all three observation stations is shown in Table 5.1. Table 5.2 shows the yearly rainfall distribution from 1993 to 2003 at the three stations. Figure 5.4, 5.5 and 5.6 shows the graphic of the rainfall distribution based on the monthly distribution of the four years observation. Figure 5.7 shows the yearly distribution for the three stations. Based on Tables 5.1 and 5.2, Figures 5.4, 5.5, 5.6 and 5.7, the heavy rain always happened in April to May and October to November every year. This is because of the changes of South West monsoon season from April to May and South East monsoon season in October to November.

Table 5.1: The rainfall distribution for all stations from year 2000 to 2003

Station	Year	Monthly Rainfall Distribution (mm)											
		Jan	Feb	Mac	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Pejabat JPS Wilayah Persekutuan	2000	37.7	286.8	514.3	314.2	118.5	216.5	68	266.5	303	314	392.5	481.5
	2001	309	118	168.5	275	129.5	179.5	149	113.5	298.5	253	253	180
	2002	81	71	152.5	360.1	212.4	292	77.2	159.8	250.4	276.2	577.1	276.6
	2003	144.5	276.5	318	348	141.5	209.5	234	224	180.5	165	374.5	195
Empangan Kelang Gate	2000	128.9	167.1	404.5	431.3	210.1	181.5	159.4	313.5	280	184.5	369	351.9
	2001	219.5	123	153.5	423.5	154.5	172.5	116.5	98	439.5	446.5	209	180.5
	2002	19.5	110	265.5	182.5	162.5	263.5	120.5	216.7	231.2	364.4	449.4	244.4
	2003	90.5	86.5	327	255	119.5	226	299.5	229	238.5	161.5	369	137.5
JPS Ampang	2000	216	380	405.4	425.4	121	190.5	59	243.5	396.5	356	475.5	390.5
	2001	270	88.5	286	575	130.5	285.2	119.5	141.5	281.5	298	190	127
	2002	18.3	79.5	420.6	362.8	264.7	224.7	136.4	79.3	322.7	218.9	282.9	314.4
	2003	118.2	149.3	328.2	296.5	172.5	201.8	245.3	175	150	199.2	442.9	87.4

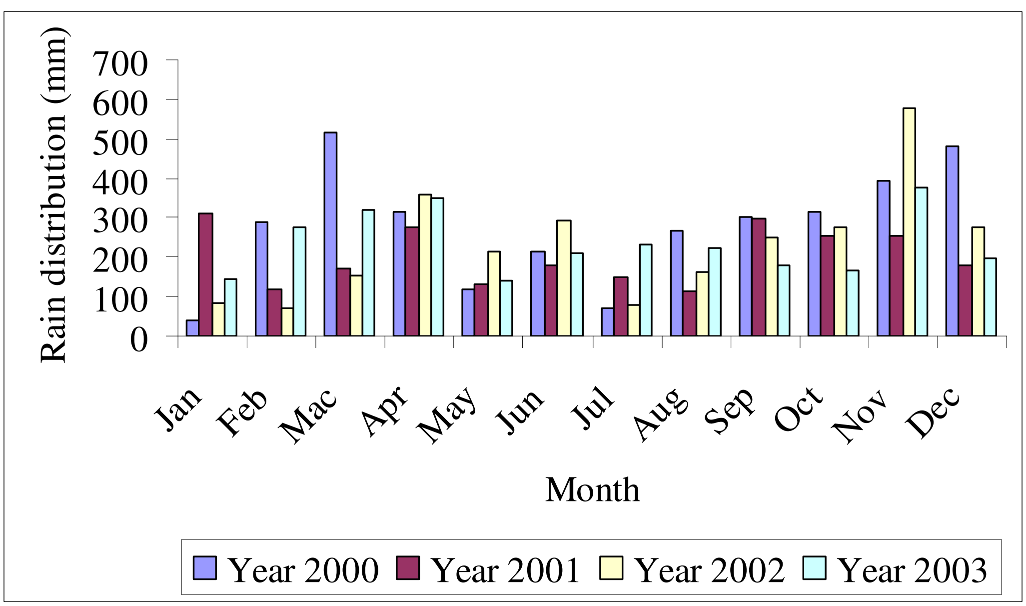


Figure 5.4: The monthly rainfall distribution at JPS Wilayah Persekutuan station.

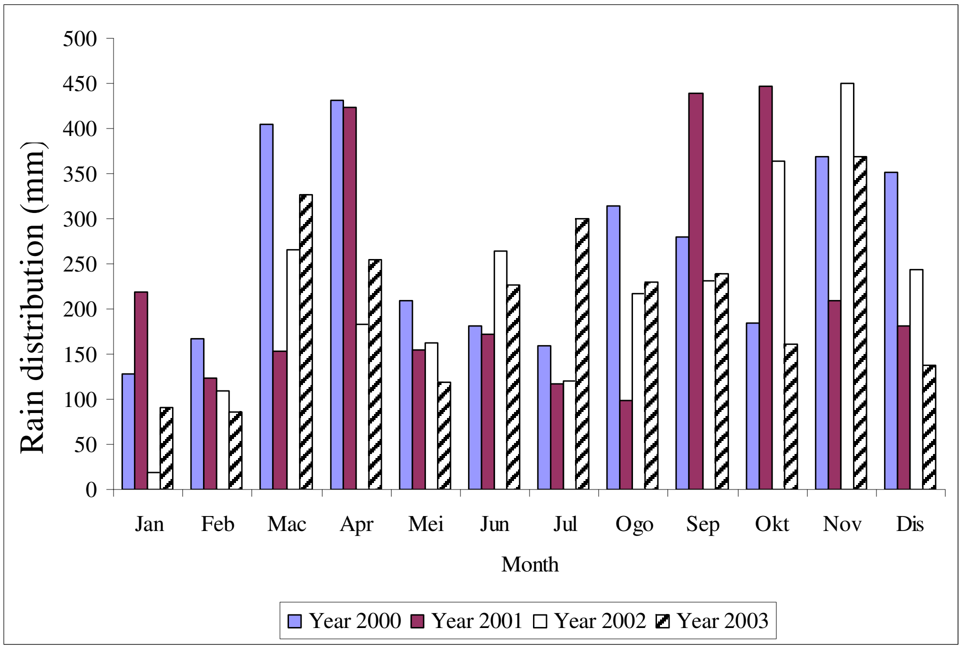


Figure 5.5: The monthly rainfall distribution at Empangan Kelang Gate Station

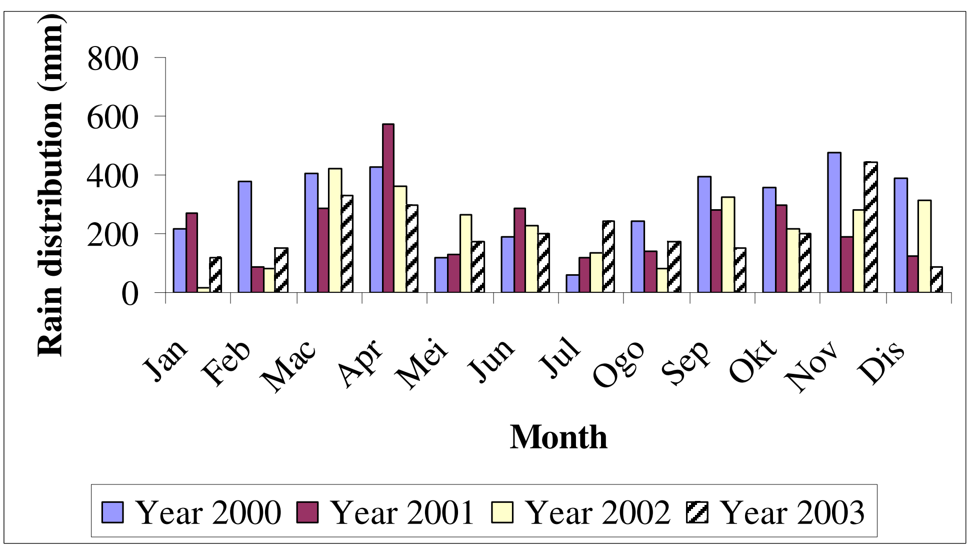


Figure 5.6: The monthly rainfall distribution at JPS Ampang station.

Table 5.2: Yearly rainfall distribution for the three observation stations from 1993 to 2003.

Year	Yearly Rainfall Distribution (mm)		
	JPS Wilayah Persekutuan	Empangan Kelang Gate	JPS Ampang
1993	2486	2115	2601.5
1994	2976.5	2162	2260
1995	2441.5	2733	3080
1996	2913	2315	3110
1997	2858.5	2726	2816
1998	2661.5	2070.5	2702
1999	2934.4	2978.2	3449
2000	3313.5	3181.7	3659.3
2001	2426.5	2736.5	2792.7
2002	2786.3	2630.1	2725.2
2003	2811	2539.5	2566.3

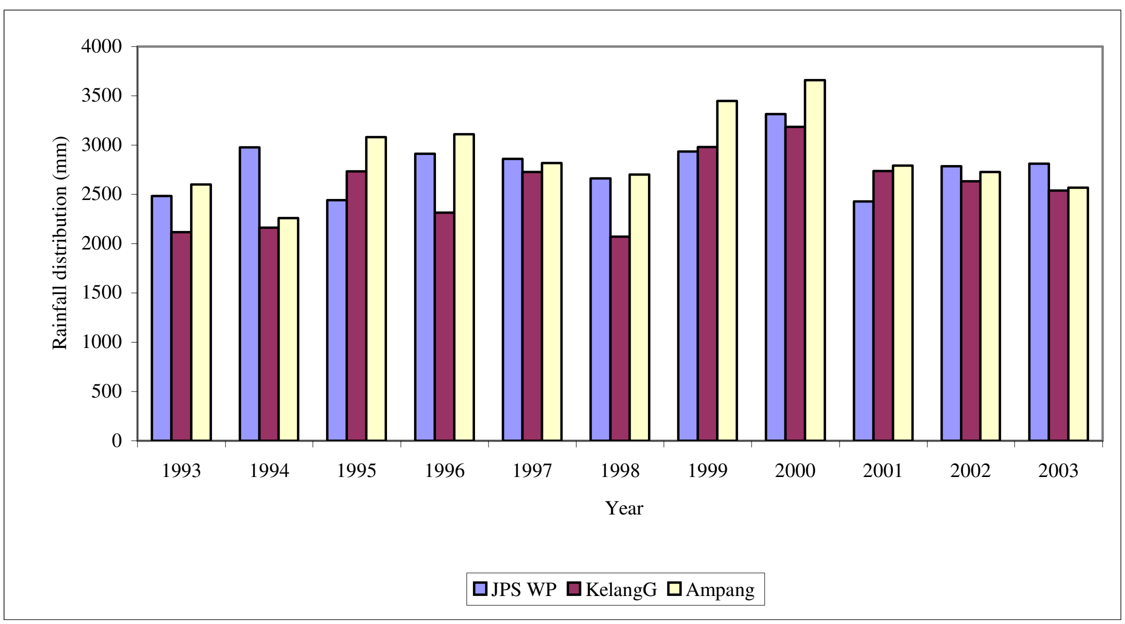


Figure 5.7: Yearly rainfall distribution for the three observation stations from 1993 to 2003.

The weekly rainfall distribution for the months of April, May and June 2001 is shown in Table 5.3 and Figure 5.8. This weekly rainfall distribution is plotted to see the picture of the weekly rainfall on that month in year 2001 which caused the landslide at the study area. From Table 5.3, JPS Ampang station recorded the highest reading (28.6 mm) on the third week of April. Whereby, Empangan Kelang Gate recorded the highest reading (27.8 mm) on the second week of April. All the three stations recorded the highest reading on the first week of June; 27.3 mm at JPS Ampang station, 12.4 mm at JPS Wilayah Perkutuan station and 15.8 mm at Empangan Kelang Gate station.. This was proven by the landslide investigation report by the developer whereby two landslides have occurred on the third week of April and first week of June 2001. Based on the discussion, it shows that heavy rain is one of the major factors at the study area.

Table 5.3: Weekly rainfall distribution in 2001.

Week	**JPS Wilayah Persekutuan**			**Empangan Kelang Gate**			**JPS Ampang**		
	April (mm)	**May (mm)**	**June (mm)**	**April (mm)**	**May (mm)**	**June (mm)**	**April (mm)**	**May (mm)**	**June (mm)**
1	10.7	6.3	12.4	12.1	7.1	15.8	9.5	3.4	27.3
2	12.4	6.6	0	27.8	7.2	0.4	20.2	4.9	1.4
3	7.0	0	3.4	8.1	2.3	6	28.6	0	3.6
4	9.0	3.4	9.9	8.3	4.0	2.4	23.2	9.7	5.0
5	0.3	7.7	4.7	14.5	3.5	0	2.0	1.7	12.2

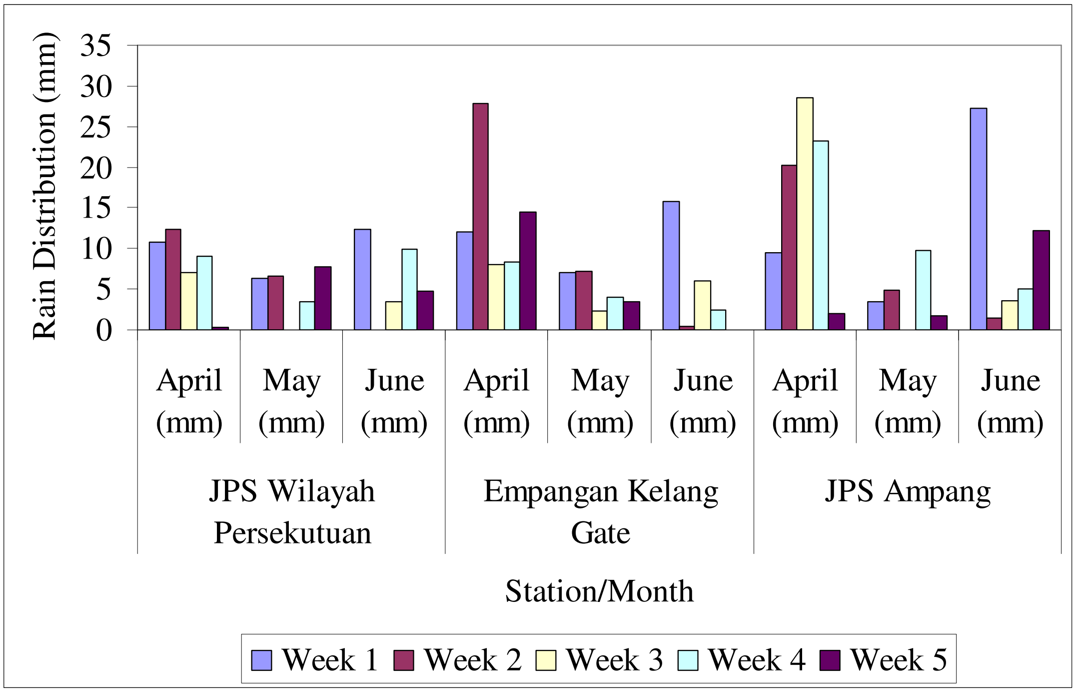


Figure 5.8: Weekly rainfall distribution at the three stations for April, May and June 2001

5.3 The Geotechnical Data Analysis

The geotechnical analysis is based on the existing soil investigation report in January 2001 and the land investigation that had been carried out in August 2004. The investigation had been divided into two; field work and laboratory work. The field work consists of boring, standard penetration test, soil sampling, under ground water survey and Mackintosh probe work. The laboratory work included the plastic limit, atterberg limit, and the soil moisture content.

The first field works was carried out by the developer from 2 April 2001 to 22 of April 2001. 16 boreholes were bored at the Section 5 hill side. Borehole no. 3 is (Figure 5.9) inside the study area and two boreholes namely BH 4 and BH 6 are outside the study area. The boreholes for BH 3, BH 4 and BH 6, are shown in Appendix A, B and C, respectively.

Figure 5.9: Borehole BH 3

It has also been proved by the laboratory test that the plasticity limit and liquid limit on the Atterberg graph was located at the sandy clay with the liquid limit of 55% to 63%, whereby, the plasticity index was 20% to 28%. Example of the laboratory test is shown in Appendix D and E. The sieve and hydrometer analyses also show that percentage of sandy clay was more than 50%, sand was 25% to 50% and clay was less than 20%. All the three log shows that the stratum of top soil at the study area is yellow brownish color and contains sandy clay and mineral substance about 1 meter high. The medium soft layer which was a black brown color and sandy clay has been founded at the depth of 1 meter to 13.5 m for BH 3, 1 to 5 meter for BH4 and 1 to 7 m for BH6. Hard layers for the three logs are at the 13.5 m depth for BH3, 5 m for BH4 and 7 m for BH6. The laboratory result also shows that the moisture content was between 15% to 35% for BH3, 21% to 30% BH4 and no record for BH6. This shows that the moisture content at this area was low.

A Mackintosh Probe work also has been done on the area. 5 Mackintosh probe log holes were carried out at the WM3, WM10, WM13, WM21 and WM29 stations. The probe blown at every 0.3 m and the number of blows recorded. The work ceased if the blow reached 400 blows at one time, to avoid it hitting hit a hard layer underneath it. All the data such as the probe height and the depth were recorded. The result showed that there was a hard layer at 8 m depth at the top of the study area and less than 5 m at the foot of the slope. Mackintosh result is shown in Appendix F.

Based on the laboratory test for the soil sample, it was found that the liquid limit for the study area varies from 38.8% to 56.4% and the plastic limit is between 23.6% and 41.8% - see Table 5.4. It shows that the study area is full of sandy clay soil.

Table 5.4: The Laboratory testing result for Atterberg limit at Section 5, Wangsa Maju

No. sample	Liquid limit test (%)	Plastic limit test (%)
WM 3 – 1	43.0	29.4
WM 3 – 1a	38.8	23.6
WM 10 – 2	40.8	28.3
WM 10 – 2a	44.1	31.1
WM 13 – 1	49.9	34.7
WM 13 – 1a	49.8	30.7
WM 21 – 1	43.4	32.2
WM 21 – 1a	48.4	41.8
WM 29 – 1	56.4	36.2
WM 29 – 1a	55.1	40.7

5.4 GPS Network Adjustment

After the entire observations using the single frequency receiver were obtained, the raw data was exported to the computer using PCCDU software as a controller for the GPS *Topcon Legant_E* receiver. From the PCCDU the data transferred to the Pinnacle version 1.0 software for the post-processing part. All the raw data were transferred to one fail for editing or to add new data. At the processing part, all vectors were processed and transfer to the StarNet Pro software for the network processing. The entire data was then separated into two files, namely Epoch 1 and Epoch 2. The data were processed based on the least square adjustment. First, this software detects all blunders and correction of the raw data was done. Then, the network adjustment was processed and one Dummy fail (*.dump) was created for the landslide detection part. This Dummy fail was transferred to GPSAD2000 using the Convert programme (Halim & Sharuddin, 2003).

The coordinates for the horizontal component of the first epoch and second epoch were shown in Table 5.5 and the vertical component in Table 5.6. Graphically the different in coordinates for both epochs are shown in Figure 5.10 and 5.11. Based on Table 5.5 and 5.6, the different of coordinate for the horizontal and vertical component vary from 0 to + 50 mm. From Table 5.5, the highest of coordinate different in the horizontal component was recorded at seven stations namely station WM5 (41 mm in Easting and 3 mm in Northing coordinates), station WM8 (30 mm in Easting and 60 mm in Northing coordinates), station WM18 (22 mm in Easting and 5 mm in Northing coordinates), station WM20 (15 mm in Easting and 36 mm in Northing coordinates), station WM23 (37 mm in Easting and 48 mm in Northing coordinates), station WM25 (21 mm in Easting and 40 mm in Northing coordinates) and station WM28 (30 mm in Easting and 3 mm in Northing coordinates).

Table 5.5: The horizontal coordinates of all stations
from STARNET Pro software.

	EPOCH 1		EPOCH 2		DIFFERENT	
Station	Easting (m)	Northing (m)	Easting (m)	Northing (m)	ΔE (m)	ΔN (m)
KTPK	135106.584	351068.135	135106.584	351068.135	0.000	0.000
W474	137519.188	356413.420	137519.188	356413.420	0.000	0.000
WM1	138477.581	353737.740	138477.565	353737.736	0.016	0.004
WM3	138459.201	353772.301	138459.195	353772.288	0.006	0.013
WM5	138451.116	353816.161	138451.075	353816.158	0.041	0.003
WM7	138453.526	353852.844	138453.523	353852.840	0.003	0.004
WM8	138482.713	353893.787	138482.675	353893.727	0.038	0.060
WM9	138487.770	353873.677	138487.761	353873.668	0.009	0.009
WM11	138464.189	353841.907	138464.175	353841.905	0.014	0.002
WM13	138465.063	353815.024	138465.062	353815.023	0.001	0.001
WM15	138472.567	353795.665	138472.566	353795.665	0.001	0.000
WM18	138475.623	353837.904	138475.601	353837.899	0.022	0.005
WM20	138483.033	353802.944	138483.018	353802.908	0.015	0.036
WM21	138490.169	353809.464	138490.163	353809.443	0.006	0.021
WM23	138486.258	353828.189	138486.221	353828.141	0.037	0.048
WM25	138496.238	353849.875	138496.217	353849.835	0.021	0.040
WM26	138495.471	353823.942	138495.464	353823.910	0.007	0.032
WM28	138499.991	353817.883	138499.961	353817.880	0.030	0.003

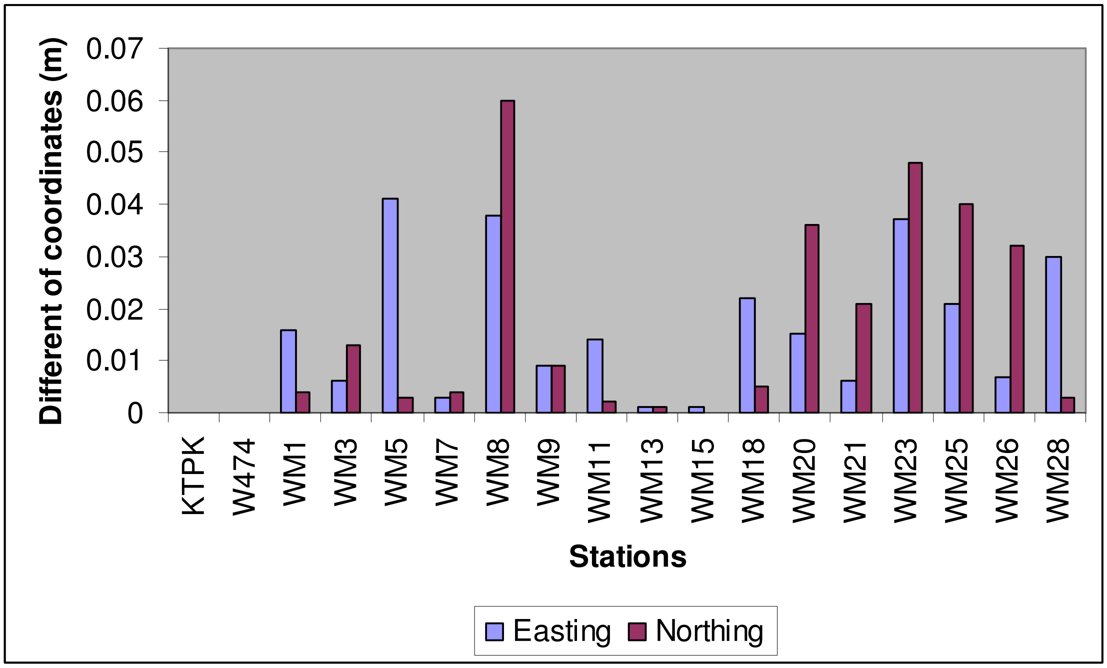


Figure 5.10: Difference of horizontal component for Epoch 1 and Epoch 2

Table 5.6: The height of all stations

from STARNET Pro sortware.

	EPOCH 1	EPOCH 2	DIFFERENT
Station	h (m)	h (m)	Δh (m)
KTPK	99.267	99.267	0.000
W474	69.030	69.030	0.000
WM1	92.074	92.043	0.031
WM3	85.903	85.872	0.031
WM5	87.090	87.079	0.011
WM7	86.389	86.357	0.032
WM8	84.527	84.492	0.035
WM9	92.823	92.802	0.021
WM11	95.539	95.521	0.018
WM13	96.708	96.704	0.004
WM15	97.574	97.573	0.001
WM18	105.259	105.225	0.034
WM20	107.134	107.104	0.030
WM21	112.522	112.512	0.010
WM23	113.962	113.924	0.038
WM25	112.204	112.184	0.020
WM26	118.793	118.776	0.017
WM28	83.024	82.991	0.033

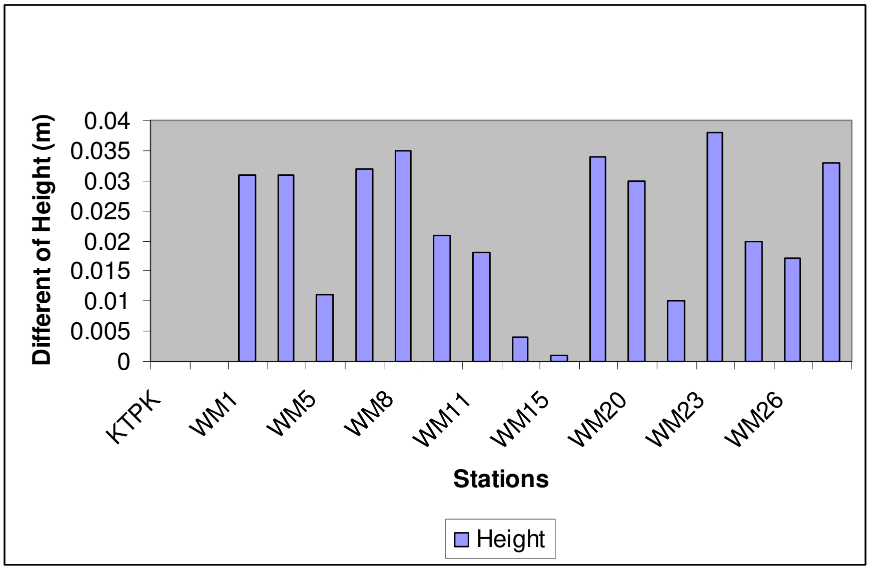


Figure 5.11: Difference of vertical component between Epoch 1 and Epoch 2

The different of coordinates for the horizontal component between the first epoch and second epoch for each monitoring station can be show by plotting the vector line for both epochs. Figure 5.13 until 5.18 shows the vector line for six stations namely WM5, WM8, WM18, WM20, WM23 and WM25.

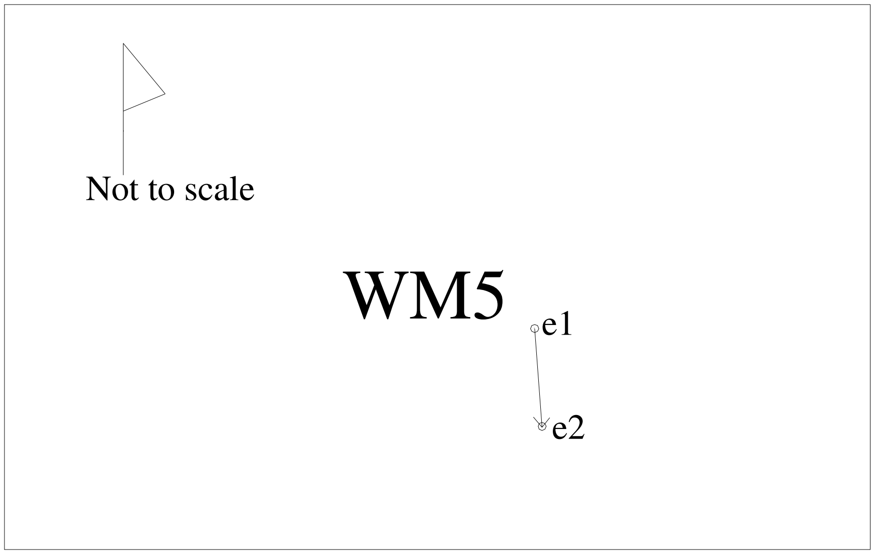


Figure 5.13: Vector for station WM5

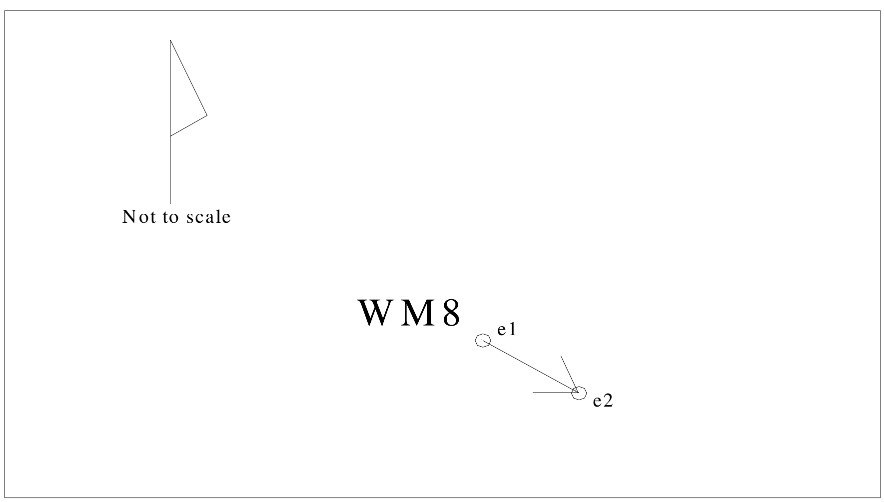


Figure 5.14: Vector for station WM8

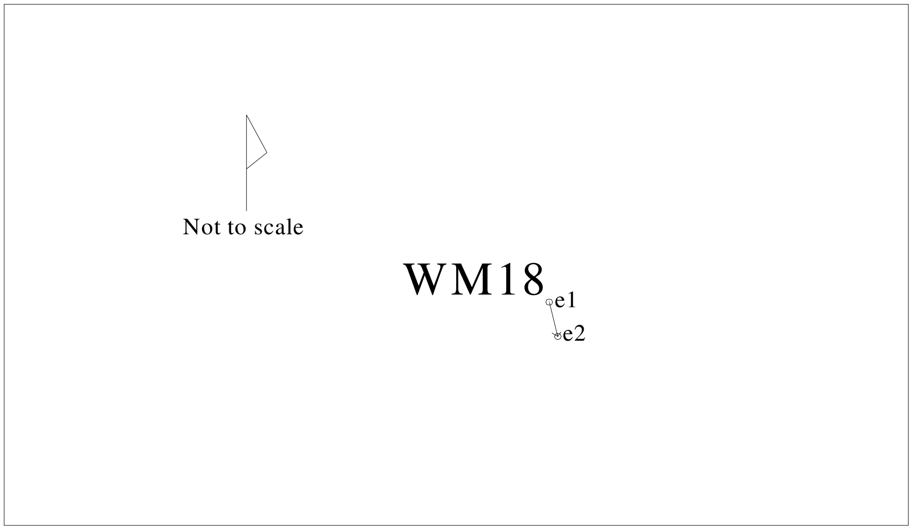


Figure 5.15: Vector for station WM18

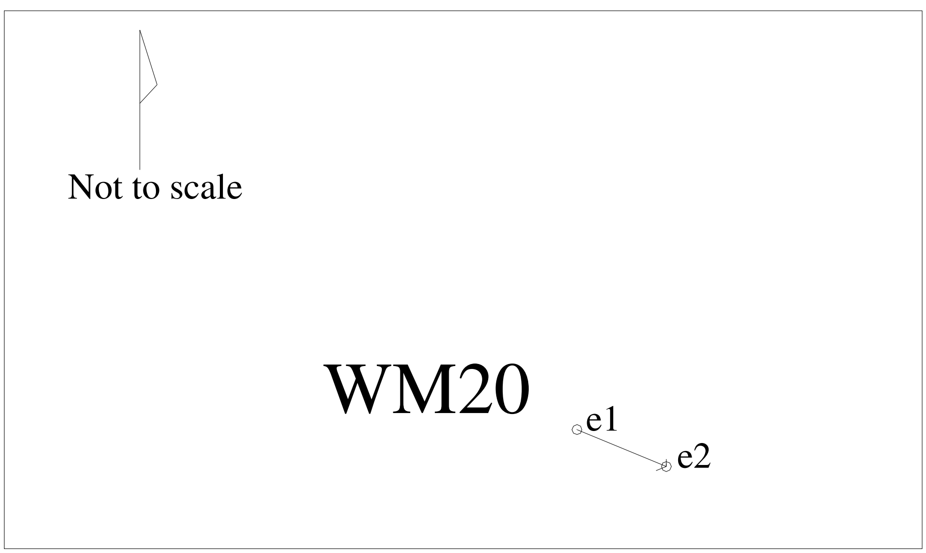


Figure 5.16: Vector for station WM20

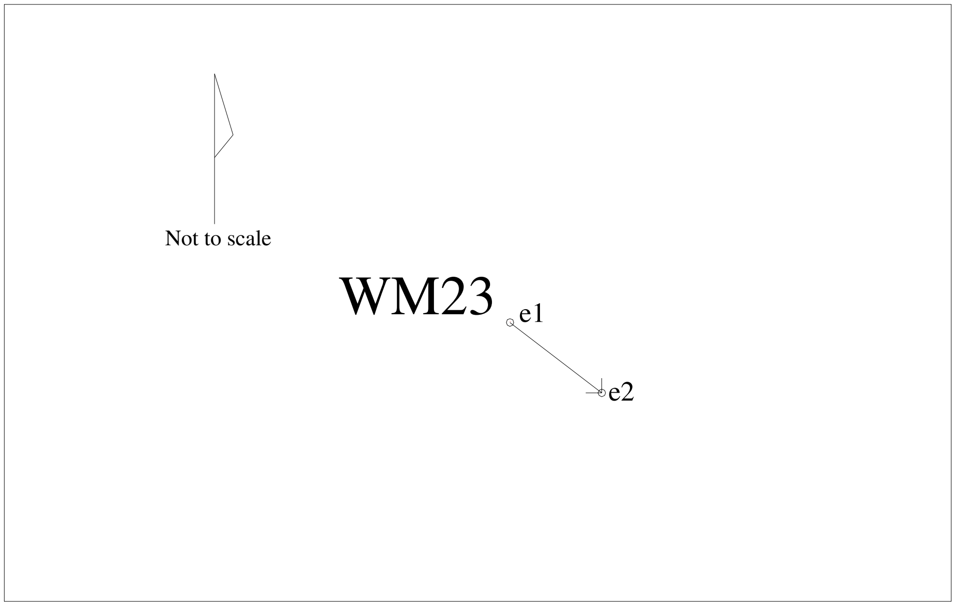


Figure 5.17: Vector for station WM23

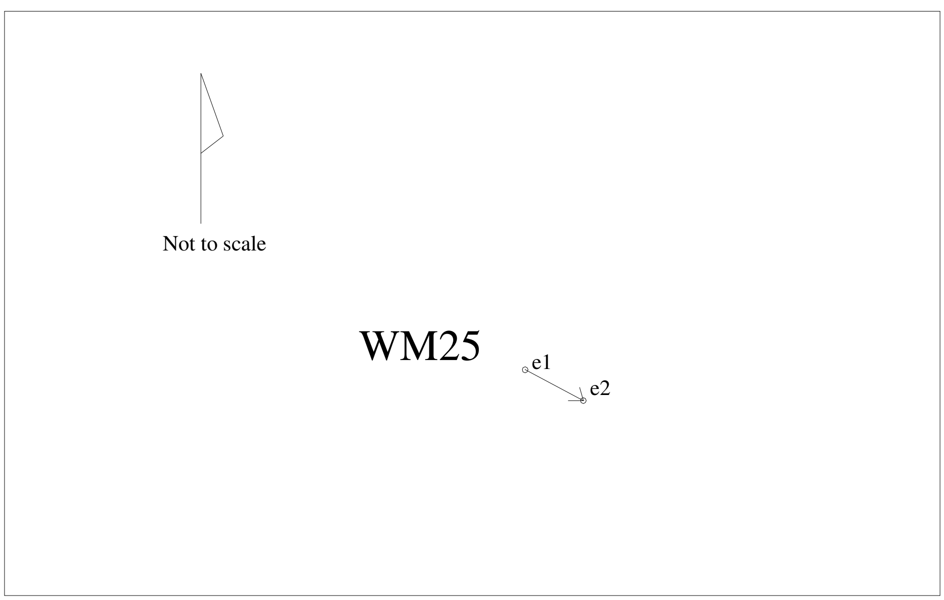


Figure 5.18: Vector for station WM25

5.5 Landslide Movement Detection

A statistical test known as the congruency test was applies to determine whether significant movements of the structures have occurred between the two epochs of observations. The application of congruency test was very simple. Initially, the congruency of common datum points at each epoch was tested by the global congruency test. If the test indicates significant movements, localization was performed, followed by a similar test on the remaining datum points through the partial congruency test. The GPSAD2000 software was used to analyze the stability of the all 16 points located at the study area. Table 5.7 and 5.8, shows the baseline vectors for all 16 stations with two fixed points for the 1st epoch and 2nd epoch, respectively. The deformation analysis was also carried out for both epochs. Table 5.9 shows that the variance ratio test at significance level 0.05 was successful for all monitoring points where the value is smaller than the critical value (1.541 < 2.620). This shows that the variance factor is the same for both epochs of observations. The corresponding test has shown that the observation for both campaigns was successful which confirmed that all reference points and monitoring points was in stable conditions.

Consequently, results from Table 5.9, shows that all the datum points and object points also passed the single points test at significance level 0.01. Again it indicates that all monitoring points at the study area were considered to be in stable conditions. Although, the different of coordinate shows there are some movement at seven stations but it does not mean that the landslide has occur at the study area. It has been proven by the deformation analysis which has clarified that the area is not prone to landslide. But, the monitoring work must be carry out at least one a year because it still has the potential of mass movement prone at the area. It was proven by the geotechnical method which found that this area was covered by sandy clay. This type of soil has the lowest of moisture content and the structure of the soil was quiet loose and has potential to loose if there a lot of rainfalls occur at the area.

Table5.7: Baselines vector for the 1st epoch Observations

From	To	X (m)	Y(m)	Z(m)
KTPK	WM1	-3256.622	-835.491	2672.624
KTPK	WM3	-3236.919	-839.256	2706.574
KTPK	WM5	-3228.853	-839.289	2750.542
KTPK	WM7	-3230.205	-842.367	2786.880
KTPK	WM8	-3257.755	-852.390	2827.697
KTPK	WM9	-3264.720	-844.136	2808.115
KTPK	WM11	-3242.624	-835.075	2776.506
KTPK	WM13	-3243.929	-832.866	2749.782
KTPK	WM15	-3251.930	-832.279	2730.546
KTPK	WM18	-3255.882	-827.615	2773.107
KTPK	WM20	-3264.009	-825.455	2738.370
KTPK	WM21	-3271.973	-821.929	2745.183
KTPK	WM23	-3268.200	-820.566	2763.983
KTPK	WM25	-3277.160	-825.448	2785.485
KTPK	WM26	-3277.624	-817.383	2760.179
KTPK	WM28	-3219.483	-842.448	2751.747
W474	WM1	-980.246	-28.066	-2663.670
W474	WM3	-961.445	-31.569	-2629.850
W474	WM5	-952.660	-31.580	-2585.910
W474	WM7	-954.342	-34.583	-2549.520
W474	WM8	-981.887	-44.562	-2508.700
W474	WM9	-989.069	-37.298	-2527.860
W474	WM11	-966.754	-27.244	-2559.880
W474	WM13	-968.201	-24.861	-2586.630
W474	WM15	-976.021	-24.430	-2605.870
W474	WM18	-981.240	-18.986	-2563.860
W474	WM20	-988.408	-17.331	-2598.060
W474	WM21	-996.870	-14.257	-2591.110
W474	WM23	-992.198	-13.269	-2572.450
W474	WM25	-1001.366	-17.559	-2550.870
W474	WM26	-1001.695	-9.916	-2576.620
W474	WM28	-943.602	-34.639	-2584.650

Table 5.8: Baselines vector for the 2nd epoch Observations

From	To	X (m)	Y(m)	Z(m)
KTPK	WM1	-3256.642	-835.481	2672.634
KTPK	WM3	-3236.917	-839.247	2706.544
KTPK	WM5	-3228.860	-839.287	2750.562
KTPK	WM7	-3230.250	-842.356	2786.860
KTPK	WM8	-3257.754	-852.378	2827.647
KTPK	WM9	-3264.740	-844.115	2808.125
KTPK	WM11	-3242.653	-835.065	2776.536
KTPK	WM13	-3243.919	-832.874	2749.719
KTPK	WM15	-3251.920	-832.266	2730.524
KTPK	WM18	-3255.833	-827.635	2773.121
KTPK	WM20	-3264.021	-825.445	2738.356
KTPK	WM21	-3271.960	-821.915	2745.153
KTPK	WM23	-3268.221	-820.564	2763.986
KTPK	WM25	-3277.147	-825.434	2785.475
KTPK	WM26	-3277.613	-817.379	2760.165
KTPK	WM28	-3219.473	-842.451	2751.732
W474	WM1	-982.779	-28.3825	-2664.139
W474	WM3	-960.725	-32.7027	-2629.91
W474	WM5	-952.534	-31.5189	-2586.153
W474	WM7	-954.279	-34.7575	-2549.537
W474	WM8	-981.431	-45.1291	-2508.52
W474	WM9	-989.069	-37.2985	-2527.862
W474	WM11	-966.765	-27.2345	-2559.877
W474	WM13	-968.205	-24.8659	-2586.629
W474	WM15	-976.021	-24.4301	-2605.866
W474	WM18	-979.993	-19.8569	-2563.304
W474	WM20	-988.202	-17.6425	-2597.936
W474	WM21	-996.063	-14.2316	-2591.237
W474	WM23	-992.24	-12.8307	-2572.54
W474	WM25	-1001.39	-18.0116	-2550.962
W474	WM26	1002.312	9.9737	2576.444
W474	WM28	943.9361	33.0063	2584.563

Table 5.9: Single point test result

Stn	Dx	Dy	Dz	Dis.Vec	fcom	Ftab	Info
KTPK	-0.0018	-0.008	0.0262	0.0275	0.02	4.22	Stable
W474	-0.0018	-0.008	0.0262	0.0275	0.02	4.22	Stable
W1	0.789	-0.1335	0.1729	0.8187	0.04	4.22	Stable
W3	-0.1132	0.0583	-0.3158	0.3405	0.06	4.22	Stable
W5	-0.3563	-0.132	0.0432	0.3824	0.06	4.22	Stable
W7	0.003	-0.0114	-0.0726	0.0736	0.03	4.22	Stable
W8	-0.0398	0.0517	-0.1082	0.1263	0.08	4.22	Stable
W9	-0.0298	-0.0077	-0.1886	0.1911	0.01	4.22	Stable
W11	0.0113	-0.0101	0.0445	0.047	0.01	4.22	Stable
W13	-0.0015	-0.0091	0.0225	0.0243	0.01	4.22	Stable
W15	-0.0966	0.0019	-0.014	0.0976	0.02	4.22	Stable
W18	0.0198	-0.0132	0.06	0.0645	0.02	4.22	Stable
W20	0.0137	0.017	-0.0243	0.0327	0.01	4.22	Stable
W21	-0.1046	-0.0196	-0.0039	0.1065	0.02	4.22	Stable
W23	-0.1341	-0.0865	0.2947	0.3351	0.02	4.22	Stable
W25	0.0227	-0.0391	-0.2934	0.2969	0.01	4.22	Stable
W26	0.5085	0.1402	0.0437	0.5293	0.38	4.22	Stable
W28	0.0699	0.0048	0.6077	0.6117	0.01	4.22	Stable

CHAPTER VI

CONCLUSION AND RECOMMANDATION

6.1 Conclusion

Landslide is an important issue and may cause considerable change on the structure or region. Hence, it may cause damage on the engineering structures such as buildings, roads, dams, subways and harbours. Therefore, monitoring the landslides and solving their mechanism are very important to prevent and to reduce their negative effects. The new mechanism includes the differentiation of moving blocks, identification of the direction of movement, location and shape of slide plane, nature of landslide block, possibility of future movement on slopes above the existing slide, and distribution of ground water.

The stability condition of naturally or man made slope can be achieved only through the proper monitoring and analysis of deformable bodies. Therefore, deformation-monitoring systems should be set up to assess the soil changes that may occur over the time. Various deformation techniques have been used in the past. The techniques can be divided into two categories such as geodetic and non-geodetic methods. The geodetic measurement techniques generally can be divided into terrestrial (e.g. precise leveling) and satellite-based geodetic methods, e.g. GPS technology. Non-geodetic technique can be divided into geotechnical method, (e.g. inclinometer, extensometer) and geophysical methods. The selection of the most appropriate technique or combination of techniques for any particular deformation application depends on the cost, the accuracy required, and the scale of the surveying involved. Therefore several aspects related to the network design, measurements and analysis techniques suitable to the monitoring surveys have to be considered. The design of monitoring scheme should satisfy not only the best geometrical strength of the network, i.e. with adequate redundancy and bracing, but should also primarily fulfill the needs of subsequent physical interpretation of the monitoring results.

A total of two datum points namely KTPK and W474 were used as the control (reference) in the GPS deformation network. The deformation network scheme was also configured by eighteen monitoring (object) points, which were marked as WM1, WM3, WM5, WM7, WM8, WM9, WM11, WM13, WM15, WM18, WM20, WM21, WM23, WM 25, and WM28. All of them were monuments at the beam along the slope. WM1, WM3 and WM5 were marked at the foot of the slope, while, WM28 was marked at the top of the slope. In this study, it was found that for such deformation-monitoring-system, the combination and corporation between the geodetic and geotechnical methods offer several advantages over individual methods, especially in terms of results and field procedures.

Effective and efficient monitoring techniques are needed to minimise the landslides effects. Although there are various types of conventional instruments and investigation, GPS has the potential in landslides monitoring continuously. GPS technology offers several advantages such as all kind of weather, the service is free worldwide and anyone with a receiver can receive the signals and locate a position, the system supports unlimited users simultaneously and it provides navigation capability. GPS has been used in many applications including landslides monitoring. It also give the boundary of the landslides area from the field observation. The measurement of tilt is usually understood as a determination of a deviation from a horizontal plane, while inclination interpreted as a deviation from the vertical. The inclinometer can be applied to determine the zone of landslide movement, the magnitude and rate of movement.

In this study, the GPS survey campaigns have been carried out at the residential area of Section 5, Wangsa Maju, for two epochs whereby a total of three observations have been carried out for each epoch. In the geotechnical survey campaigns the Mackintosh Probe and Laboratory tests for the disturbed sample at a total of one day observation has been done. The designed monitoring network was properly adjusted and analyses before the results are subsequently being used for the deformation analysis.

The analysis of deformation survey for the GPS observation has been performed using the GPSAD2000 software. The analysis of deformation surveys includes:

1. Geometrical analysis which describes the geometrical status of the deformable body, its change in shape and dimensions, as well as the translation and rotation of the whole deformable body with respect to a stable references frame; and
2. Physical interpretation.

A statistical test known as the congruency test is required to determine whether significant movements have occurred between the two epochs for the GPS observations by using the GPSAD2000 software. In the constraint adjustment, the results of the stability determination with two fixed points have shown that the variance ratio test at significance level 0.05 is successful for all three days of GPS observations. Finally, it has been shown that from the single points test analysis at significance level 0.01, all the monitoring stations were confirmed in stable condition, i.e. no movement during and after the reconstruction of the slope. From all observation and analysis technique, it can be stated that from this study, the slope near the residential area of Section 5, Wangsa Maju still exhibits a safe deformation behavior. This also shows that the combination of both GPS and geotechnical methods can improve the landslide movement result, for a successful comment, planning and action to be presented.

6.2 Recommendation

The results of the deformation surveys test demonstrate that the geotechnical methods by subsurface investigation and the satellite-based method (GPS survey) have the potential to be employed for monitoring the landslide behavior. However, from the work done in this study, the following recommendations are considered to be worthy for future investigations:

- More research is required to fully understand all sources of errors and their influence on GPS and other geotechnical instruments. Results for high precision deformation surveys because some anomalies in these results still occur (systematically or randomly) in geodetic measurements which cannot yet be fully explained. Therefore, ideally, the subsequent surveys would exactly repeat the survey deformation campaign throughout the years, in terms of sites occupied, the survey method used, processing techniques and analysis.

- The GPS observation should be extended to one hour observation so that a better GPS data can be processed to get more information.
- This study also can be extended by observing the underground water level to see the trend of water flow at the study area and it can also be integrated with GPS data to have the good view of the slope movement at the study area.
- One GIS landslide map should be developed and all data should be combined in the GIS database with the 3D view of the study area, so that it can be referred by those interested in landslide study.

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APPENDIX A

The Information of BH 3 Borehole

STRATA DRILL SDN. BHD (Co. No. 265855-P)

Sheet 2 of 3
Job no. SD/2580/2001
Borehole BH-3

CLIENT	: PGK SDN. BHD.	DATE STARTED	: 28.4.2001
CONSULTANT	: PERUNDING MAJUCIPTA SDN. BHD.	DATE COMPLETED	: 3.5.2001
PROJECT	: PROPOSED DEVELOPMENT AT SECTOR R12, BANDAR WANGSA MAJU, WILAYAH PERSEKUTUAN, KUALA LUMPUR	FINAL W.L.	: Nil
		COORDINATES	: N 2427.061 : E 5073.511
DRILL TYPE	: ROTARY WASH BORING	REDUCED LEVEL	126.78m.

Depth m	Sample no	Depth m	Thickness m	Soil Description & Lithology	R.Ratio %	Field Test & Sampling S.P.T./Vane Shear 75 mm	75 mm	75 mm	75 mm	75 mm	75 mm	N	Remark
10.50 10.95	P4/D5		3.00	Stiff, yellowish brown to white, slighlty clayey SILT with some sand and gravels	44	2	2	3	3	4	4	14	
12.00 13.00	MS4			No recovery	Nil								
13.50 13.95	P5/D6	13.50	1.50	Very stiff, yellowish brown to white, slighlty clayey sandy SILT with some gravels	40	2	2	3	5	5	12	25	
15.00 16.00	MS5	15.00	1.50	Very stiff, yellowish brown to white, slighlty clayey sandy SILT with traces of gravels	40								
16.50 16.95	P6/D7	16.50	1.00	Hard, yellowish brown to white, clayey SILT with some sand and traces of gravels	40	3	5	6	10	13	15	44	
18.00 19.00	MS6	17.50	5.00	Hard, yellowish brown mottled with white slighlty clayey sandy SILT with traces of gravels	35								
19.50 19.95	P7/D8			Ditto	31	4	7	7	6	12	18	43	

V Vane Shear Test
UD Undisturbed Sample
P Standard Penetration Test
C Rock Coring
MS Mazier
D Disturbed Sample
N No. of Blows/300 mm
• No Recovery

Cohesive Soil
0 - 2 = Very Soft   2 - 4 = Soft
4 - 8 = Medium Stiff   8 - 15 = Stiff
15 - 30 = Very Stiff   >30 = Hard

Non Cohesive Soil
0 - 4 = Very Loose   4 - 10 = Loose
10 - 30 = Medium Dense   30 - 50 = Dense
> 50 = Very Dense

Supervisor : William
Check by : Chia

APPENDIX B

The Information of BH 4 Borehole

STRATA DRILL SDN. BHD (Co. No. 265855-P)

Sheet 1 of 1
Job no. SD/2580/2001

Borehole BH-4

CLIENT : PGK SDN. BHD
CONSULTANT : PERUNDING MAJUCIPTA SDN. BHD.
PROJECT : PROPOSED DEVELOPMENT AT SECTOR R12, BANDAR WANGSA MAJU, WILAYAH PERSEKUTUAN, KUALA LUMPUR
DRILL TYPE : ROTARY WASH BORING

DATE STARTED : 18.4.2001
DATE COMPLETED : 18.4.2001
FINAL W.L. : Nil
COORDINATES : N2377.907 E5195.056
REDUCED LEVEL : 172.03m

Depth m	Sample no	Depth m	Thickness m	Log	Soil Description & Lithology	R.Ratio %	Field Test & Sampling S.P.T./Vane Shear 75 mm	75 mm	75 mm	75 mm	75 mm	75 mm	N	Remark
0.00 0.15	D1	0.15	0.15		Top Soil : Yellowish brown, clayey SILT with traces of sand and decomposed organic material									
1.50 1.95	P1/D2		2.85		Stiff, yellowish brown, clayey SILT with some sand and traces of gravels	55	1	2	2	2	2	2	8	
3.00 4.00	MS1	3.00	1.50		Stiff, yellowish brown, slighhlty clayey SILT with some sand	50								
4.50 4.95	P2/D3	4.50	0.83		Very stiff, yellowish brown, clayey SILT with some sand and traces of gravels	33	3	4	5	8	6	4	20	
5.30 6.33	P4/D4	5.33			Hard, white to yellow brown, - ditto -	100	50 /30 mm						50 /30 mm	1000hrs 18.4.2001 wl. Nil

End of borehole at 6.33m

V Vane Shear Test
UD Undisturbed Sample
P Standard Penetration Test
C Rock Coring
MS Mazier
D Disturbed Sample
N No. of Blows/300 mm
* No Recovery

Cohesive Soil
0 - 2 = Very Soft 2 - 4 = Soft
4 - 8 = Medium Stiff 8 - 15 = Stiff
15 - 30 = Very Stiff >30 = Hard

Non Cohesive Soil
0 - 4 = Very Loose 4 - 10 = Loose
10 - 30 = Medium Dense 30 - 50 = Dense
> 50 = Very Dense

Supervisor : William
Check by : Chia

APPENDIX C

The Information of BH 6 Borehole

STRATA DRILL SDN. BHD (Co. No. 265855-P)

Sheet 1 of 1
Job no. SD/2580/2001

Borehole BH-6

CLIENT : PGK SDN. BHD.
CONSULTANT : PERUNDING MAJUCIPTA SDN. BHD.
PROJECT : PROPOSED DEVELOPMENT AT SECTOR R12, BANDAR WANGSA MAJU, WILAYAH PERSEKUTUAN, KUALA LUMPUR
DRILL TYPE : ROTARY WASH BORING

DATE STARTED : 4.5.2001
DATE COMPLETED : 5.5.2001
FINAL W.L. : Nil
COORDINATES : N 2292.755
: E 5027.448
REDUCED LEVEL : 106.38m

Depth m	Sample no	Depth m	Thick-ness m	Soil Description & Lithology	R.Ratio %	Field Test & Sampling S.P.T./Vane Shear (75 mm ×6)	N	Remark
0.00 – 0.95	D1	1.00	1.00	Top Soil : Pale yellowish grey, sandy clayey SILT with traces of quartz gravels and root				
1.00	C1			Medium strong, whitish grey to pale brown, slightly fractured SILTSTONE	93	CL : 1.50m RQD : 43%		
2.50	C2	3.50	2.50	Ditto	90	CL : 1.50m RQD : Nil		1800hrs 4.5.2001 wl. Nil 0830hrs 5.5.2001 wl. Nil
3.50		4.50	1.00	Sandy, highly weathered rock, SILTSTONE				
4.50	C3			Medium strong, whitish light grey to pale brown, spotted with red, slighlty fractured SILSTONE	100	CL : 1.50m RQD : 47%		
6.00	C4	7.50	3.00	Medium strong, whitish light grey to brown, slightly fractured SILTSTONE	100	CL : 1.50m RQD : 43%		
7.50				End of Borehole at 7.50m				1400hrs 5.5.2001 wl. Nil

V Vane Shear Test
UD Undisturbed Sample
P Standard Penetration Test
C Rock Coring
MS Mazier
D Disturbed Sample
N No. of Blows/300 mm
* No Recovery

Cohesive Soil
0 - 2 = Very Soft 2 - 4 = Soft
4 - 8 = Medium Stiff 8 - 15 = Stiff
15 - 30 = Very Stiff >30 = Hard

Non Cohesive Soil
0 - 4 = Very Loose 4 - 10 = Loose
10 - 30 = Medium Dense 30 - 50 = Dense
> 50 = Very Dense

Supervisor : Zani
Check by : Chia

APPENDIX D

The Laboratory Test

SUMMARY OF TEST RESULTS

GEOCRETE LABORATORY SENDIRIAN BERHAD (Co. No.417880 - H)

PROJECT : PROPOSED DEVELOPMENT AT SECTOR R12 BANDAR BARU WANGSA MAJU, K.L

REF : P012/(L/70/01/026/01
DATE : 13/06.2001

SAMPLE AND SPECIMEN DETAILS.						ATTERBERG LIMITS				SIEVE AND HYDROMETER ANALYSIS							CIU TEST		UU TEST		CONSOLIDATION TEST		CHEMICAL TEST				
Borehole No.	Specimen	Depth (m)	Moisture Content (%)	Bulk Density (Mg/m3)	Dry Density (Mg/m3)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Linear Srinkage (%)	Clay (%)	Silt (%)	Sand (%)	Gravels (%)	Specific Gravity	Unconfined Compression Test	Point Load Test	C' (kPa)	φ' (Deg)	Cu (kPa)	φ (Deg)	Pc (kPa)	Cc	Organic Content (%)	SO3 (%)	Cl (%)	pH	Loss On Ignition (%)
BH3	P1/D2	1.50	24			58	30	28		3	55	34	8														
	MS1	3.00	22	2.069	1.690	59	32	27		3	64	32	1				0	32.0									
	P2/D3	4.50	31			61	34	27		3	70	26	1											<.01		5.1	
	P3/D4	7.50	35			63	34	29		16	66	17	1														
	MS3	9.00	32			61	33	28		4	60	29	7														
	P4/D5	10.50	30			62	35	27		9	61	16	14														
	P5/D6	13.50	28			57	38	19		4	52	30	14														
	MS5	15.00	19	2.094	1.760	56	37	19		6	52	35	7				0	35.0									
	P6/D7	16.50	29			59	38	21		14	59	25	2														
	MS6	18.00	30			60	38	22		2	60	32	6														
	P7/D8	19.50	28			49	30	19		3	51	37	9														
	P8/D9	22.50	22			50	30	20		4	52	36	8														
	P9/D10	25.50	18			44	31	13		13	24	54	9														
	P10/D11	27.00	15			42	28	14		3	35	46	16														

Note : NES = NOT ENOUGH SAMPLE NP = NON PLASTIC NA = NOT APPLICABLE

APPENDIX E

The Laboratory Test

SUMMARY OF TEST RESULTS

GEOCRETE LABORATORY SENDIRIAN BERHAD (Co. No.417880 - H)	PROJECT : PROPOSED DEVELOPMENT AT SECTOR R12 BANDAR BARU WANGSA MAJU, K.L	REF : P012/(L/70/01/026/01 DATE : 13/06.2001

SAMPLE AND SPECIMEN DETAILS. Borehole No.	Specimen	Depth (m)	Moisture Content (%)	Bulk Density (Mg/m3)	Dry Density (Mg/m3)	ATTERBERG LIMITS Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Linear Srinkage (%)	SIEVE AND HYDROMETER ANALYSIS Clay (%)	Silt (%)	Sand (%)	Gravels (%)	Specific Gravity	Unconfined Compression Test	Point Load Test	CIU TEST C ' (kPa)	φ ' (Deg)	UU TEST Cu (kPa)	φ (Deg)	CONSOLIDATION TEST Pc (kPa)	Cc	CHEMICAL TEST Organic Content (%)	SO3 (%)	Cl (%)	pH	Loss On Ignition (%)
BH4	P1/D2	1.50	30			41	26	15		18	58	18	6														
	MS1	3.00	21	2.023	1.672	40	27	13		3	68	28	1				0	31.0						0.02		4.6	
	P2/D3	4.50	26			40	39	11		10	62	24	4														
	P4/D4	5.30	24			40	27	13		21	56	19	4														
BH5	P1/D2	1.50	29			41	29	12		22	63	13	2														
BH7	P1/D2	1.50	11			NP				16		25	59														
	MS1	3.00	23			41	25	16		6	51	32	11														
	MS3	6.00	21	2.084	1.722	39	28	11		7	61	27	5				0	33.0						<0.01		4.8	
	P2/D3	7.5	14			37	29	8		3	45	18	34														

Note : NES = NOT ENOUGH SAMPLE NP = NON PLASTIC NA = NOT APPLICABLE

APPENDIX F

The Mackintosh Result

MACKINTOSH PROBE

UNIVERSITI TEKNOLOGI MALAYSIA

Project/Location: SECTION 5, WANGSA MAJU

Client: KUALA LUMPUR (WM 3)

Date of Testing: 18/7/2004

MP1				
Depth (m)	Blows	mm	Unconfined Compressive Strength	
0.0 - 0.3	4			
0.3 - 0.6	9			
0.6 - 0.9	17			
0.9 - 1.2	143			
1.2 - 1.5	200			
1.5 - 1.8	400			
1.8 - 2.1				
2.1 - 2.4				
2.4 - 2.7				
2.7 - 3.0				
3.0 - 3.3				
3.3 - 3.6				
3.6 - 3.9				
3.9 - 4.2				
4.2 - 4.5				
4.5 - 4.8				
4.8 - 5.1				
5.1 - 5.4				
5.4 - 5.7				
5.7 - 6.0				
6.0 - 6.3				
6.3 - 6.6				
6.6 - 6.9				
6.9 - 7.2				
7.2 - 7.5				
7.5 - 7.8				
7.8 - 8.1				
8.1 - 8.4				
8.4 - 8.7				
8.7 - 9.0				
9.0 - 9.3				
9.3 - 9.6				
9.6 - 9.9				
9.9 - 10.2				
10.2 - 10.5				
10.5 - 10.8				
10.8 - 11.1				
11.1 - 11.4				
11.4 - 11.7				
11.7 - 12.0				
12.0 - 12.3				
12.3 - 12.6				
12.6 - 12.9				
12.9 - 13.2				
13.2 - 13.5				
13.5 - 13.8				
13.8 - 14.1				
14.1 - 14.4				
14.4 - 14.7				
14.7 - 15.0				
15.0 - 15.3				
15.3 - 15.6				
15.6 - 15.9				
15.9 - 16.2				
16.2 - 16.5				
16.5 - 16.8				
16.8 - 17.1				
17.1 - 17.4				
17.4 - 17.7				
17.7 - 18.0				
18.0 - 18.3				
18.3 - 18.6				
18.6 - 18.9				
18.9 - 19.2				
19.2 - 19.5				
19.5 - 19.8				
19.8 - 20.1				
20.1 - 20.4				
20.4 - 20.7				
20.7 - 21.0				
21.0 - 21.3				
21.3 - 21.6				
21.6 - 21.9				
21.9 - 22.2				
22.2 - 22.5				
22.5 - 22.8				
22.8 - 23.1				
23.1 - 23.4				
23.4 - 23.7				
23.7 - 24.0				
24.0 - 24.3				
24.3 - 24.6				
24.6 - 24.9				
24.9 - 25.2				

Final Depth 1.660 m

R. Level m

Water Table 1.030 m

Notes: 1 Ton/Sq.ft = 100 kPa